



Warblington Road
Emsworth
(9 Unit Scheme)

FLOOD RISK ASSESSMENT

On behalf of
Mr. Charles Glanville

ITR/MT/5042.FRA.7
August 2020

Experience and expertise working together

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1.0 INTRODUCTION

1.1 Bellamy Roberts LLP has been retained by Mr. Charles Glanville to prepare a Flood Risk Assessment (FRA) in support of a planning application for a residential development of 9 dwellings on land south of Warblington Road, Emsworth. This assessment has been undertaken to ascertain whether there are any constraints to the development of this site, and to assess the impact of the proposal, with regard to flood risk.

1.2 The site has an approximate area of 0.83ha and is located in Flood Zone 1. Government guidance states that Flood Risk Assessments are generally only necessary in Flood Zone 1 when the development area is greater than 1ha. However, an FRA is required for developments of less than 1ha when the proposed development will provide a more vulnerable use class. This Assessment has been prepared to accompany an indicative sustainable drainage design, and will also consider the expected greenfield run-off rates, soil types, and Environment Agency (EA) Flood Mapping.

1.3 The FRA has been updated to address consultation comments received from the Environment Agency (23rd June 2020) who have objected to the proposal. A copy of the EA consultation comments are provided at **Appendix 1**, however in summary the EA has requested that an appropriate freeboard is added to the design flood levels for 2115 to achieve Finished Floor Levels (FFLs) that will make the development flood resilient for its lifetime.

Site Location and Features

1.4 The National Grid reference for the site is 474407, 105452. For clarity, a site location plan is provided at Figure 1.

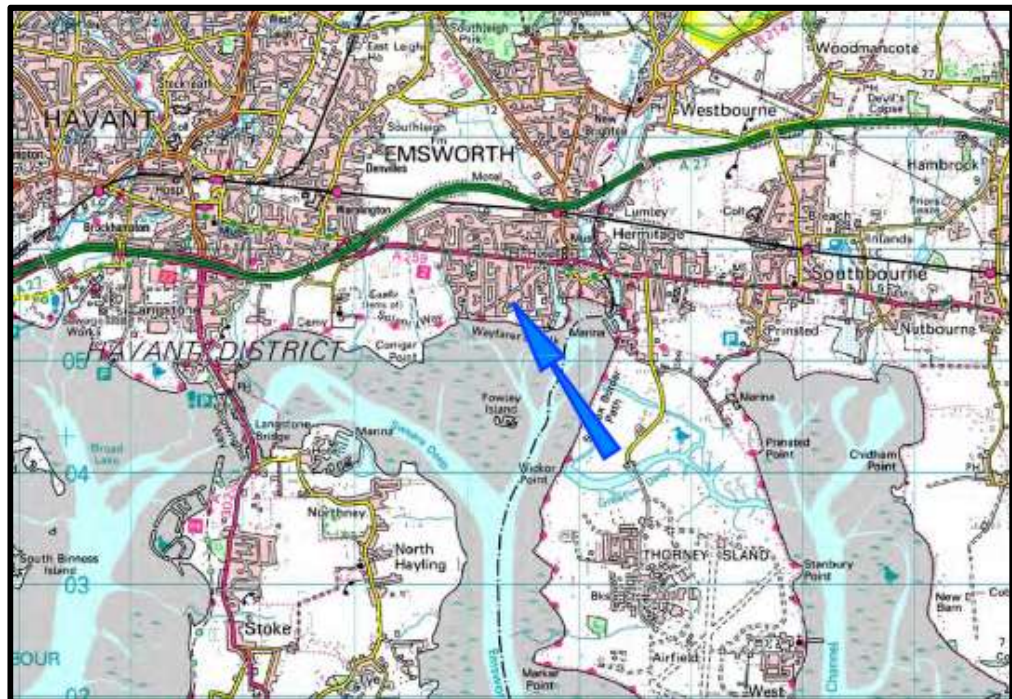


Figure 1: Site Location Plan

Site Topography

- 1.5 The topography falls from 6.75m in the northern corner to 1.98m in the southern corner. See Figure 2 on the following page.



Figure 2: (Source LIDAR) – Topography of site.

- 1.6 The site is accessed off Warblington Road which borders the northern boundary of the site. Residential development is located on all sides other than the southern boundary which is bounded by Chichester Harbour.

Appropriateness of Development Proposal

- 1.7 National guidance states that Flood Zone 1 is suitable for all types of development. Reproduced at Figure 3 is an extract from the EA Flood Zone map which demonstrates that the proposed dwellings will lie within Flood Zone 1. The southern extent of the application site is shown to be located within Flood Zone 3, and it will be ensured that no dwellings will be located within this area. The site access will also be located solely within Flood Zone 1.

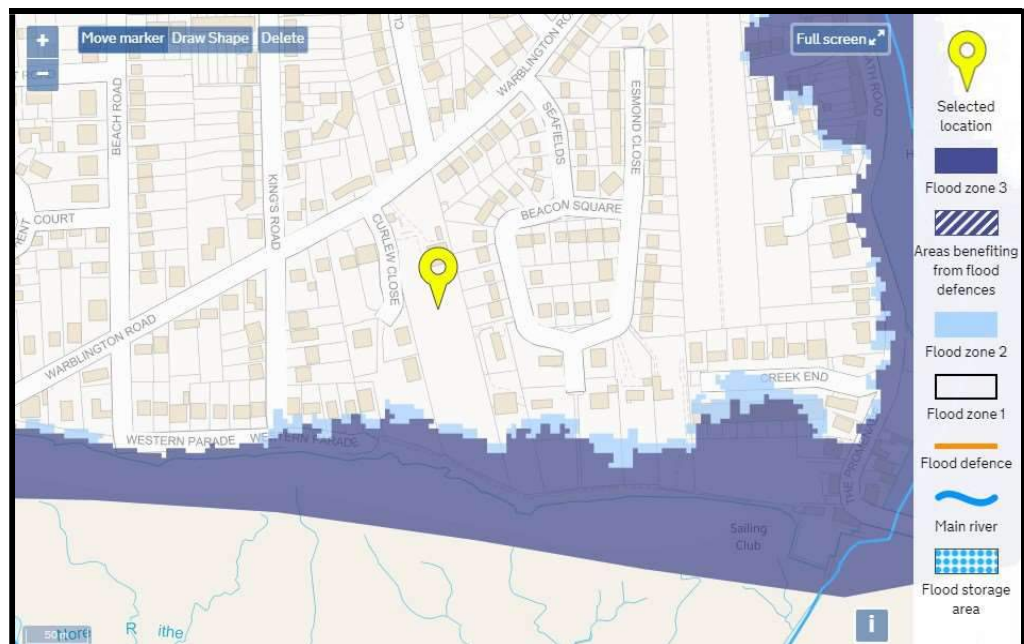


Figure 3: Flood Zone Map (Source: EA)

1.8

An extract from the EA surface water flood risk map is provided in Figure 4. This demonstrates that the site is not at risk of flooding from this source.

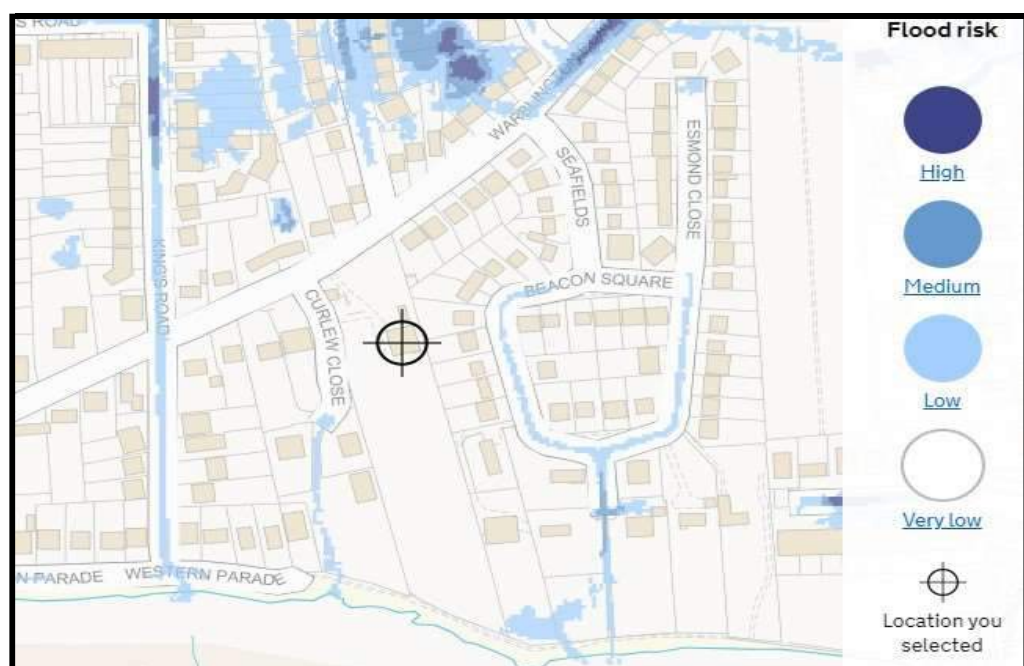


Figure 4: Surface Water Flood Risk Map (Source: EA)

- 1.9 In addition to surface water flood risk, it is also important to consider the risk of flooding from rivers and the sea. In this regard, EA mapping has further been interrogated to determine the risk posed from this type of flooding (see Figure 5).

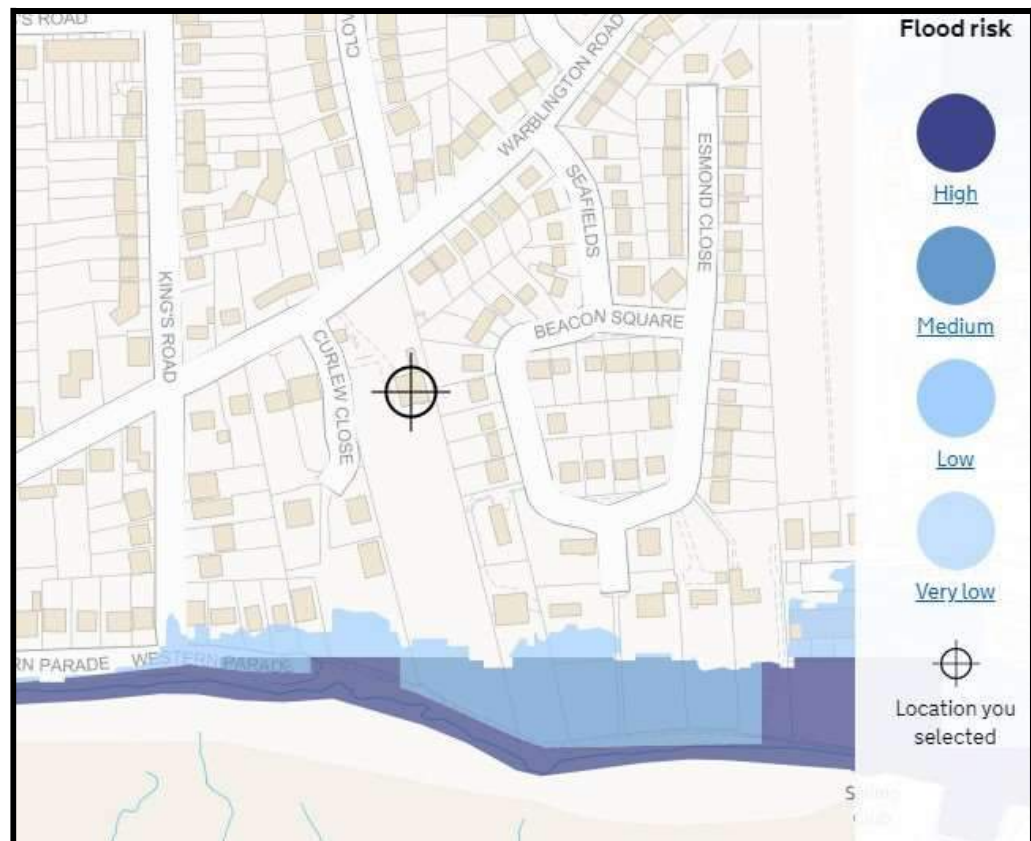


Figure 5: Rivers/Sea Flood Risk Map (Source: EA)

- 1.10 Figure 5 demonstrates that the southern extremity of the site has a low to medium risk of river or sea flooding.
- 1.11 The proposed development has been designed with due regard to this potential risk. As such, the proposed units have not been located within the area at risk of flooding.
- 1.12 Given the site's location, it is important to consider rising sea levels as a result of climate change. In this regard, the Product 4 Package and Emsworth to Littlehampton West Bank Coastal Modelling Report (2016) has been obtained from the EA. This reveals a maximum flooding level of 4.59m AOD which occurs during the 0.5% AEP storm event for the year 2115. A copy of both documents are provided at **Appendix 2**.

- 1.13 When designing the appropriate FFLs for the proposed units, it is appropriate to include an additional 'freeboard' allowance on top of the forecasted flood level to account for residual uncertainty. The freeboard level has been calculated using the Environment Agency (EA) document '*Accounting for residual uncertainty: updating the freeboard guide*' (February 2017), and the data received from the EA, including the Product 4 package and the Emsworth to Littlehampton West Bank Coastal Modelling (2016).
- 1.14 The method of determining the level of residual uncertainty involves assigning confidence scores to different aspects of the evidence and combining these estimates into an overall confidence rating. The three stages of this assessment are as follows:
- Stage 1 - Identify and record all primary sources of uncertainty
 - Stage 2 - Estimate the magnitude of the residual uncertainties
 - Stage 3 - Determine the appropriate response
- 1.15 The results of Stages 1 and 2 are set out in Table 1, which accords with Tables 3.1 and 3.2 of the EA freeboard guidance.

Table 1: Scale of Residual Uncertainty

Consideration	Applicability	Score	Reasons
How appropriate is the flood risk analysis?	Yes	1	There have been no changes in the intervening period that would affect the accuracy of the model.
How well is the flood plain modelled?	Yes	1	LIDAR coverage of the floodplain was used, with the majority of the area covered with 1m data.
How well has the potential for defence failure been modelled?	N/A	-	No flood defences at the site.
What is the confidence in the hydrology?	N/A	-	Tidally dominated site.
How good is the coastal/estuarine/tidal boundary?	Yes	1	The Chichester Harbour Model is within the desired calibration accuracy of +/- 150mm. The water level rise for climate change was based on the latest UKCP09 sea-level change guidance.
How have the fluvial threats been represented?	N/A	-	Tidally dominated site.
How have coastal threats been represented?	Yes	1	2D model. Wave overtopping based on EurOtop methods. Up to date Bathymetry data used in assessment. Eight annual likelihoods modelled (20, 5, 3.33, 1.33, 1, 0.5, 0.2, and 0.1% AEP events)
How has surface water run-off been represented?	N/A	-	The site is not at risk of surface water flooding.
How have groundwater hazards been represented?	N/A	-	No history of groundwater flooding.
What is the strength of evidence?	Yes	1	The wave model was calibrated against ten storm events. Further calibration took place using four more recent storm events. Calibration accuracy +/-150mm.

1.16 To calculate the confidence rating, the two lowest scores in Table 1 are applied to Table 3.3 within the EA freeboard guidance to determine the confidence rating. In this instance, the two lowest scores are '1' and in this regard the confidence rating has been calculated as '5 star', which indicates that the flood modelling is highly likely to be locally reliable. A '5 star' confidence rating corresponds to a minimum freeboard depth of 300mm.

1.17 In this regard, it will be ensured that the proposed dwellings will have a minimum FFL of 4.89m AOD (i.e. 2115 flood level of 4.59m AOD + 300mm freeboard allowance). It will also be ensured that the level of the road will be set at a minimum level of 4.89m AOD. To ensure safe access for all vehicles, including emergency vehicles. It is suggested that this is controlled by a suitably worded planning condition.

Ground Investigation Data

- 1.18 Soakaway testing, in accordance with BRE365, has previously been undertaken at the site. The testing observed that no infiltration was possible and concluded that the soils are not likely to perform to a satisfactory standard. The full testing results are attached at **Appendix 3**.
- 1.19 In addition, British Geological Survey mapping indicates that there are superficial deposits of sand, silt and clay (fluvial in origin), and the bedrock at the site is chalk. The BGS mapping extract is provided at **Appendix 4**.

2.0 FLOOD RISK ASSESSMENT

Flood Risk Assessment Methodology and Objectives

- 2.1 It is recognised that developments that are designed without regard to flood risk may endanger lives, damage property, cause disruption to the wider community, damage the environment, be difficult to insure and require additional expense on remedial works. Current guidance on development and flood risk identifies several key aims for a development to ensure that it is sustainable in flood risk terms. These aims are as follows:
- the development should not be exposed to flood risk such that the health, safety and welfare of the users of the development, or the population elsewhere, are threatened;
 - the development should not be at a significant risk of flooding and should not be susceptible to damage due to flooding;
 - safe access to and from the development should be possible during flood events;
 - the development should not increase flood risk elsewhere;
 - the development should not prevent safe maintenance of watercourses or maintenance and operation of flood defences;
 - the development should not be associated with an onerous or difficult operation and maintenance regime to manage flood risk. The responsibility for any operation and maintenance required should be clearly defined;
 - future users of the development should be made aware of any flood risk

issues relating to the development;

- the development design should be such that future users will not have difficulty obtaining insurance or mortgage finance, or in selling all or part of the development, as a result of flood risk issues;
- the development should not lead to degradation of the environment, and;
- the development should meet all of the above criteria for its entire lifetime, including consideration of the potential effects of climate change.

- 2.2 This Assessment has been prepared with due consideration of these sustainability aims, to inform the proposed scheme. A development lifetime of 100 years has been assumed in line with PPG guidance on residential development life cycles.

Existing Greenfield Run-off Rates

- 2.3 The existing greenfield run-off rates have been obtained from within the MicroDrainage computer software (see **Appendix 5**), which has identified the following Qbar figures:

- Qbar - 3.4l/s
- Q1 - 2.9l/s
- Q30 - 7.7l/s
- Q100 - 10.9l/s

3.0 DRAINAGE STRATEGY

Disposal of Surface and Foul Water

- 3.1 The existing dwelling utilises the Southern Water surface water and foul water sewers located in the vicinity of the application site. The location of the existing foul and surface water sewers is provided on the plan attached at **Appendix 6**.
- 3.2 The surface water sewer borders the western boundary of the application site and outfalls into Chichester Harbour. There are two foul water sewers in the vicinity of the site, one bisects the application site on a broadly west/east alignment and the other is present within Warblington Road.

Surface Water

- 3.3 In accordance with SuDS principles, the proposal will initially seek to provide soakaway drainage to serve each plot. However, based on the results of the infiltration testing previously undertaken at the site, the sub-strata is not suitable for soakaway drainage (see Appendix 2).
- 3.4 In this regard, an alternative option has been considered which will provide three storage tanks within the site to store the surface water from each property and carriageway (all driveways to be permeable paving). Each tank will discharge into the nearby surface water sewer. The discharge rate will be controlled via a hydro-brake, and will be limited to 10.9 l/s, as per the calculated Qbar figure (see paragraph 2.3). The total storage capacity has been calculated using the MicroDrainage software. The indicative surface water drainage strategy is provided at **Appendix 7**, and the MicroDrainage analysis is at **Appendix 8**, which shows no risk of flooding during the modelled storm events.
- 3.5 As stated previously, it will be ensured that the Finished Floor Level (FFL) of the proposed dwellings, and level of the access road, will be a minimum of 4.89m AOD, i.e. above the 2115 estimated flood level of 4.59m AOD and to also include a freeboard allowance of 300mm. Flood resistant construction can prevent entry of water or minimise the amount that may enter a building where there is short duration flooding outside with water depths of 0.6 metres or less. This form of construction should be used with caution and accompanied by resilience measures, as effective flood exclusion may depend on occupiers ensuring some elements, such as barriers to doorways, are put in place and maintained in a good state. Examples of flood resistance measures include flood doors and windows; and non-return valves fitted on drain/pipes.
- 3.6 In accordance with Planning Practice Guidance, it is suggested that the buildings are designed and constructed with flood resilience measures in mind, thus reducing reduce the impact of flood water entering the building to ensure no permanent damage is caused, structural integrity is maintained and drying and cleaning is easier. Furthermore, flood resistant construction can prevent entry of water or minimise the amount that may enter a building where there is short duration flooding outside with water depths of 0.6 metres or less. This form of construction should be used with caution and accompanied by resilience

measures, as effective flood exclusion may depend on occupiers ensuring some elements, such as barriers to doorways, are put in place and maintained in a good state.

Foul Water

- 3.7 With regards to the foul water strategy, it is proposed that plots 6-9 will connect to the existing foul water sewer within Warblington Road, and the remainder (plots 1-5) will connect to the foul water sewer that bisects the site. The foul strategy is shown on the plan at Appendix 7.

Maintenance and Management

- 3.8 The continued management and maintenance of the drainage system will be essential in ensuring the continued protection of the development from the surface water flooding.
- 3.9 Visual inspection of the drainage network and outfalls should be undertaken on a regular basis to identify any siltation/blockage/disrepair of the drainage network and attenuation facilities.
- 3.10 A surface water management system therefore needs to be drawn up as part of any planning consent that is issued for the site.

4.0 SUMMARY AND CONCLUSIONS

- 4.1 Bellamy Roberts LLP has been retained by Mr. Charles Glanville to undertake a Flood Risk Assessment (FRA) in support of a planning application for a residential development of 9 units on land south of Warblington Road, Emsworth.
- 4.2 Environment Agency mapping has been reviewed and it has been determined that the site lies wholly within Flood Zone 1, and no risk of flooding from sources such as surface water and rivers.
- 4.3 Due to the topography of the application site, part of the proposed built development falls within the estimated future tidal flood risk area for 2115. In this regard, floor levels of the southernmost properties, and the access road, will be designed to be above the 2115 estimated flood level of 4.59m AOD and will also

include a freeboard allowance of 300mm.

- 4.4 Surface water storage tanks will be provided to serve the proposed dwellings and access road. Each tank will discharge into the nearby surface water sewer and the discharge rate will be controlled via a hydro-brake, limited to the calculated Qbar figure of 10.9l/s.
- 4.5 With regards to the foul water strategy, it is proposed that the northernmost plots will connect to the existing foul water sewer within Warblington Road, and the remaining plots will connect to the foul water sewer that bisects the site.

APPENDICES

APPENDIX 1

Havant Borough Council
Development Control
Civic Offices
Civic Centre Road
Havant
Hampshire
PO9 2AX

Our ref: HA/2020/122296/01
Your ref: APP/20/00376
Date: 23 June 2020

Dear Sir or Madam,

RETAIN THE EXISTING 'FOWLEY COTTAGE' DWELLING AND TENNIS COURT AND CONSTRUCT NINE NEW DWELLINGS AND 1NO. GARDEN STUDIO (REVISED).

FOWLEY COTTAGE, 46 WARBLINGTON ROAD, EMSWORTH, PO10 7HH.

Thank you for consulting the Environment Agency on the above application.

We have reviewed the information as submitted and set out our position and comments below.

Environment Agency position

In the absence of an acceptable Flood Risk Assessment (FRA) we **object** to this application and recommend that planning permission is refused.

Reasons

The submitted FRA does not comply with the requirements for site-specific flood risk assessments, as set out in the Planning Practice Guidance to the National Planning Policy Framework (NPPF) for Flood Risk and Coastal Change. The FRA does not therefore adequately assess the flood risks posed by the development. In particular, the FRA fails to consider the flood risk mitigation measure of an additional appropriate freeboard to the design flood levels for 2115 of 4.5m AOD to achieve Finished Floor Levels (FFLs) that will make the development flood resilient for its lifetime. Consequently, the development proposes inadequate raised FFLs.

Paragraph 1.13 of the submitted FRA "*FLOOD_RISK_ASSESSMENT-1391668*" states "*In this regard, it will be ensured that the Finished Floor Level (FFL) of these properties will be a minimum of 4.5m AOD, and therefore above the 2115 estimated flood level*", but it does not specify what they will be and whether freeboard has been included.

Overcoming our objection

Environment Agency
Oving Road, Chichester, West Sussex, PO20 2AG
Customer services line: 03708 506 506
www.gov.uk/environment-agency

Cont/d..

To overcome our objection, the Applicant should submit a revised FRA which addresses the points highlighted above. If this cannot be achieved, we are likely to maintain our objection.

Please re-consult us on any revised FRA submitted and we will respond within 21 days of receiving it.

Advice to the Local Planning Authority

If you are minded to approve the application contrary to our objection, please contact us to explain why material considerations outweigh our objection. This will allow us to make further representations. Should our objection be removed, it is likely we will recommend the inclusion of condition(s) on any subsequent approval.

In accordance with the Planning Practice Guidance (Ref. ID: 20150415), please notify us by email within 2 weeks of a decision being made or application withdrawn. Please provide us with a URL of the decision notice, or an electronic copy of the decision notice or outcome.

Advice to the Applicant

Finished Floor Levels (FFLs)

For the built development proposal to be acceptable, an appropriate freeboard needs to be added to the to the design flood level for 2115 of 4.5mAOD. The following government website will assist the Applicant in determining the adequate freeboard for the development (this includes coastal, estuarine and tidal):

<https://www.gov.uk/government/publications/accounting-for-residual-uncertainty-an-update-to-the-fluvial-freeboard-guide>

Flood Risk Activity Permit

The Environmental Permitting (England and Wales) Regulations 2016 requires a permit (called a 'Flood Risk Activity Permit') to be obtained for any activities which will take place:

- on or within 8 metres of a main river (16 metres if tidal);
- on or within 8 metres of a flood defence structure or culvert (16 metres if tidal);
- on or within 16 metres of a sea defence;
- involving quarrying or excavation within 16 metres of any main river, flood defence (including a remote defence) or culvert; and/or
- in a floodplain more than 8 metres from the river bank, culvert or flood defence structure (16 metres if it's a tidal main river) and you do not already have planning permission.

For further guidance, please visit <https://www.gov.uk/guidance/flood-risk-activities-environmental-permits> or contact our National Customer Contact Centre on **03708 506 506** (Monday to Friday 8am to 6pm) or by emailing enquiries@environment-agency.gov.uk.

The Applicant should note that a permit is separate to and in addition to any planning permission granted. The granting of planning permission does not necessarily lead to

the granting of a permit.

Please do not hesitate to contact me using the contact details shown below should you have any queries regarding the above information.

Yours faithfully,

Mrs Sophie Brown
Sustainable Places Advisor

Direct dial: 02030 257250

Direct e-mail: sophie.brown@environment-agency.gov.uk

APPENDIX 2

Matt Twinberrow
Bellamy Roberts
Clover House
Western Lane
Odiham
Hampshire
RG29 1TU

Our ref: SSD177914
Date: 07/08/2020

Dear Matt Twinberrow,

Enquiry Regarding Product 4 for Flood Risk Assessment for Fowley Cottage, Warblington Road, Emsworth, PO10 7HH.

Thank you for your enquiry which was received on 14 July 2020.

We respond to requests under the Freedom of Information Act 2000 and Environmental Information Regulations 2004. The information is attached.

The information on Flood Zones in the area relating to this address is as follows:

The site is in an area located within Flood Zone 1, 2 & 3 as shown on our Flood Map for Planning (Rivers and Sea).

Note - This information relates to the area that the above named property is in and is not specific to the property itself as it is influenced by factors such as the height of door steps, air bricks or the height of surrounding walls. We do not have access to this information and is not currently used in our flood modelling.

Flood Zone definitions can be found at www.gov.uk/guidance/flood-risk-and-coastal-change#Table-1-Flood-Zones

Flood Defences

There are no formal raised flood defences in the vicinity of the site.

Model Information

The model used was the Chichester Coastal Modelling (2D) which was completed by JBA Consulting in 2016.

Flood History

We hold no record of previous flooding events affecting this site.

Please note our records are not comprehensive and may not include all events. I recommend contacting the Lead Local Flood Authority, **Hampshire County Council** or the Local Authority, **Havant District Council** for a more comprehensive flood history check.

[FRA advisory text](#)

Name	Product 4
Description	Detailed Flood Risk Assessment Map for Fowley Cottage, Warblington Road, Emsworth, PO10 7HH.
Licence	Open Government Licence
Information Warnings	The flood risk data provided is based on existing EA hydraulic models for existing 0.5% annual probability events with an allowance for climate change. Please note the climate change allowances provided are not up to date. These were updated on 17 December 2019. You should refer to ' Flood risk assessments: climate change allowances ' for the most up to date allowances. You will need to undertake further assessment of future flood risk using different allowances to ensure your assessment of future flood risk is based on best available evidence.
Information Warning - OS background mapping	<i>The mapping of features provided as a background in this product is © Ordnance Survey. It is provided to give context to this product. The Open Government Licence does not apply to this background mapping. You are granted a non-exclusive, royalty free, revocable licence solely to view the Licensed Data for non-commercial purposes for the period during which the Environment Agency makes it available. You are not permitted to copy, sub-license, distribute, sell or otherwise make available the Licensed Data to third parties in any form. Third party rights to enforce the terms of this licence shall be reserved to OS.</i>
Attribution	Contains Environment Agency information © Environment Agency and/or database rights. Contains Ordnance Survey data © Crown copyright 2020 Ordnance Survey 100024198.

Data Available Online

Many of our flood datasets are available online:

- Flood Map For Planning ([Flood Zone 2](#), [Flood Zone 3](#), [Flood Storage Areas](#), [Flood Defences](#), [Areas Benefiting from Defences](#))
- [Risk of Flooding from Rivers and Sea](#)
- [Historic Flood Map](#)
- [Current Flood Warnings](#)

Please get in touch if you have any further queries or contact us within two months if you'd like us to review the information we have sent.

Yours sincerely,

Tom Lamboo

FCRM Officer, Partnership & Strategic Overview West Sussex, Solent and South Downs

Environment Agency, Guildbourne House, Chatsworth Road, Worthing, West Sussex, BN11 1LD

Flood Map for Planning (Rivers and Sea). Centred PO10 7HH. Created 03/08/2020.



1: 10,000

0 Metres 250



Flood Map for Planning (Rivers & Sea)

- Defences
- Flood Storage Areas
- Areas benefiting from flood defences
- Flood Zone 3
- Flood Zone 2

Flood Map Areas (assuming no defences)

Flood Zone 3 shows the area that could be affected by flooding:

- from the sea with a 1 in 200 or greater chance of happening each year
- or from a river with a 1 in 100 or greater chance of happening each year.

Flood Zone 2 shows the extent of an extreme flood from rivers or the sea with up to a 1 in 1000 chance of occurring each year.

Use of Environment Agency Information for Flood Risk Assessments

Important

The Environment Agency are keen to work with partners to enable development which is resilient to flooding for its lifetime and provides wider benefits to communities. If you have requested this information to help inform a development proposal, then we recommend engaging with us as early as possible by using the pre-application form available from our website:

<https://www.gov.uk/government/publications/pre-planning-application-enquiry-form-preliminary-opinion>

We recognise the value of early engagement in development planning decisions. This allows complex issues to be discussed, innovative solutions to be developed that both enables new development and protects existing communities. Such engagement can often avoid delays in the planning process following planning application submission, by reaching agreements up-front. We offer a charged pre-application advice service for applicants who wish to discuss a development proposal.

We can also provide a preliminary opinion for free which will identify environmental constraints related to our responsibilities including flooding, waste, land contamination, water quality, biodiversity, navigation, pollution, water resources, foul drainage or Environmental Impact Assessment.

In preparing your planning application submission, you should refer to the Environment Agency's Flood Risk Standing Advice and the Planning Practice Guidance for information about what flood risk assessment is needed for new development in the different Flood Zones. This information can be accessed via:

<https://www.gov.uk/flood-risk-assessment-standing-advice>
<http://planningguidance.planningportal.gov.uk/>

You should also consult the Strategic Flood Risk Assessment or other relevant materials produced by your local planning authority.

You should note that:

1. Information supplied by the Environment Agency may be used to assist in producing a Flood Risk Assessment (FRA) where one is required, but does not constitute such an assessment on its own.
2. This information covers flood risk from main rivers and the sea, and you will need to consider other potential sources of flooding, such as groundwater or surface water runoff. Information produced by the local planning authority referred to above may assist here.
3. Where a planning application requires an FRA and this is not submitted or is deficient, the Environment Agency may raise an objection.



Environment
Agency

N



Legend

 Site_Nodes

 Site_Boundary

Annual Exceedance Probability
(AEP) The probability of a flood
of a particular magnitude,
or greater occurring in any given
year.

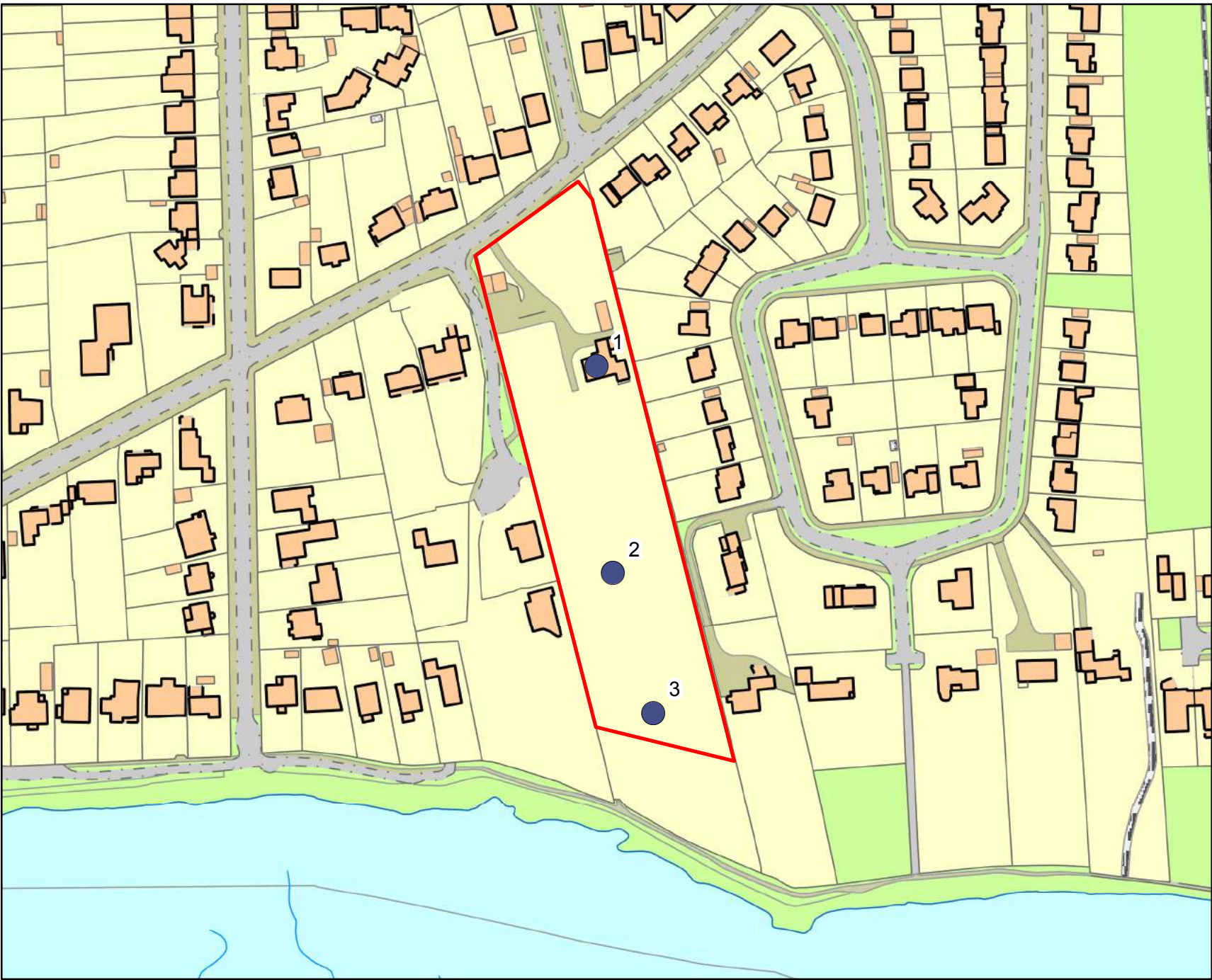
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0

0.045

0.09

Kilometers



Product 4 Flood Risk Data Requested by: Bellamy Roberts

Site: Fowley Cottage, Warblington Road, Emsworth, PO10 7HH

Table 1: Water Levels: Tidal Undefended

Node Ref	NGR		Modelled Flood Levels in Metres AOD			
	Eastings	Northings	Undefended Annual Exceedance Probability			
			0.5%	0.5% (2070)	0.5% (2115)	0.1%
1	474404	105497	-	-	-	-
2	474410	105420	-	-	4.57	-
3	474425	105368	-	3.96	4.57	-

Table 2: Water Levels: Tidal Defended

Node Ref	NGR		Modelled Flood Levels in Metres AOD			
	Eastings	Northings	Defended Annual Exceedance Probability			
			0.5%	0.5% (2070)	0.5% (2115)	0.1%
1	474404	105497	-	-	-	-
2	474410	105420	-	-	4.59	-
3	474425	105368	-	3.95	4.59	-

Table 3: Water Depths: Tidal Undefended

Node Ref	NGR		Modelled Flood Depths in Metres			
	Eastings	Northings	Undefended Annual Exceedance Probability			
			0.5%	0.5% (2070)	0.5% (2115)	0.1%
1	474404	105497	-	-	-	-
2	474410	105420	-	-	0.01	-
3	474425	105368	-	0.13	0.74	-

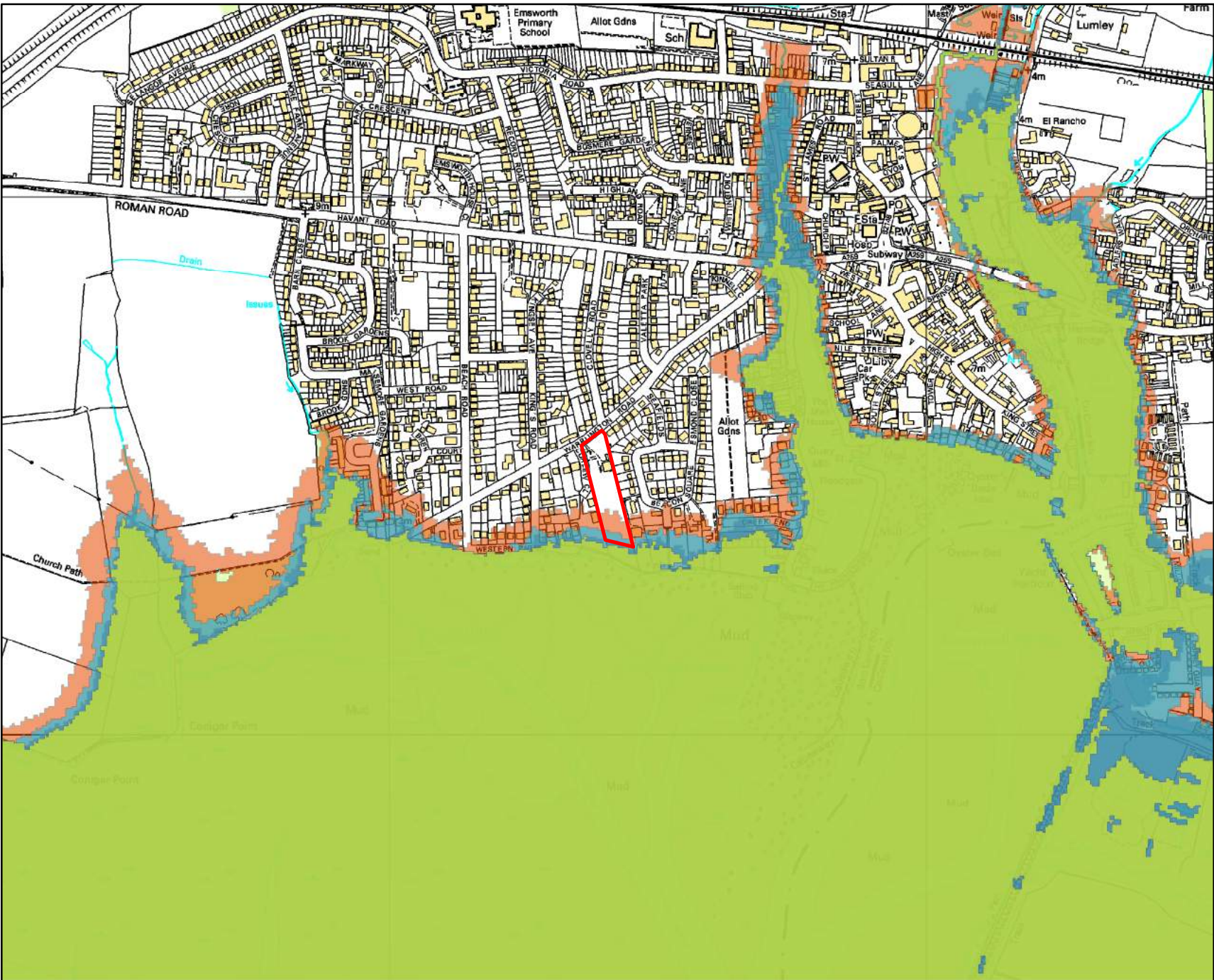
Table 4: Water Depths: Tidal Defended

Node Ref	NGR		Modelled Flood Depths in Metres			
	Eastings	Northings	Defended Annual Exceedance Probability			
			0.5%	0.5% (2070)	0.5% (2115)	0.1%
1	474404	105497	-	-	-	-
2	474410	105420	-	-	0.01	-
3	474425	105368	-	0.17	0.81	-

All levels taken from: Chichester (2D)

Produced on: 07/08/2020

There is no additional information or health warnings for these levels/depths or the model from which they have been produced.

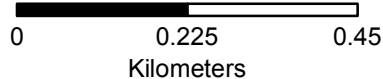


Legend

- Site_Boundary
- 0.5% AEP (Defended Tidal)
- 0.5% AEP (2070) NPPF (Defended Tidal)
- 0.5% AEP (2115) NPPF (Defended Tidal)
- 0.1% AEP (Defended Tidal)

Annual Exceedance Probability (AEP) The probability of a flood of a particular magnitude, or greater occurring in any given year.

Scale: 1:10,000

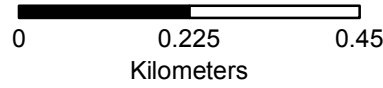




- Legend**
- Site_Boundary
 - 0.5% AEP (Un defended Tidal)
 - 0.5% AEP (2070) NPPF (Un defended Tidal)
 - 0.5% AEP (2115) NPPF (Un defended Tidal)
 - 0.1% AEP (Un defended Tidal)

Annual Exceedance Probability (AEP) The probability of a flood of a particular magnitude, or greater occurring in any given year.

Scale: 1:10,000



Risk of flooding from Surface Water. Centred PO10 7HH. Created 03/08/2020.



1: 10,000

0 Metres 250



Likelihood of flooding from Surface Water

- High ($\geq 3.3\%$)
- Medium (3.3% - 1%)
- Low (1% - 0.1%)
- Very Low

Likelihood of flooding from Surface Water

- High:** Greater than or equal to 3.3% (1 in 30) chance in any given year
- Medium:** Less than 3.3% (1 in 30) but greater than or equal to 1% (1 in 100) chance in any given year
- Low:** Less than 1% (1 in 100) but greater than or equal to 0.1% (1 in 1,000) chance in any given year
- Very Low:** Less than 0.1% (1 in 1,000) chance in any given year

This information is shown on the Risk of Flooding from Surface Water map on GOV.UK.

Solent & South Downs Area

Pre-application Advice Note

September 2019

This document sets out the environmental issues we will consider when providing our planning application consultation advice to Local Councils. It can be used by applicants, developers and consultants at the pre-planning stage.

Fluvial Flood Risk

Development must be safe and should not increase the risk of flooding.

You can view a site's flood zone on the Flood Map for Planning on our website: <https://flood-map-for-planning.service.gov.uk>

If your proposed development is located within flood zone 2 or 3 you should consult the Flood Risk and Coastal Change pages of the National Planning Policy Guidance (NPPG)
<http://planningguidance.communities.gov.uk/blog/guidance/flood-risk-and-coastal-change/>

Here you can determine whether the flood risk vulnerability of your proposed development and the flood zone are compatible. You can also establish if there are flood risk sequential test and exception test requirements for your proposed development. In the first instance we recommend the developer/applicant liaises with the Local Planning Authority (LPA) to undertake the Sequential Test in accordance with the National Planning Policy Framework (NPPF).

If your proposed development is located within flood zone 2 or 3 and its vulnerability and flood zone are considered acceptable under the NPPG then a site specific Flood Risk Assessment (FRA) is required to support any subsequent planning application. This is required by paragraph 163 of the NPPF: https://www.gov.uk/government/uploads/system/uploads/attachment_data/file/6077/2116950.pdf

Guidance on the content of a site specific FRA can be found in the NPPG and online: <https://www.gov.uk/guidance/flood-risk-assessment-for-planning-applications>

More detailed flood risk modelling data is available to help you produce a FRA please contact our Customers and Engagement team at SSDenquiries@environment-agency.gov.uk.

Climate Change Allowances

On 19 February 2016, we published new guidance for planners and developers on how to use climate change allowances in a site-specific FRA: <https://www.gov.uk/guidance/flood-risk-assessments-climatechange-allowances>

Groundwater Quality

Development must not cause pollution to the water environment.

Source Protection Zones

We have defined Source Protection Zones (SPZs) for 2000 groundwater sources such as wells, boreholes and springs used across the country for public drinking water supply. These zones are more sensitive to contamination from activities that might cause pollution in the area. The closer the activity, the greater the risk.



SPZ1s are the areas designated as most at risk from contamination and development activities and in these areas we may consider it inappropriate for development to discharge foul or surface water into the ground.

To see if your proposed development is located within a Source Protection Zone, please use our online map: <https://magic.defra.gov.uk/>

Contaminated Land

The NPPF takes a precautionary approach to land contamination. Before the principle of development can be determined, land contamination should be investigated to see whether it could preclude certain development due to environmental risk or cost of remediation. Where contamination is known or suspected, a desk study, site investigation, remediation and other works may be required to enable safe development.

Pollution

If the proposed development use has the potential to pollute ground or surface water receptors then an assessment to establish whether the risk of pollution is acceptable or can be mitigated will be required within any planning application.

Foul Drainage

When drawing up wastewater treatment proposals for any development, the first presumption is to provide a system of foul drainage discharging into a public sewer to be treated at a public sewage treatment works (those provided and operated by the water and sewerage companies). This should be done in consultation with the sewerage company of the area prior to the submission of a formal planning application.

If connection to a public sewage treatment plant is not feasible, a package sewage treatment plant may be considered. If you would like further advice please call 03708 506 506.

Cemeteries

The development of new cemeteries in areas where groundwater vulnerability is high should be avoided, except where the thickness and nature of the unsaturated zone, or the impermeable formations beneath the site, protect groundwater; or where the long-term risk is mitigated by appropriate engineering methods.

Main Rivers

Ecology

In accordance with the National Planning Policy Framework (NPPF), any development proposal should avoid significant harm to biodiversity and seek to protect and enhance it. Opportunities to incorporate biodiversity in and around the development will be encouraged.

Your scheme should be designed with a naturalised buffer zone of at least 8 metres from the main river to protect and enhance the conservation value of the watercourse and ensure access for flood defence maintenance.

This buffer zone should be managed for the benefit of biodiversity for example by the planting of locally appropriate, UK native species. The buffer zone should be undisturbed by development with no fencing, footpaths or other structures. This buffer zone will help provide more space for flood waters, provide improved habitat for local biodiversity and allows access for any maintenance requirements.

To identify any Main Rivers in proximity to your proposed development please see our Main Rivers Consultation Map: <http://apps.environment-agency.gov.uk/wiyby/151293.aspx>

customer service line
03708 506 506

www.gov.uk/environment-agency

incident hotline
0800 80 70 60

floodline
0345 988 1188

Culverting

The Environment Agency is likely to oppose culverting as it is damaging to the ecological integrity of the river channel and its corridor and acts as a barrier to the movement of wildlife, including fish and may also increase flood risk. If the proposal will impact an existing culvert the Environment Agency may oppose planning consent for development either over, or within 8 metres of an existing culvert. Wherever possible, existing culverts should be removed and the river channel and bankside habitat reinstated to restore the ecological continuity of the river channel and its corridor.

Water Framework Directive (WFD)

Any marine works below MHWS require an assessment of possible impacts on Water Framework Directive (WFD). The assessment should include all elements of the works that fall within, or have the potential to affect, a WFD water body and any of the protected areas therein (including Bathing Waters and Shellfish Waters).

The WFD assessment should follow the 'Clearing the Waters for All' guidance available at <https://www.gov.uk/guidance/water-framework-directive-assessment-estuarine-and-coastal-waters>

Where appropriate, a WFD Assessment should assess any potential impacts and demonstrate that the required enhancements will be delivered. In some cases the requirements of a WFD assessment can be incorporated into an Environmental Impact Assessment (EIA). Any development that has the potential to cause deterioration in classification under WFD or that precludes the recommended actions from being delivered in the future is likely to be considered unacceptable to us.

Permits & Consents

Environmental Permitting Regulations

To see if your proposed development requires an Environmental Permit under the Environmental Permitting Regulations please refer to our website: <https://www.gov.uk/guidance/check-if-you-need-an-environmental-permit>

From 6 April 2016 an Environmental Permit is required for any proposed works or structures, in, under, over or within 8 metres of the top of the bank of designated Main River, and within 16 metres of a tidal defence.

Ordinary Watercourse Consent

The prior written consent of the relevant Lead Local Flood Authority is required for the erection of any flow control structures, culverting or diversion of ordinary watercourses, including streams, land drains and ditches.

Marine Licence

A marine licence may be required for any activities at the mean high water spring tide up to the territorial limit. This also includes the waters of every estuary, river or channel where the tide flows at mean high water spring tide.

Any development must demonstrate how adverse impacts on migratory fish, bathing waters, shellfish waters, designated sites, protected and priority species and habitats will be avoided, minimised, mitigated and if necessary compensated for. Works within or affecting a Water Framework Directive (WFD) waterbody will need to demonstrate that compliance with WFD objectives will be achieved. 'Clearing the Waters for All' provides guidance on how the impacts on WFD should be addressed, and should be used when preparing an assessment, including the screening and scoping of activities. <https://www.gov.uk/guidance/water-framework-directive-assessment-estuarine-and-coastal-waters>

Further pre-application options

The information provided above details generic information which may or may not be applicable to your development. We are able to provide more detailed and bespoke advice and answer technical questions for a charged fee of £100 per person per hour +VAT.

If you are interested in finding out more about this service, please email:

planningssd@environment-agency.gov.uk

We can explain this service and provide you with a bespoke quote for further pre-application advice that you may require please see .gov - <https://www.gov.uk/government/publications/pre-planning-application-enquiry-form-preliminary-opinion>

Please note

Please note that the view expressed in this letter by the Environment Agency is in response to the enquiry only and does not represent our final view in relation to any future planning application made in relation to this site.

We reserve the right to change our position in relation to any such application.

As part of this preliminary response we have not technically reviewed any documents. This opinion is based on the information submitted and current planning policy and guidance.

If you have any questions please contact the Solent & South Downs Sustainable Places team:

planningssd@environment-agency.gov.uk

To make a request for data

Please submit your request for data to ssdenquiries@environment-agency.gov.uk. You should get the information within 20 working days. We will tell you when to expect the information if we need more time.

There are many datasets available online at www.data.gov.uk including flood maps, historic landfill, waste exemptions, consented discharges to controlled waters, and much more.

customer service line
03708 506 506

www.gov.uk/environment-agency

incident hotline
0800 80 70 60

floodline
0345 988 1188

Emsworth to Littlehampton West Bank Coastal Modelling

Final report

April 2016

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Revision History

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Contract

This report describes work commissioned by the Environment Agency. The Environment Agency's representative for the contract was Emma Dauben. The JBA Consulting Project Manager for the contract was Matthew Hird and Robert Quirke, Emma Rendle, Ian Gaskell, Kathryn Williams and Thomas van Lanschot carried out the work.

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Senior Analyst

Purpose

This document has been prepared as a final report for the Environment Agency. 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 the Environment Agency.

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JBA is aiming to reduce its per capita carbon emissions.

Executive Summary

The study was commissioned by the Environment Agency, to investigate the risk of coastal flooding along several key sections of the West Sussex coastline. The study area includes the communities of Littlehampton, Bognor Regis, Pagham, Selsey, Wittering and Bosham and will aim to detail the vulnerability of each of these communities to flooding from the sea based on a range of possible scenarios.

Much of the coastline within the study area is defended to a high standard of protection and previous studies have shown that still water flooding is unlikely, even during extreme surge events. As such, most flooding results through wave action or defence failure. To quantify the flood risk detailed hydraulic models were constructed to enable the production of coastal flood risk extents.

The study used comprehensive extreme sea-level and wave information, improved quality topographic data and advanced inundation modelling techniques to produce revised tidal flood risk information, including Flood Zones 2 and 3. The revised Flood Zones were used to examine the potential impacts of climate change on flood risk, investigating flooding on a community based scale. Climate change simulations were completed using both NPPF and UKCP09 guidance, both to 2070 and 2115.

In summary, the study objectives included:

- Simulation of a range of defended scenarios including the effects of wave overtopping for the 20%, 5%, 3.33%, 1.33%, 1%, 0.5%, 0.5% (2070 UKCP09), 0.5% (2115 UKCP09), 0.5% (2070 NPPF), 0.5% (2115 NPPF), 0.2% and 0.1% AEP events;
- Assess the sensitivity of wave overtopping estimation due to different profile schematisations, beach states and percentile overtopping rates;
- Analyse the projected impact of rising sea levels associated with climate change on flood extents and depths using both NPPF and UKCP09 guidance;
- Provide an indicative standard of protection estimate for the coastal defences based on still water levels and the calculated overtopping volumes;
- Update the current Environment Agency Flood Zones; and
- Improvement the current Environment Agency Flood Warning Procedures for the above communities.

The study objectives were met through the construction of new coastal models, including:

- a wave transformation model to transform the offshore waves into the nearshore,
- wave overtopping modelling to calculate wave overtopping discharges using data from the wave transformation model, and
- the adaption and development of two flood inundation models to map the flood risk.

New flood map components

The new tidal Flood Zones derived for this study are similar to the previous Flood Zones, albeit slightly more extensive. The proposed Flood Warning thresholds are generally lower than the current Flood Alert and Warning thresholds, due to the method of calculation using first property to flood rather than defence crest levels. In the Arun model, the proposed Flood Warning Area A includes significant inundation at Littlehampton West Bank from the River Arun as a result of still water flooding as well as coastal flooding at Bognor Regis and Pagham Harbour. Flood Warning Area B (the area between the 0.5% AEP defended flood extent and Flood Zone 2 extent) includes additional flooding at Climping, Elmer, Bognor Regis, Selsey and Bracklesham. Likewise in the Chichester Harbour model, the new Flood Zones are similar to the existing Flood Zones, with slight increases in extent for most regions. The most noticeable increases are at Birdham and on Thorney Island.

Impact of Climate Change

In addition to the present day flood simulations used to produce the revised Flood Zones for the study area, climate change simulations for the 0.5% AEP event were also modelled. These simulations indicate the expected increase in flood risk up to the year 2115 based on the latest UKCP09 and NPPF guidance on sea-level rise estimates. These simulations, which were run for the 0.5% AEP event, show that with the increases to still water level, the region and its current defence structures are vulnerable to flooding due to climate change. The defence network along

the coast becomes heavily inundated as a result of increased wave overtopping of the current defences. In the Arun model, during the 0.5% AEP defended simulation, 968 properties were flooded, compared with 5,115 and 7,090 to 2115 based on UKCP09 and NPPF guidance respectively. In the Chichester Harbour model 1,581 properties were flooded during the 0.5% AEP event, compared with 9,190 and 16,663 to 2115 based on UKCP09 and NPPF guidance respectively.

The climate change wave overtopping scenario suggests that the effects of climate change could make still water flooding and wave overtopping an increasing hazard. The coastal frontage at risk of increased flooding includes Bognor Regis, Selsey, Bracklesham, East Wittering, West Wittering, West Itchenor, Bosham, Emsworth, Thorney Island and Hayling Island. The impacts of climate change on the West Sussex coastline are going to be an ever increasing hazard to both residential and commercial property and infrastructure.

Impact on critical infrastructure

As part of the study, the first critical infrastructure to flood for both Arun and Chichester models were identified. The first to flood in the Arun to East Head model domain was a sewage pumping station in Littlehampton, with a water level threshold of 3.84m relating to the water level at Littlehampton gauge. In the Chichester Harbour model, the first to flood was also an electricity sub-station, on Hayling Island with postcode PO11 0NQ. The water level threshold for this sub-station was 3.14m which related to the Wittering tidal curve.

Limitations and recommendations

The approaches taken in this study include the most appropriate 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 applied such that the data management and methods used in the study are appropriate, 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. These assumptions include decisions taken with regard to the application of the joint probability approach and the profiles used for the wave overtopping schematisations, for example. A full list of the assumptions and limitations is included in the final chapter.

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 there are a number of recommendations for future work and updates which could be undertaken to improve confidence in the modelling.

It is recommended that the joint probability approach is reviewed once new practices from the State of the Nation project come into use.

Following the winter storms of 2013/14 it is recommended that the CFB data are updated to account for the additional gauged data and the notable extremes that occurred.

The model should be updated to use the latest version of the TUFLOW software to allow incorporation of the operational control rules.

As many of the beaches are shingle beach defences it is recommended that an alternative method is used to calculate the wave overtopping, such as X-Beach-G, to take account of the dynamic response of the beach throughout the course of the event and compare these to the rates used within this study.

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.

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Abbreviations

1D	One dimensional
2D	Two dimensional
ABD.....	Areas Benefiting from Defences
AIMS	Asset Information Management System
BKB.....	Bracklesham wave buoy
CCO	Channel Coastal Observatory
CD	Chart Datum
CEFAS	Centre for Environment Fisheries and Aquaculture Science
CFB.....	Coastal Flood Boundary
CS3X.....	Met Office surge model
Defra	Department for the Environment, Food and Rural Affairs
DSM	Digital Surface Model
DTM	Digital Terrain Model
ESL	Extreme sea level
EurOtop.....	European Overtopping Manual
GIS.....	Geographic Information System
GMT	Greenwich Mean Time
GPD	Generalised Pareto Distribution
HAT	Highest Astronomical Tide
HGI.....	Hayling Island wave buoy
HR.....	Hazard Rating
JONSWAP	Joint North Sea Wave Project
LIDAR	Light Detecting and Ranging
mAOD	metres Above Ordnance Datum
NPPF	National Planning and Policy Framework
NRD	National Receptor Data

PoT	Peaks over Threshold
OS.....	Ordnance Survey
RFRA	Residual Flood Risk Area
RMSE.....	Root Mean Square Error
RST	Rustington wave buoy
S.....	South
SE	South East
SoP	Standard of Protection
SW	South West
SWAN	Simulating WAVes Nearshore
SWL	Still Water Level
TUFLOW.....	Two-dimensional Unsteady FLOW
UK	United Kingdom
UKCP09	UK Climate Projections 09
WWIII	Wave Watch 3

1 Introduction

1.1 Study objectives

The study was commissioned by the Environment Agency, to investigate the risk of coastal flooding along parts of the West Sussex coastline. The study area extended from Emsworth in the west to Littlehampton in the east, encompassing the communities of Emsworth, Bosham, Chichester, Wittering, Bracklesham, Selsey, Pagham, Bognor Regis and Climping. The study aimed to detail the vulnerability of each of these communities to flooding from the coastal processes through a range of possible defended, undefended and breach model scenarios.

The study had a number of key objectives. The first was to consider the flooding expected from a series of defended scenarios to simulate the present day defences for a range of extreme tidal events. Much of the coastline within the study area is defended with a mixture of shingle beaches and hard defences. The standard of protection (SoP) provided by these defences makes still water level (SWL) flooding unlikely. Therefore, the main flood risk stems from wave overtopping. It is essential that flooding from both the SWL and wave overtopping is better understood within the study area. The results from this modelling will be used by the Environment Agency, for a variety of applications including assessment of flood risk for new developments, flood warnings and supporting decision making on maintenance and defence schemes.

A further objective of the study was to update the current Environment Agency Flood Zones. To do so required the modelling of a number of undefended scenarios. Areas Benefiting from Defences (ABDs) were created using the outputs from both defended and undefended scenarios to ascertain the value of the defences.

In addition to present day scenarios, the study investigated the impact of predicted sea-level rise for both defended and undefended scenarios. The sea-level rise was based on projected sea levels for the years 2070 and 2115 using estimates from the National Planning Policy Framework (NPPF) and from the UK Climate Projections 2009 (UKCP09) data. This information is used for decision making on the sustainability of development within flood risk areas and early warning on weaknesses in the current defence line.

Finally, improvements to the existing flood warning systems were required to help ensure that suitable warnings are issued for areas at flood risk and to limit the incidence of unnecessary warnings for unaffected areas. The sensitivity analysis described as part of the first objective was also be used to inform the range of possible impacts on communities within the flood risk zone.

The principal driver for the study is to gain an understanding of the coastal flood risk in the study area making use of the most up to date data. In February 2011, the Environment Agency's Evidence Directorate published the study *Improved Coastal Flood Boundary (CFB)¹ Conditions for the UK mainland and islands*. This project, which resulted in the derivation of new extreme sea-level estimates, was undertaken as part of the joint Environment Agency/Defra Flood and Coastal Erosion Risk Management Research and Development Programme (SC060064). As a result of the new estimates, the tidal Flood Zones produced previously are now out-of-date. Finally, in July 2011 new guidance with respect to future sea-level rise was released based on the UKCP09 report.

The main objectives of the study were to:

- produce tidal graphs based on the CFB sea-level data;
- produce a wave transformation model to transform the offshore waves into the nearshore;
- schematise the flood defences to calculate wave overtopping along the coastal frontage;
- use ForeCoast to assess the frequency of overtopping and flooding over a period of 20-years using hindcast data;
- produce two dimensional (2D) TUFLOW hydrodynamic models to represent flood depths, velocities, extents, and hazards for the open coast, estuaries and harbours along the coastline;
- estimate flood levels for the 20, 5, 3.33, 1.33, 1, 0.5, 0.2 and 0.1% Annual Exceedance Probability (AEP) flood events;

¹ Defra, SEPA, The Scottish Government, Environment Agency (2011). Coastal flood boundary conditions for UK mainland and islands. Project: SC060064/TR2: Design sea-levels.

- estimate flood levels for the 0.5% AEP plus climate change flood event based sea-level rise estimates from the National Planning Policy Framework and the medium emission 95th percentile UKCP09 scenario for the years 2070 and 2115;
- produce breach model flood extents for five locations and three AEP events;
- map extents of the above events for both defended and undefended scenarios and produce updated Flood Zones 2 and 3;
- derive the ABDs;
- produce flood onset and duration grids for a selection of scenarios;
- produce Flood Hazard Rating outputs;
- provide estimates of the SoP provided by the defences;
- update the coastal Flood Warning Areas and use ForeCoast to optimise the Flood Alert and Warning thresholds against the hindcast data; and
- produce a study report summarising all of the work undertaken.

The dominant source of flood risk in the tidal floodplain is related to the coincident occurrence of a high astronomical tide, the passage of a storm surge and the effects of both sea and wind related waves. This risk was the main focus of this study. The study did not investigate the flood risk associated with fluvial flooding, surface water or groundwater flooding.

1.2 Study extent

The study area extended from Emsworth in the west to Littlehampton in the east, encompassing the communities of Emsworth, Bosham, Chichester, Wittering, Bracklesham, Selsey, Pagham, Bognor Regis and Climping. Figure 1-1 shows the study area.



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Figure 1-1: Study extent

The following sections describe the main urban areas within the study extent moving from Emsworth in the west to Littlehampton in the east.

1.3 Emsworth

Emsworth is located at the north western corner of Chichester Harbour. This area is sheltered from the effects of waves but is at risk from still water flooding. The defended and undefended model extents from the previous studies indicate that there are few defences.

1.4 Chichester

Although not directly exposed to oceanic forces and largely sheltered by land, the Chichester area is considered a flood risk through still water flooding. The area of Chichester incorporates many urbanised areas as well as important infrastructure. Flooding through overtopping is not considered to be a risk at this site due to the sheltered nature of the location. Four sites were included in the wave overtopping calculations in the Chichester Harbour model, including three at West Wittering and one at Emsworth to confirm whether wave overtopping poses additional risk.

1.5 Wittering

West Wittering has in recent years been inundated through tidal still water flooding and has consequently had coastal defences built and improved to prevent flooding. It was recommended in the North Solent Shoreline Management Plan², and the Pagham to East Head Coastal Defence Strategy, that improvements are made to West Wittering's flood defences. The defences at this location comprise of large sand and shingle beaches leading to a long earth and rock embankment which protects the low lying planes which are designated as a flood risk zone. The three defences in the Chichester Harbour model mentioned above incorporate this stretch of defence.

East Wittering is located on the open coast. The frontage along this stretch of the coast faces south-west leaving it vulnerable to both storm waves generated by the local prevailing wind, and swell waves which have an uninterrupted pathway from the Atlantic. The wave exposure here means that the risk of wave overtopping is high and it is this that forms the main coastal flood mechanism for the area. The defences along this stretch are primarily comprised of shingle beaches with regular groynes and an intermittent low earth embankment reinforced with timbers for small lengths of the shoreline.

1.6 Bracklesham Bay

The Environment Agency's Flood Zone maps show regions of low lying land likely to be at flood risk in the Bracklesham Bay area which extends inland. The reclaimed marsh land to the south-west towards the Medmerry outfall area is at a particular risk of flooding. The Medmerry coastline itself has in recent years been breached, the sediment washed out and flood water inundating into the adjacent low lying areas. The Environment Agency has since built major new flood defences between Bracklesham and Selsey to the south, which fall under the Medmerry managed realignment scheme. The original coastline has since fallen back to create a vast new flood designated area as well as designated salt water marsh and wetlands along the river bank. On edge of the newly created wetlands is an earth and rock embankment, which is illustrated in Figure 1-2.

² Beachy Head to Selsey Bill Shoreline Management Plan First Review, Final Report, May 2006, Halcrow
2014s0753_Emsworth to Littlehampton_Coastal final report_v2.docx



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Figure 1-2: Medmerry managed realignment scheme

1.7 Selsey

The town of Selsey is a major Sussex south coast resort attracting a large number of visitors each year in local holiday parks. The town is situated to the south-east of Medmerry and the new defence system. The town lies on a peninsular and is therefore exposed to open sea from the east and the west.

The land north of the town is low lying and has historically been flooded as mentioned in Section 2.2. Much of this coastline is regularly exposed to storm and high water waves and is, therefore, protected by a series of significant defences. The west side being exposed to Atlantic storms has a shingle beach backed by high concrete wave return walls, promenades, rock dumped armouring and rock revetments. Given the height of the defences this section of coast is highly unlikely to flood during still water events. However, wave overtopping should be considered a mechanism for flooding and in recent years the threat from waves has been severe. To the west of Selsey, a new privately funded defence scheme has been constructed to protect the 3,000 holiday chalets and static caravans at the Bunn Leisure complex. The scheme incorporates rock groynes and a wide recharged beach, Figure 1.3.



Figure 1-3: Wide shingle beach and rock groynes at Bunn Leisure complex

The eastern side of Selsey has defence crest levels which are capable of defending the town against the most extreme still water levels. Although there may be lower potential wave energy when compared to the west side, refracting waves and storms from easterly directions means that wave overtopping is still the main coastal flood risk mechanism for this stretch of coastline. Large stretches have long continuous defences tying in to high ground; including a long promenade and wave return wall with high crest levels around dense urban areas, significant infrastructure and property. There is a shingle beach which extends along this area and varies in width and height. Historically Selsey has been periodically cut off from the mainland when low ground between Pagham and Bracklesham has become inundated. The new managed realignment at Medmerry should prevent this from being an immediate threat in the near future.

1.8 Pagham

Pagham lies to the north-east of Selsey and is separated by a large lagoon and river system. The coastline stretching from Pagham lagoon to the western extent of the defences at Bognor Regis is south-east facing and therefore somewhat sheltered from Atlantic storms; however it is exposed to refracting swell wave, localised fetch limited storm waves and extreme sea levels. Wave overtopping is not the primary flood mechanism at Pagham although it is exposed and has flooded in the past due to beach erosion of the natural sediments, and high water during periods of increased set-up and storm surge events. The primary defence running the length of this site is shingle beach defences being natural and manmade. Since the 1960s the shingle beach has been managed through both groynes and beach recharge. A natural build-up of material on the harbour spits has meant that recharging has not been required since 2004. This build-up of material has impacted on Pagham Beach as the spit has grown and migrated towards the beach; if this migration continues the spit could eventually close the entrance to the harbour (Figure 1.4). The proximity of the spit has forced the incoming and outgoing tidal currents into a narrower channel which has resulted in the erosion of Pagham Beach, potentially putting the properties on the beach at risk. Emergency repairs in the form of rock armour have been put in place to provide some protection for the beach. A process of adaptive management was adopted and urgent works to manage the erosion were undertaken. The coastal management at Pagham is detailed in the Pagham to East Head Coastal Defence Strategy.



Figure 1-4: Pagham Beach and spit now almost converge

There are a number of management plans associated in this area that have been completed or are in the process of being improved in the near future:

- Adaptive management commenced at Pagham and East Head (2009)
- Medmerry managed realignment (2014)
- Selsey West Beach improvements (2013)
- East Wittering and Bracklesham improvements (2014)
- West Wittering flood banks (2013).

1.9 Bognor Regis

The town of Bognor Regis is a major holiday destination for the south coast of the UK and is the largest urbanised area within the study extent. Its coastline faces predominantly to the south-east and is exposed to wave action. Historically the area has been inundated by extreme sea levels during surge and storm events. The Aldingbourne rife discharges through a pumped outfall and is at risk from tide locked fluvial flooding and inundation from wave overtopping of the defences.

Overtopping is also a coastal flooding mechanism in this area but only in extreme events due to the level of coastal protection. The coastal defences are predominantly shingle ridges with some sections also having additional protection from wave return walls, wide promenades, and rear splash walls (Figure 1.5).



Figure 1-5: Flood defences in Bognor Regis

The area between the town centres moving towards Elmer includes a promenade with lower crest levels and a shingle beach. A series of offshore breakwaters have been constructed to help stabilise the beach and provide additional coastal protection (Figure 1.6).



Figure 1-6: Elmer coastal defence and beach management system (image courtesy of South Downs Coastal Group)

http://www.sdcg.org.uk/sdcg_right2.html)

1.10 Climping

The town of Climping lies close to the River Arun and is at risk from both coastal and fluvial flooding. The few properties that do lie within close proximity to the coastline are protected by a high wall and irregular rock armouring. The first line of defence in the area is the shingle beach which historically has been monitored and managed by the Environment Agency. However, the Environment Agency will cease maintenance of this defence in the near future. The A259 road crossing to the north of Climping is elevated above the floodplain and has been assessed as providing a 1.5% AEP standard of protection against fluvial flooding.

Outside of the urban stretches of coastline the defences become simpler, comprising of shingle beaches, high earth banks and sand dunes.

1.11 Report structure

In addition to this introductory chapter, the report includes the following chapters and appendices:

- **Chapter 2 (Qualitative description of flood response)** reviews the mechanisms of coastal flood risk in the study area.
- **Chapter 3 (Modelling approach)** outlines the three principal components of numerical modelling applied, including wave transformation, wave overtopping and flood inundation modelling.
- **Chapter 4 (Input data)** discusses the data that were gathered and used within the study and how these were applied within the modelling.
- **Chapter 5 (Technical method and implementation)** summarises the approaches taken to model the wave transformation, wave overtopping and flood inundation.
- **Chapter 6 (Model proving)** summarises the model sensitivity tests and the performance against the calibration and validation events.
- **Chapter 7 (Results)** discusses the model results and summarises the flood risk in the main urban areas.
- **Chapter 8 (Flood warning updates)** outlines the proposed updates to the Flood Alert and Flood Warning areas.
- **Chapter 9 (Conclusions)** summarises the main findings from the study and includes some 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 sandy beaches, sand dunes, shingle beaches, hard and soft defences, breakwaters, groynes and a major managed realignment scheme at Medmerry. The tidal flood defence network 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 largely tidally influenced estuaries of the West Sussex coastline. 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.

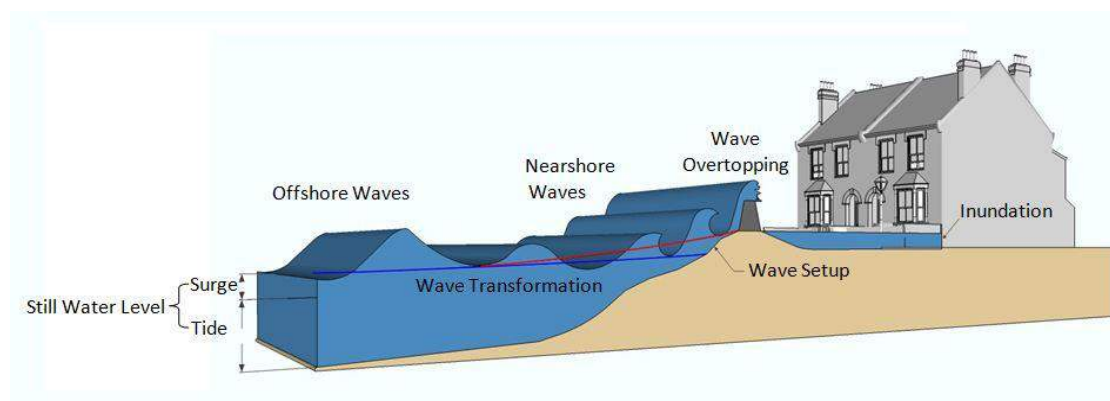


Figure 2-1: Components of sea-level variation that lead to coastal flooding

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. For the exposed locations, such as the coastal frontage at Selsey and Bognor Regis, most flooding can occur 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 shingle beaches.

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 sources of flooding 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 Chichester and Pagham harbours), 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.

3 Modelling approach

3.1 Overview

The Sussex coastline is dominated by 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.

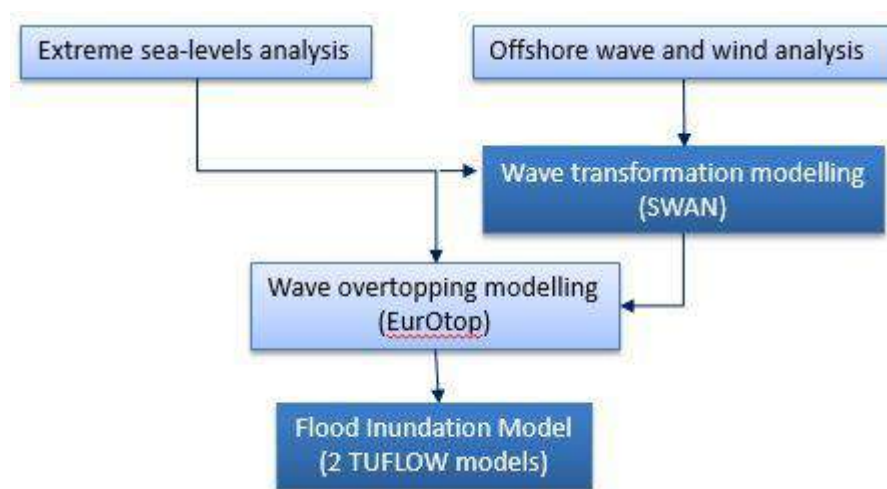


Figure 3-1: Coastal flood modelling system architecture

3.2 Wave transformation modelling

To transform offshore wave heights into the nearshore, 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 Technology³. A new SWAN model was developed for use within this study. The wave transformation model development, calibration and simulations are discussed further in Chapter 9.

3.3 Wave overtopping

As mentioned in section 2.2 wave overtopping has to be calculated separately, as no one model is capable of simulating both still water flooding and wave overtopping. The techniques used to calculate wave overtopping values are discussed in Chapter 5 and are based on EurOtop methods. There are some uncertainties inherent in this process 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.

3.4 Inundation modelling

Flood inundation modelling was undertaken using TUFLOW⁴; 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 Arun, 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

³ SWAN User Manual, SWAN Cycle III version 40.81, Delft University of Technology, 2010

⁴ 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.

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. TUFLOW is particularly good at representing the progression of floodwaters in the out of bank situations brought about by extreme events.

3.5 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 an 8m grid resolution. An 8m grid size allows accurate representation of the general topography and bathymetry and provides detail on possible flow routes. An 8m grid size will not accurately represent some of the finer topographic details (i.e. anything smaller than 8m), however, the important features such as defences will manually be read into the model to ensure they are represented. Following the completion of the modelling within this study, the Estuary modelling standards (CH2MHill, 2015) were released and these recommend a grid resolution of less than 6m to be able to accurately model flow velocities and hazards. The models are described in more detail in the Model Development Report that accompanies this summary report.

4 Input data

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 progression of the flood wave in the model was appropriate. This involved the use of detailed Ordnance Survey (OS) mapping data.

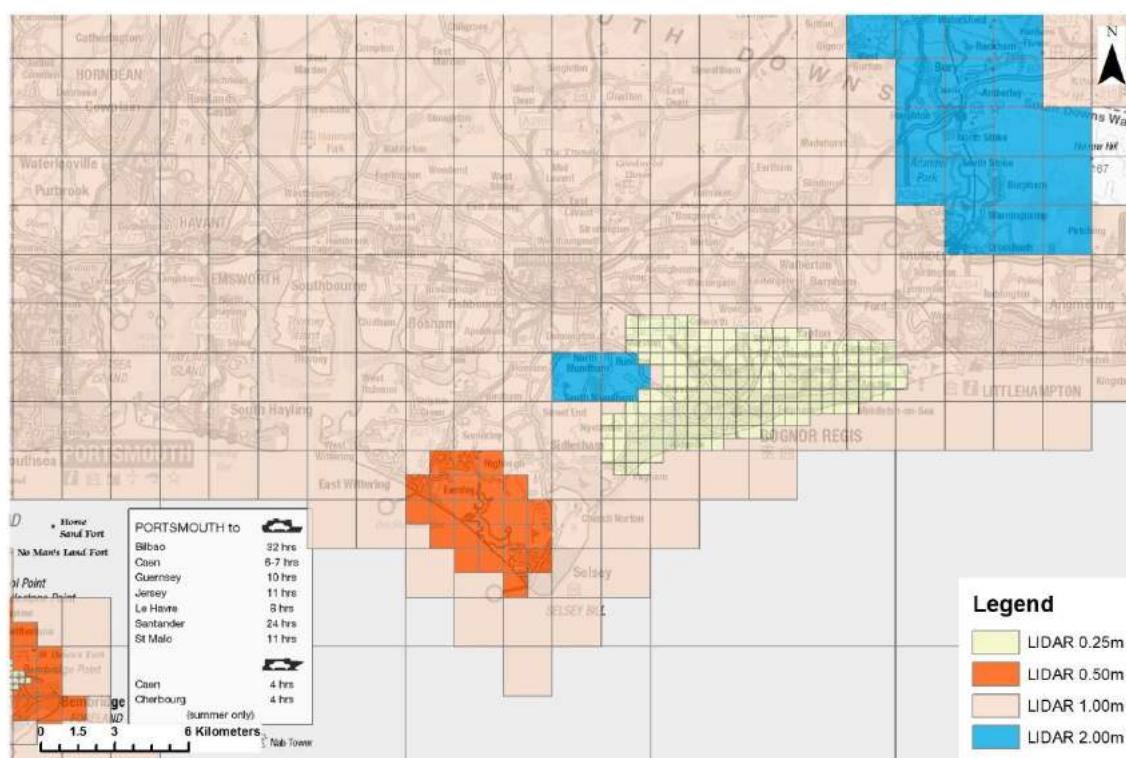
Terrain data

Light Detection and Radar (LIDAR) data for the project were provided by the Environment Agency and form the main source of terrain data used. The LIDAR data available to the project can be seen in Figure 4-1. These datasets were supplied in 0.25m, 0.5m, 1m and 2m resolution grids with each dataset covering some portion of the floodplain.

LIDAR coverage in the study area is complete (Figure 4-1) with the majority of the area covered with 1m data. The higher resolution data was used in preference to the lower resolution data so the 0.25m LIDAR is layered on top of the 0.5m, the 0.5m on top of the 1m and the 1m on top of the 2m such that the model used the most accurate elevation data to create the model grid. The LIDAR data are of higher resolution than necessary to represent the surface of the floodplain for modelling purposes, as the model grid size is 8m, but it was still used and read into the model to benefit from the more accurate levels. At Pagham harbour the spit is evolving rapidly; the LIDAR used in this area was from a 1m resolution survey in October 2010.

Once the holes in the DTM were filled using interpolation techniques, 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 Digital Terrain Model (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, lines called gully lines, were drawn across the structures in a Geographical Information System (GIS). These gully lines were then used by TUFLOW, during a model simulation, to cut through the DTM and 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. Many of the drainage channels on the floodplain are of the order of a metre or two wide. Consequently, many of the culverts associated with these small drains are of a similar width. The resolution of the modelling for this study was 8m; if gully lines were used for these structures the flow allowed through would be unrealistically large. For these smaller structures such as culverts on drainage ditches and river channels less than 8m 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.

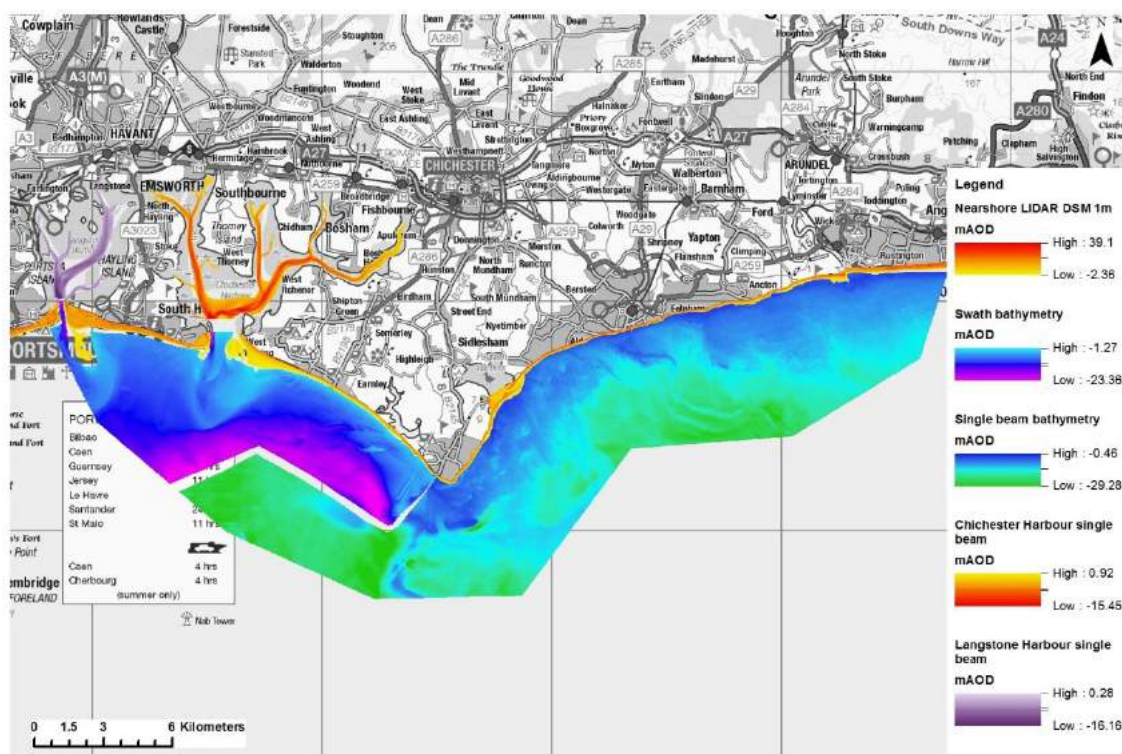


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Figure 4-1: Availability of Light Detection and Ranging data

Bathymetry data

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



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Figure 4-2: Bathymetry data

Land use data

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

4.2.2 Flood defence data

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

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

⁵ Chow, V.T. (1959). Open-Channel Hydraulics. McGraw-Hill: Auckland.
2014s0753_Emsworth to Littlehampton_Coastal final report_v2.docx

dataset is generally used to link the points and allow TUFLOW to interpolate a continuous defence line within the model.

Defence elevation data

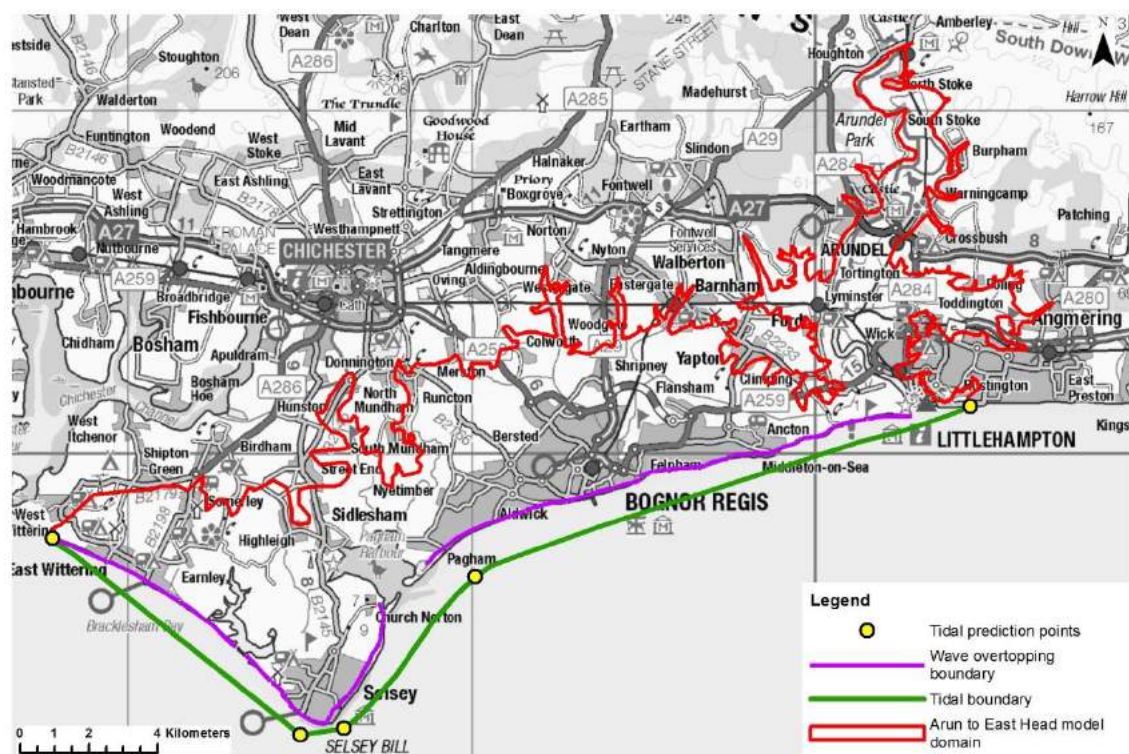
The basis of the defence dataset created for this study was the existing defence data from the Chichester Harbour models, the surveyed crest heights from the beach survey data from the Channel Coastal Observatory (CCO) and levels from high resolution LIDAR data. All defence elevations were verified against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

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

4.2.3 Inflow boundary data

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

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



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Figure 4-3: Arun to East Head model domain



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Figure 4-4: Chichester Harbour model domain

Offshore still water boundary

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

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

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

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

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

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

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

Climate change

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

Wave overtopping

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

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

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

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

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

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

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

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

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

Wave conditions

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

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

4.2.4 Flooding from other sources

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

5 Technical method and implementation

5.1 General methodology

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

5.2 Wave transformation modelling

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

5.2.1 Wave modelling software

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

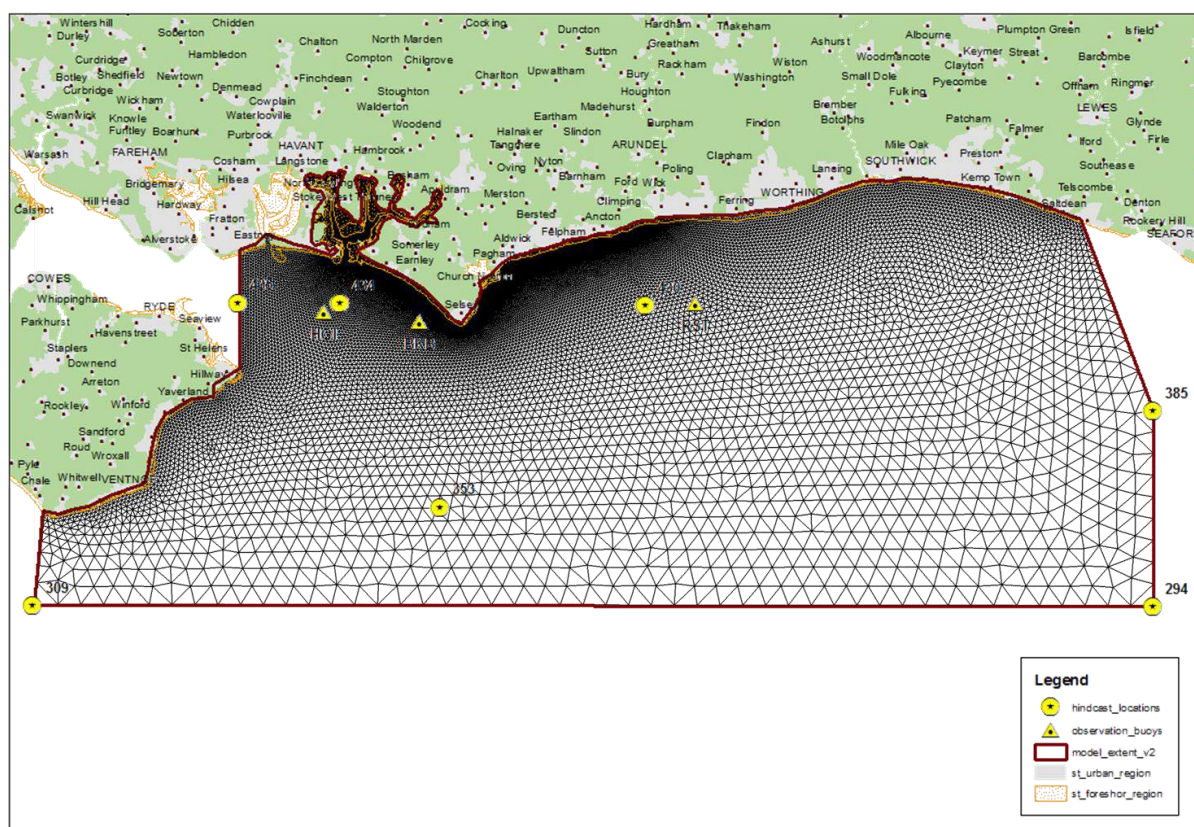
- Wave growth due to wind
- Wave – wave interactions (quadruplets and triads)
- Wave breaking
- Bed friction
- White-capping
- Diffraction
- Refraction
- Reflection.

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

5.2.2 Model domain

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

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

5.2.3 Terrain and bathymetry data

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

5.2.4 Wave model mesh development

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

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

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

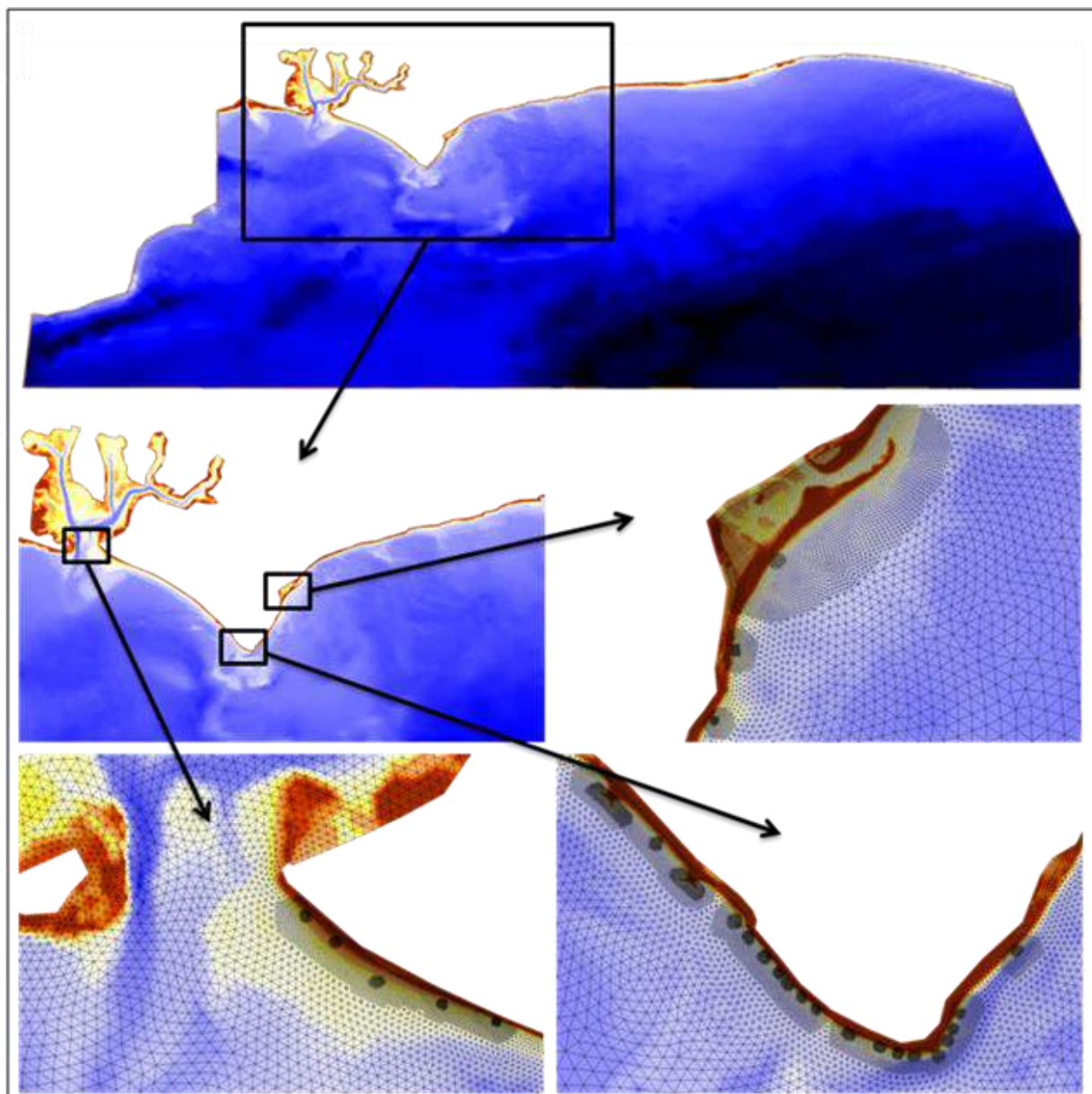


Figure 5-2: Final model bathymetry and unstructured model mesh

5.2.6 Sea-level boundary data

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

5.2.7 Wave boundary data

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

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

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

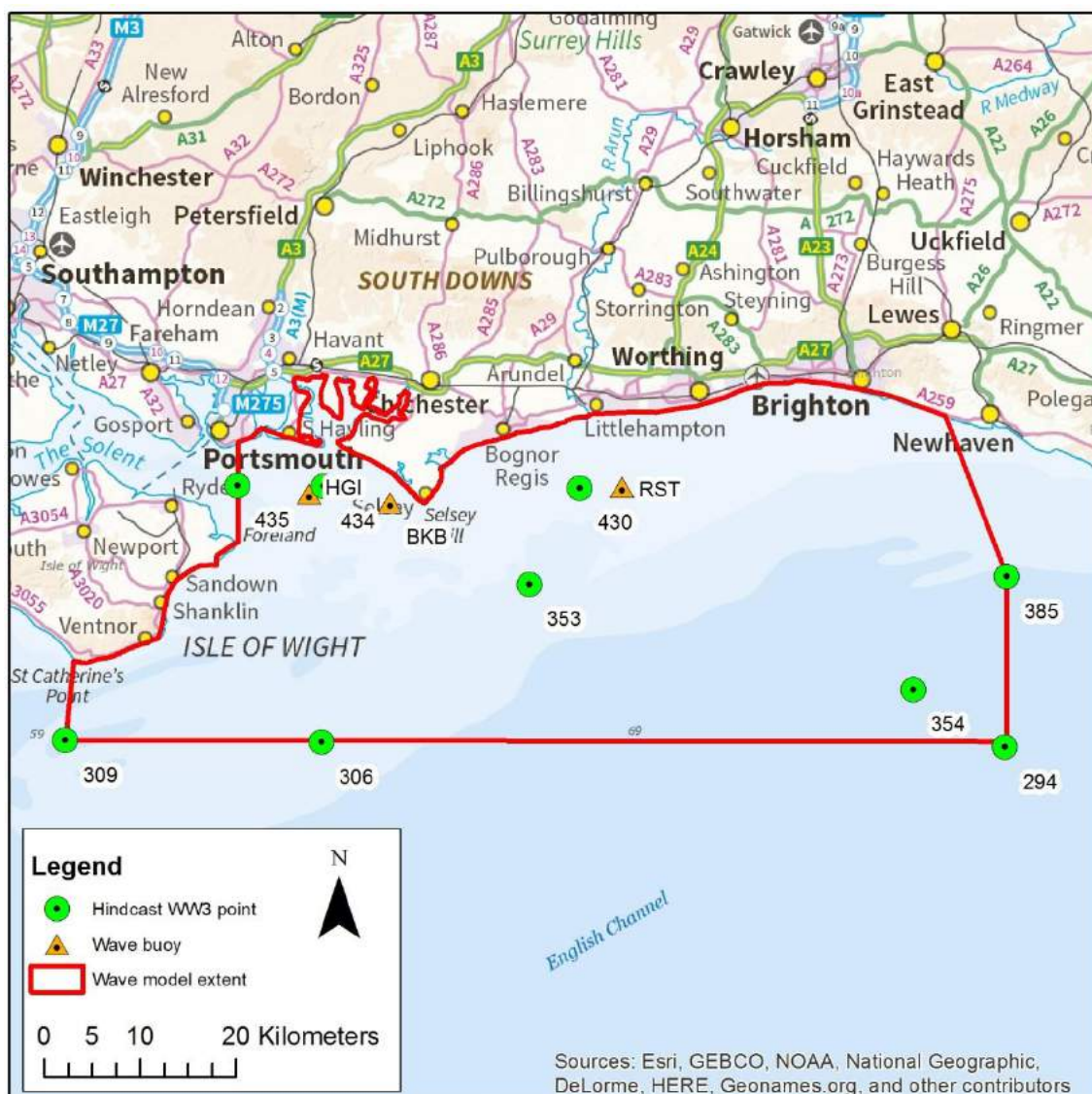


Figure 5-3: Location of Met Office hindcast wave data points used for comparison of offshore conditions

Extreme value analysis

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

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

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

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

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

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

5.2.8 Joint probability of sea-level and wave heights

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

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

5.2.9 Wind boundary data

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

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

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

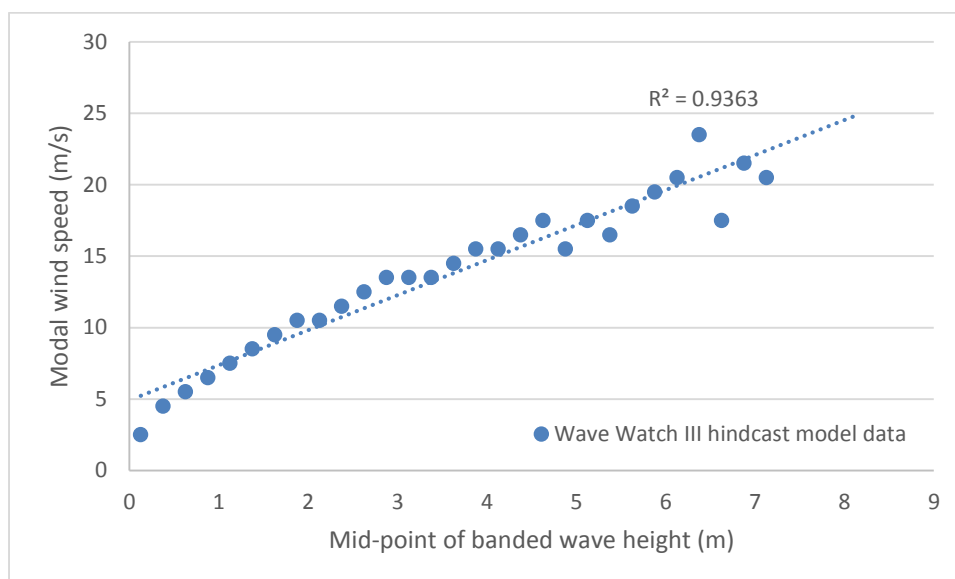


Figure 5-4: Wave height and wind speed relationship for the south-west

5.2.10 Wind and swell waves

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

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

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

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

5.2.11 Wave period boundary data

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

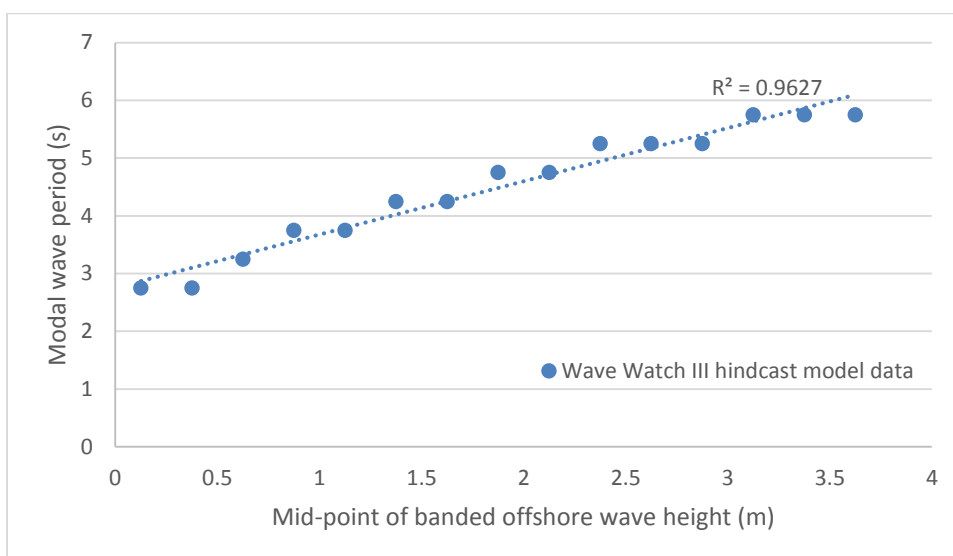


Figure 5-5: Wave Watch III hindcast south-east wave regression relationship between wave period (y) and offshore total sea wave height (x)

5.2.12 Swell wave period

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

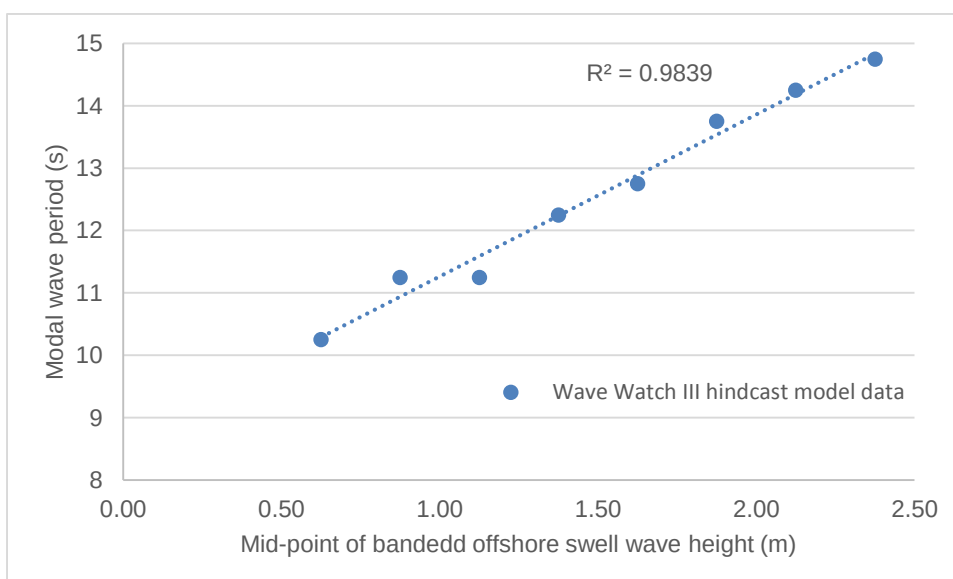


Figure 5-6: Wave Watch III hindcast south-west wave regression relationship between wave period (y) and offshore swell wave height (x)

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

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

5.3 Wave overtopping modelling

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

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

5.3.1 Derivation of Wave Overtopping Inputs

5.3.2 Sea-state Conditions

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

5.3.3 Defence Profile Shape

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

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

5.4 Calculation of Wave Overtopping

5.4.1 Overtopping Model

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

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

Table 5-2: Neural Network input parameters

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

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

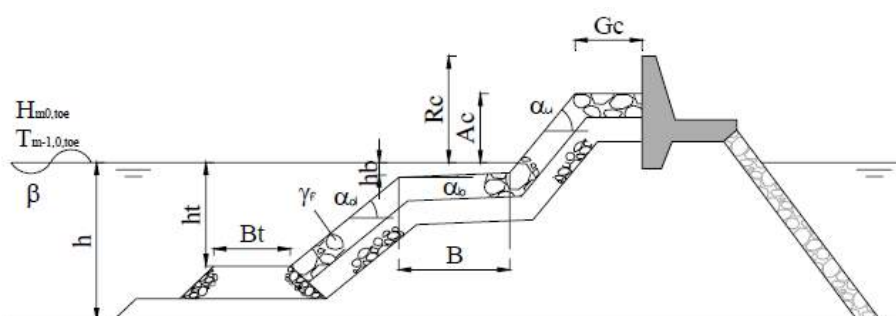


Figure 5-7: Parameters used (and the descriptions) in Neural Network schematisation for wave overtopping modelling¹¹

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

5.4.2 Overtopping Simulation

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

5.5 Flood Inundation modelling

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

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

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

5.5.1 Model approach

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

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

5.6 Model outputs

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

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

5.6.1 Flood outlines

Remove dry islands

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

Wave overtopping gaps filled

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

Final revised Flood Zones

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

5.6.2 Creation of areas benefiting from defences

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

5.6.3 Hazard rating grids

Flood Hazard Rating grids were generated as part of this study. The grids required for these maps were generated according to the following equation:

$$\text{Hazard Rating (HR)} = \text{Depth} \times (\text{Velocity} + 0.5) + (\text{Debris Factor})$$

This equation is in line with the Defra's hazard rating thresholds¹². A debris factor of 0.5 was used for depths greater than 0.25m and a debris factor of 1.0 was used for depths less than 0.25m. These debris factors correspond to those recommended for urban floodplains in Defra's Supplementary Note. This Note also provides guidance on classifying Flood Hazard Ratings, as detailed below. These were adopted for the study.

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

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

5.6.4 Onset of flooding

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

5.6.5 Flood inundation animations

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

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

6 Model proving

6.1 Introduction

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

6.2 Wave transformation model calibration

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

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

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

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

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

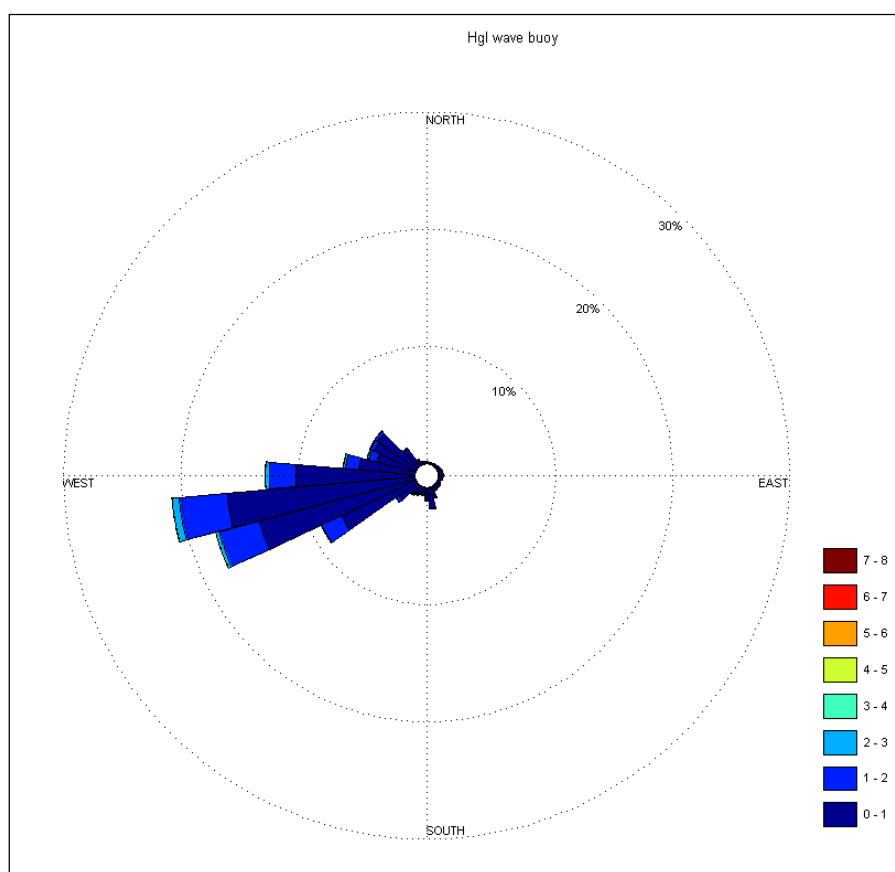


Figure 6-1: Wave direction histogram illustrating observations at Hayling Island (HGI) waverider buoy. Filled bins are wave heights (m)

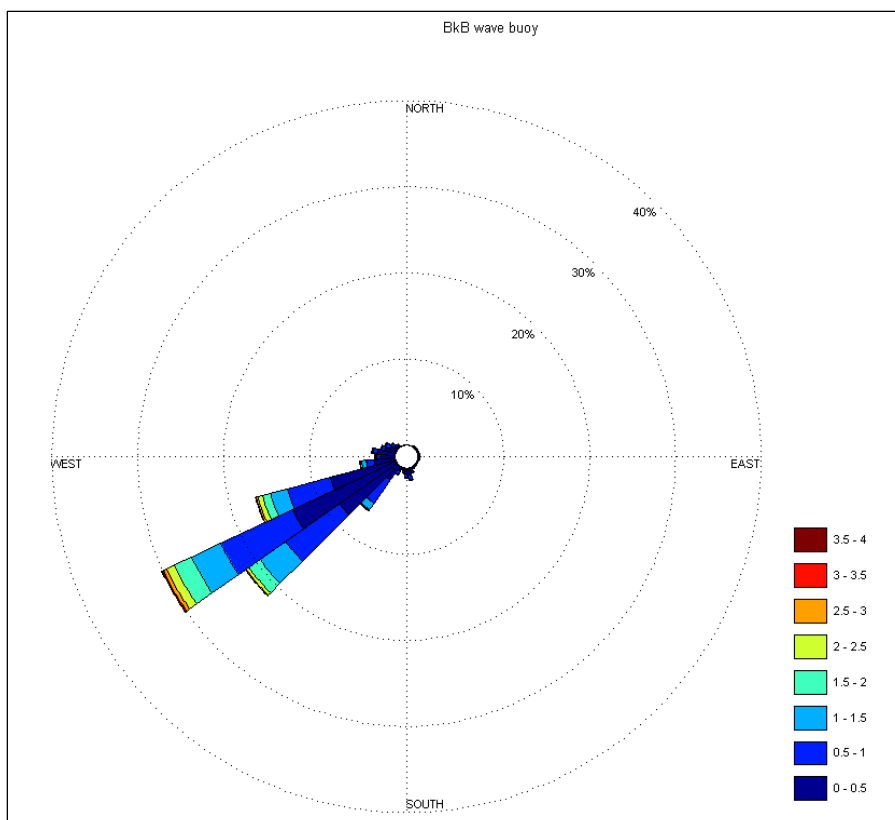


Figure 6-2: Wave direction histogram illustrating observations at Bracklesham Bay (BKB) waverider buoy. Filled bins are wave heights (m)

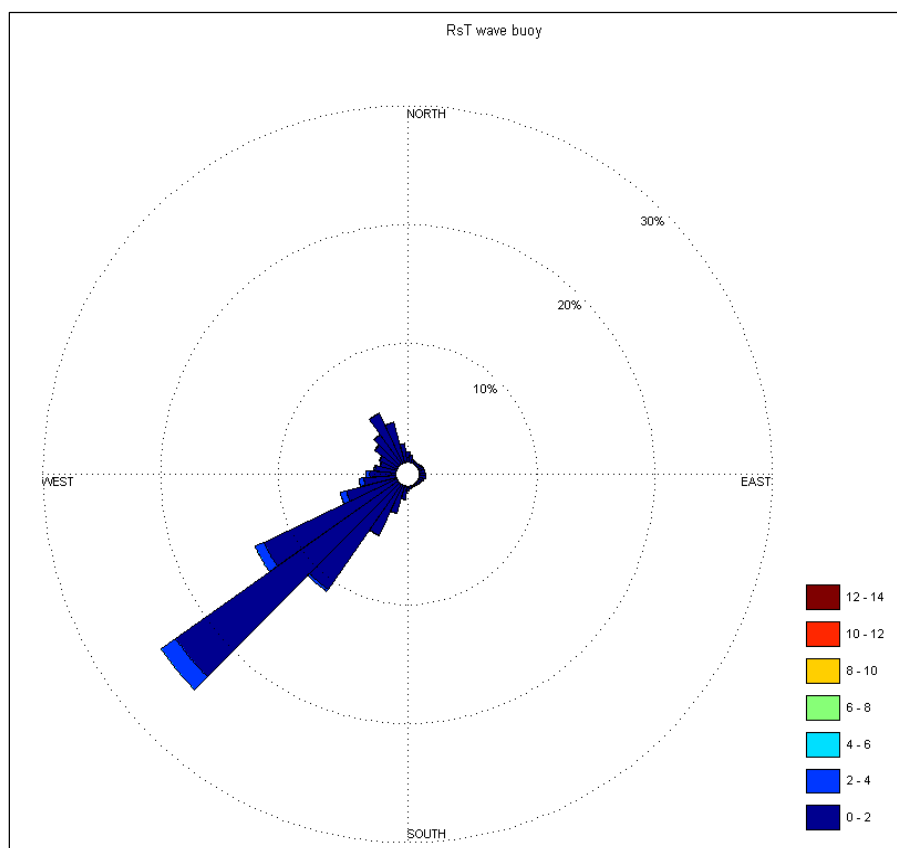


Figure 6-3: Wave direction histogram illustrating observations at Rustington (RST) waverider buoy. Filled bins are wave heights (m)

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

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

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

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

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

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

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

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

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

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

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

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

6.2.1 Calibration: Winter 2014 Storms

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

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

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

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

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

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

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

6.2.2 Model simulations

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

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

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

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

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

6.2.3 Model outputs

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

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

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

6.3 Wave overtopping model validation

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

6.3.1 Frequency analysis of wave overtopping

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

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2 Calibration of wave overtopping models using flood inundation model

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

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

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

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

6.4 Flood inundation model calibration

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

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

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

6.4.1 Arun to East Head model

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

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

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

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

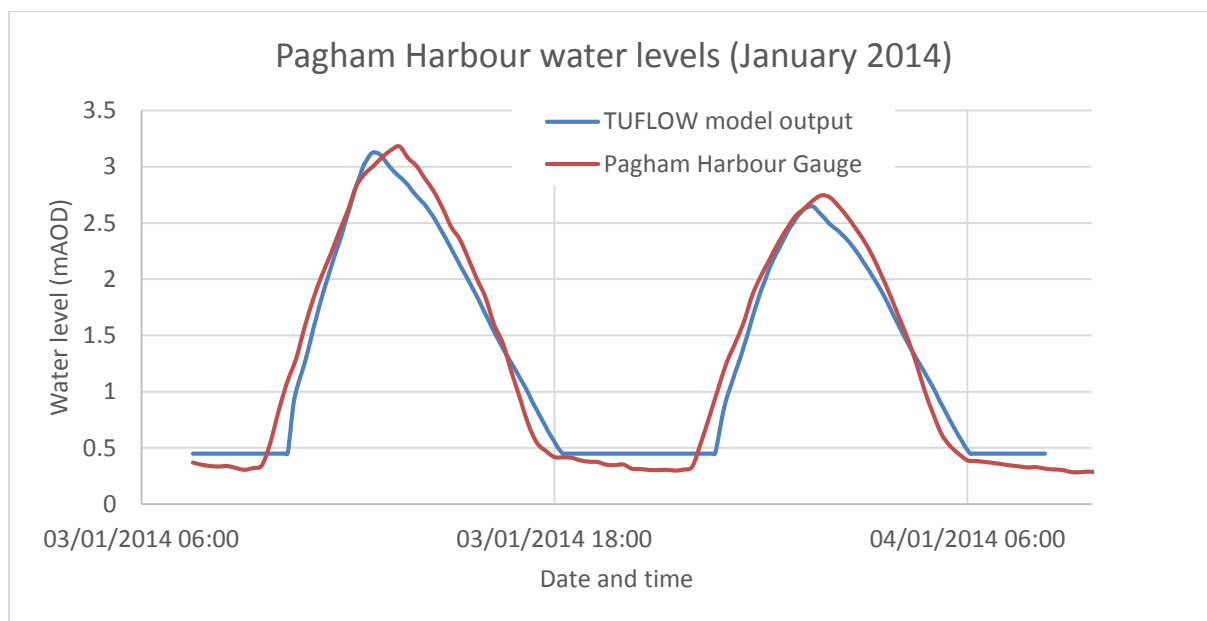


Figure 6-4: Model and observed data from the January 2014 event

6.4.2 Chichester Harbour model

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

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

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

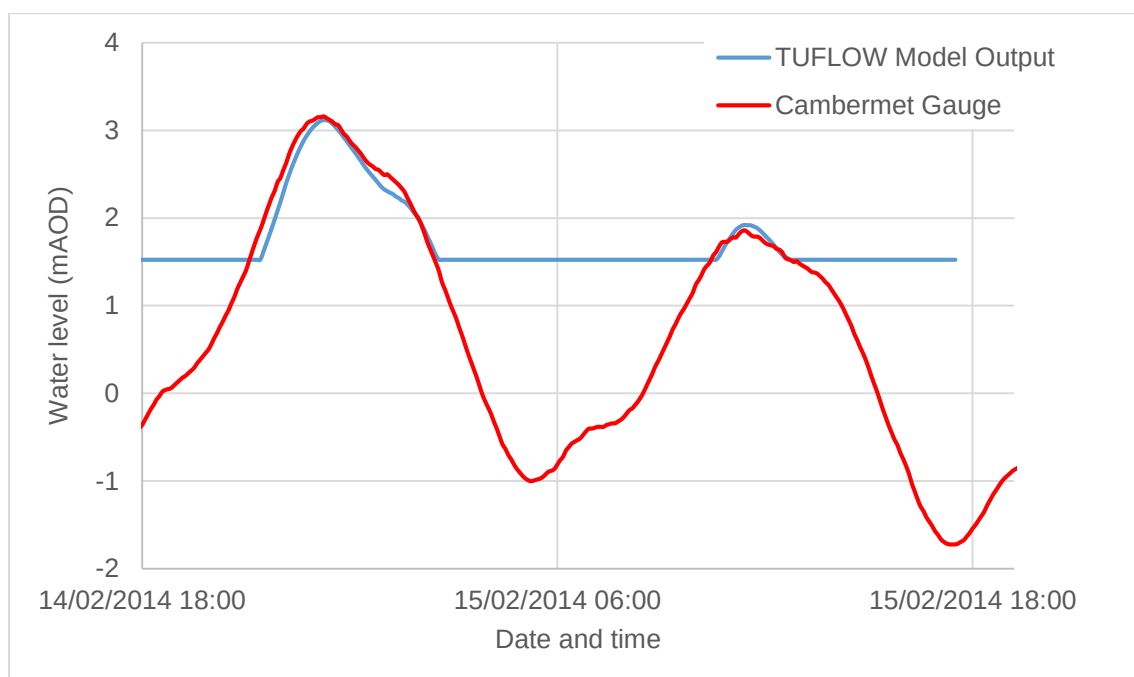


Figure 6-5: Model and observed data from the February 2014 event

6.5 Sensitivity testing

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

6.5.1 Sensitivity to model roughness

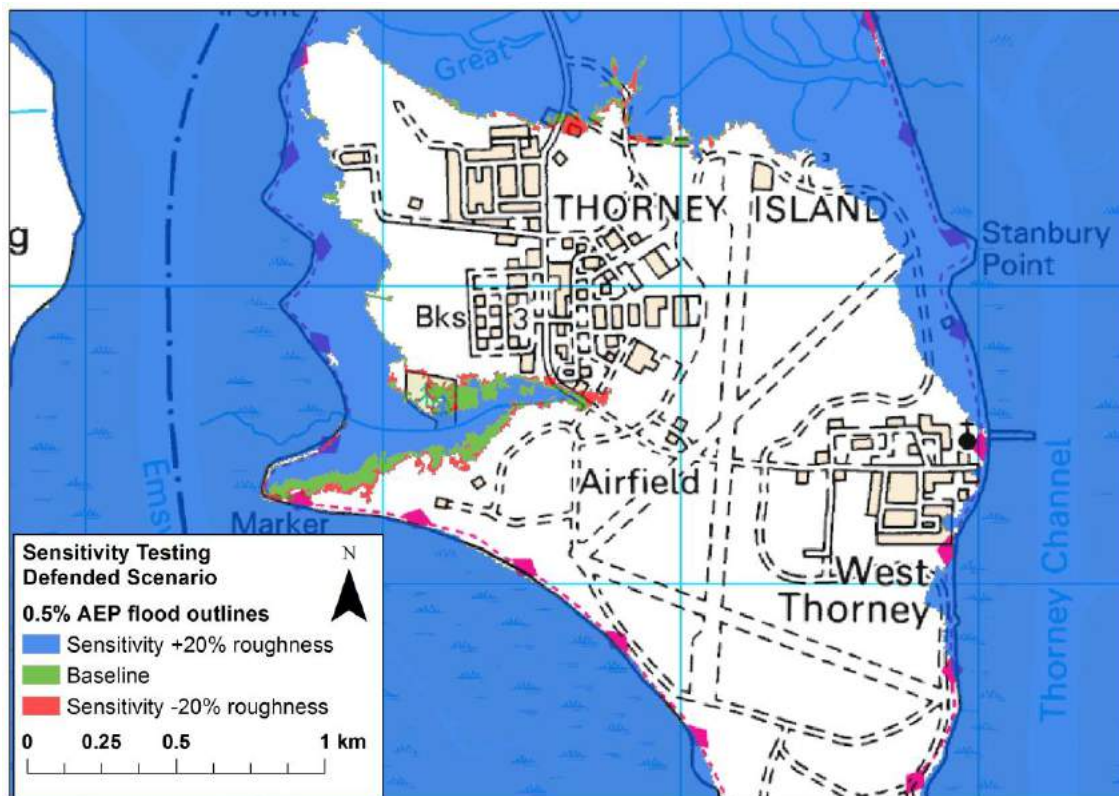
Overall there was very little change to the flood extents when an increase or decrease to the model roughness values were applied. The area with the most noticeable change in extent in the Chichester Harbour model was on Thorney Island where depths of flooding on the floodplain changed by $\pm 0.30\text{m}$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

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

6.5.2 Sensitivity to representation of buildings

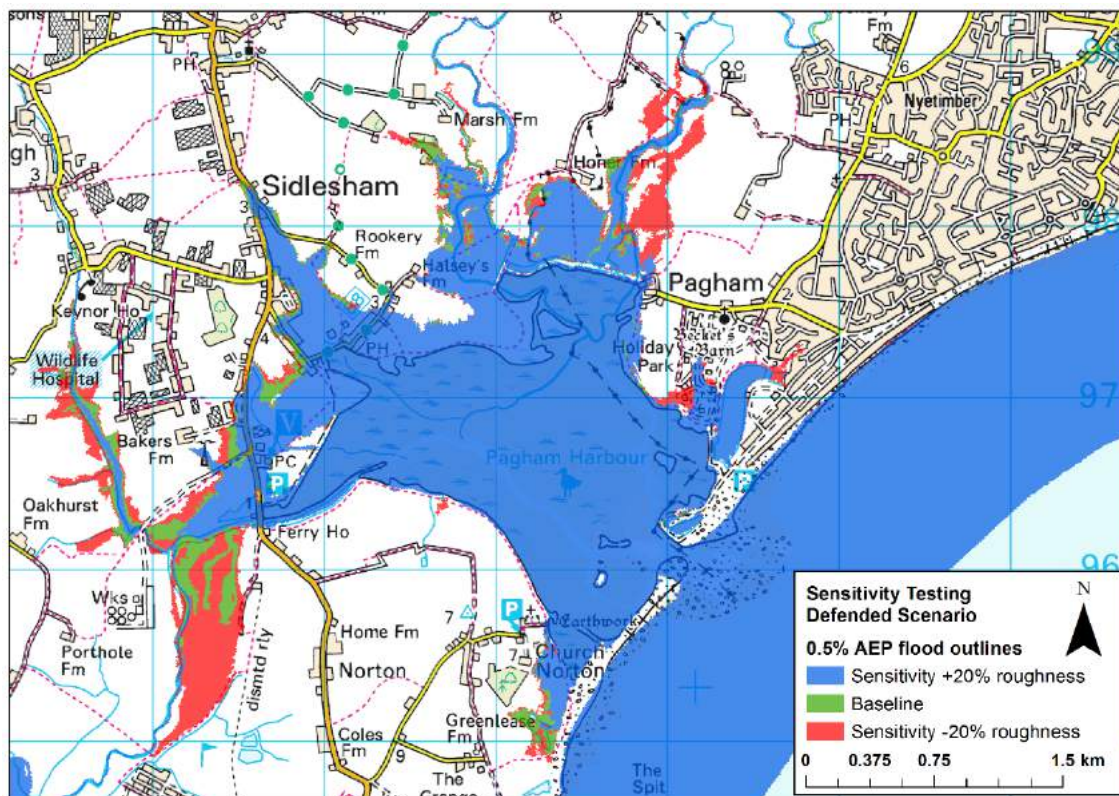
In the design simulations the buildings were represented by areas of increased roughness. For sensitivity testing two alternative approaches were simulated. In the first the building footprints were raised by 0.3m to represent flooding commencing when a threshold is reached in combination with increased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

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



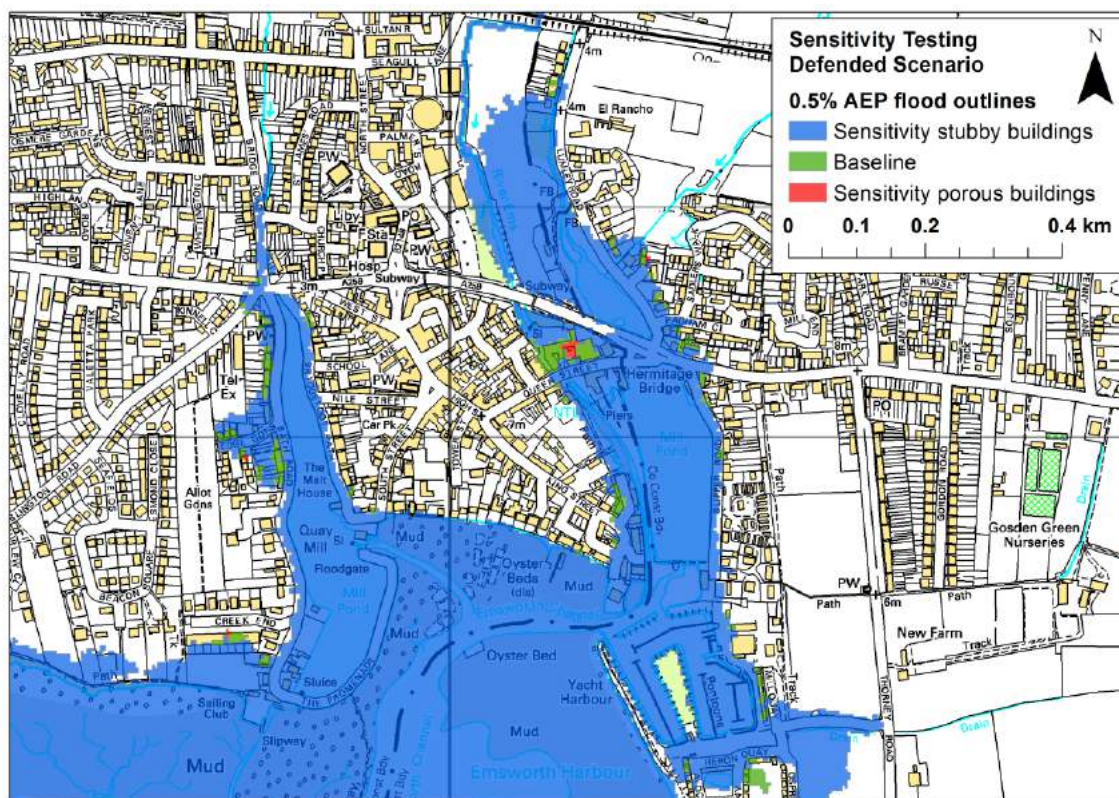
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Figure 6-6: Sensitivity to model roughness in the Chichester Harbour model



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



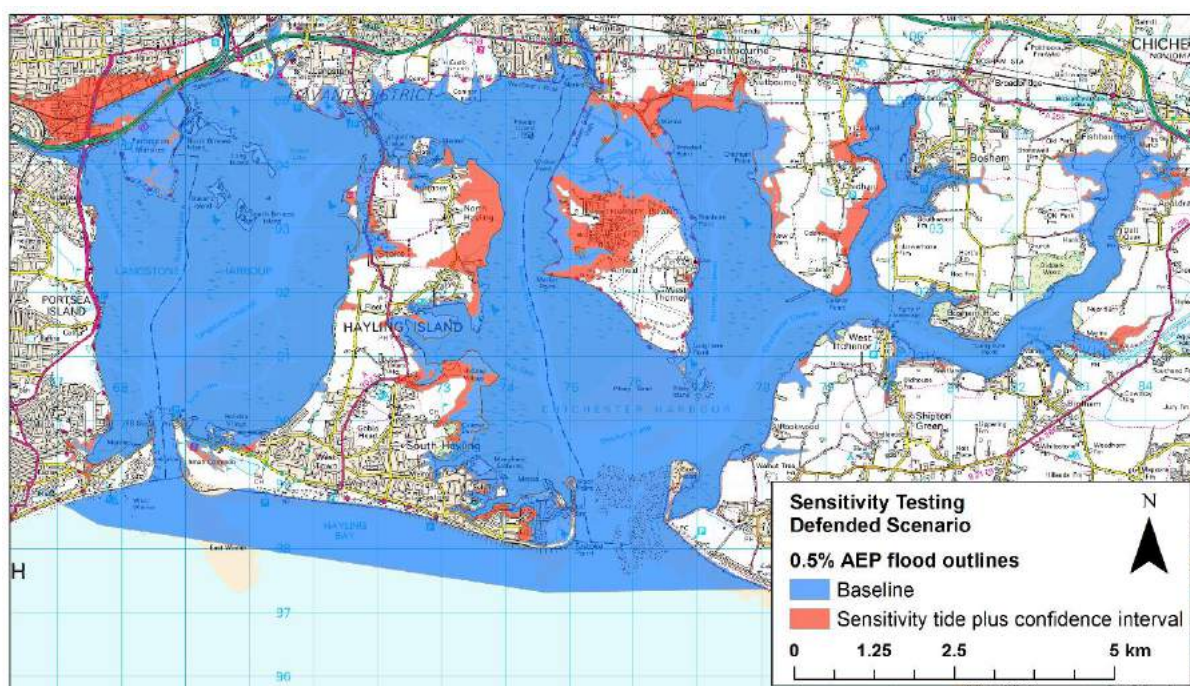
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Figure 6-8: Sensitivity to building representation within Emsworth

6.5.3 Sensitivity to downstream boundary

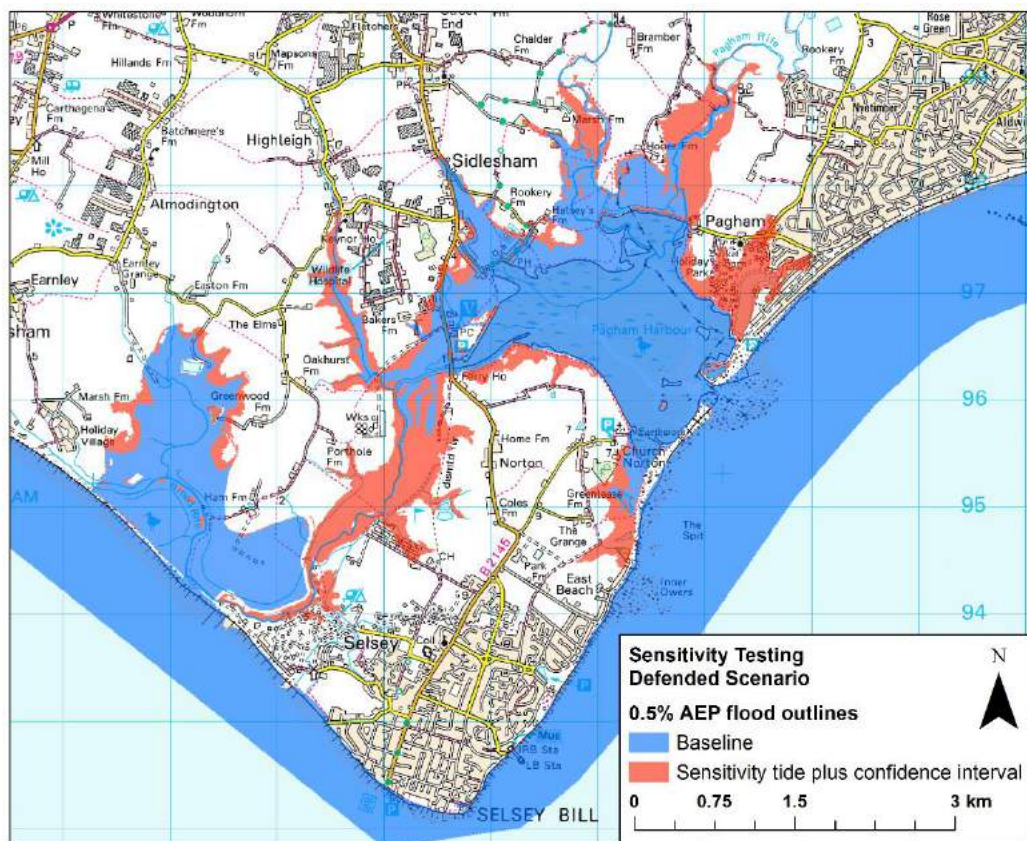
A final sensitivity test performed on the flood inundation model was to test the sensitivity to the downstream boundary. This was tested by adding on the confidence interval values representing the uncertainty in the CFBD sea-levels for the 0.5% AEP event. For the Chichester Harbour model the increased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

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



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



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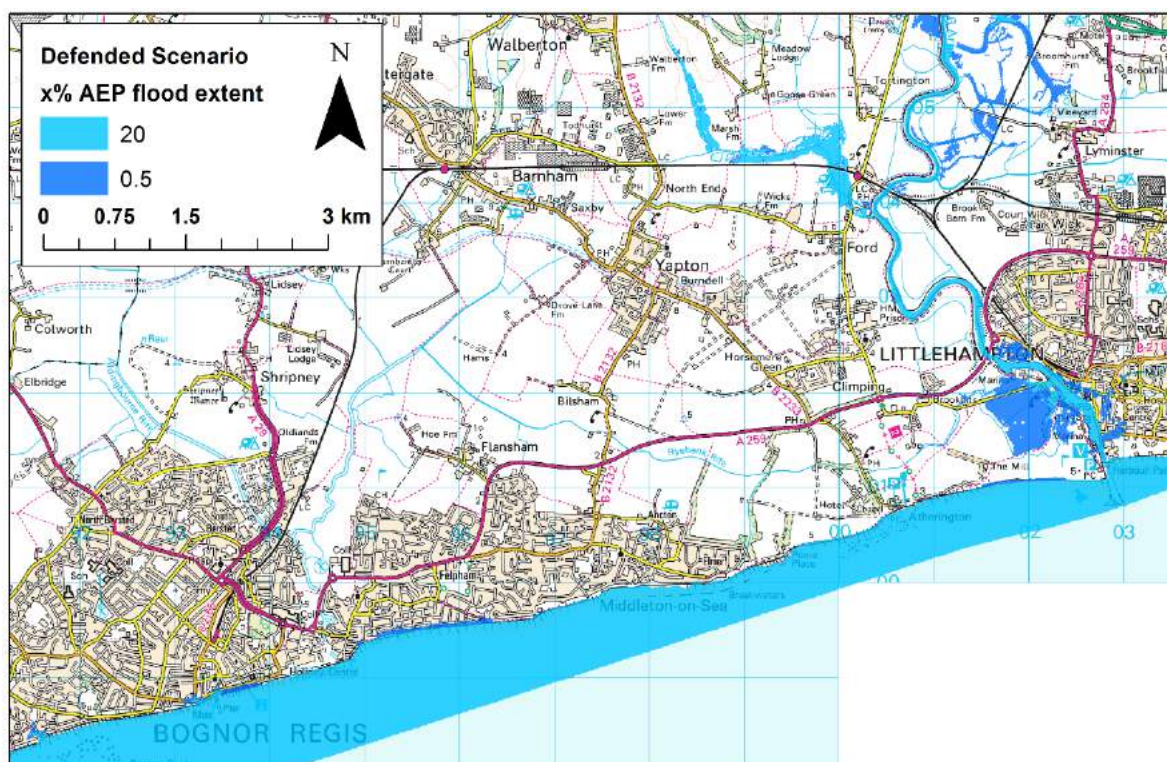
Figure 6-10: Sensitivity to downstream boundary for the Arun to East Head model

7 Results

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

7.1 River Arun to East Head model

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



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

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

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



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

7.1.1 Undefended model results

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

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

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

7.2 The impact of climate change

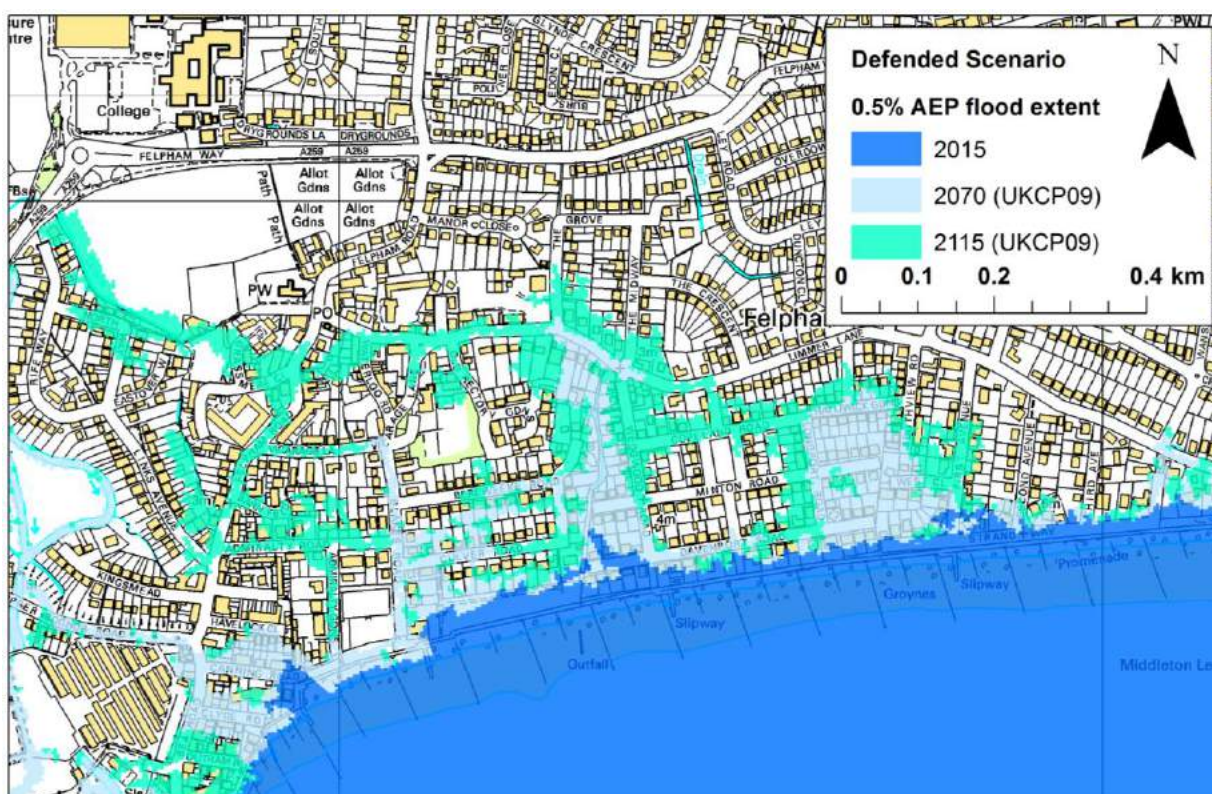
In July 2011, new climate change guidance was released based on the UKCP09 report¹⁵. In addition to present day extreme still water level events, 0.5% AEP climate change scenarios for the years 2070 and 2115 were also modelled based on the 95th percentile medium emissions scenario. The sea-level rise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

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

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

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

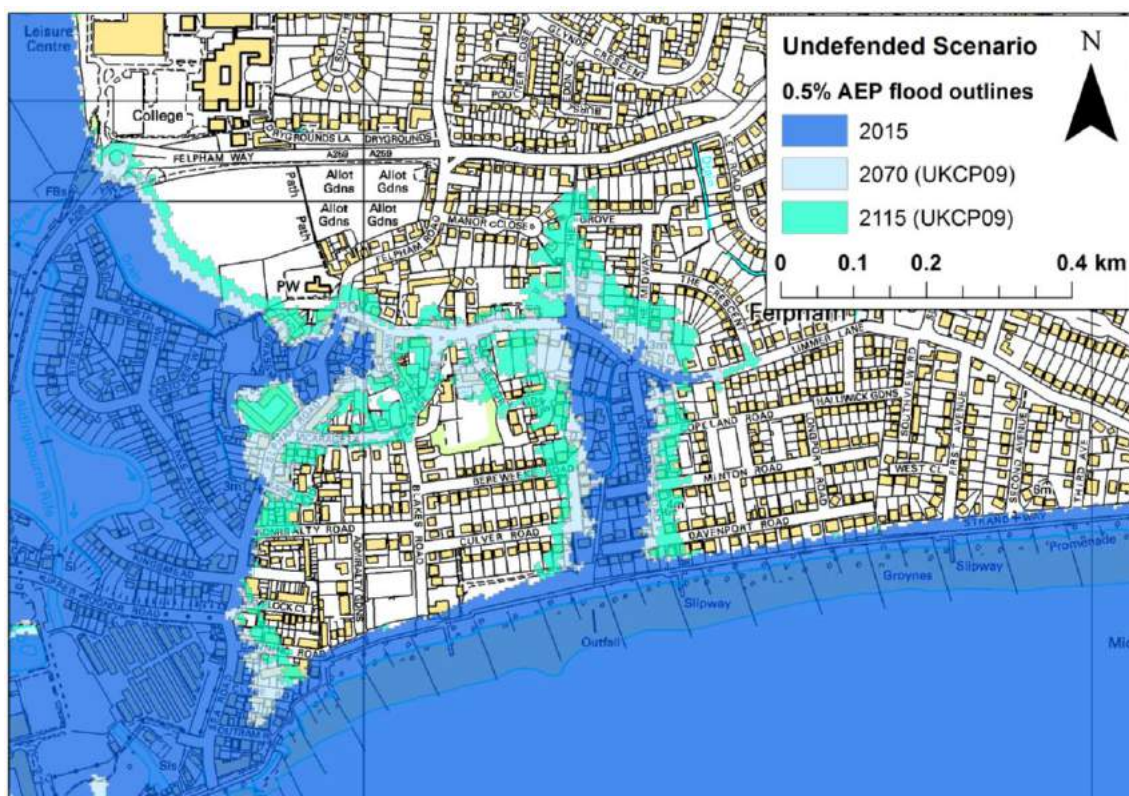
¹⁵ <http://ukclimateprojections.defra.gov.uk/>
2014s0753_Emsworth to Littlehampton_Coastal final report_v2.docx



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

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



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

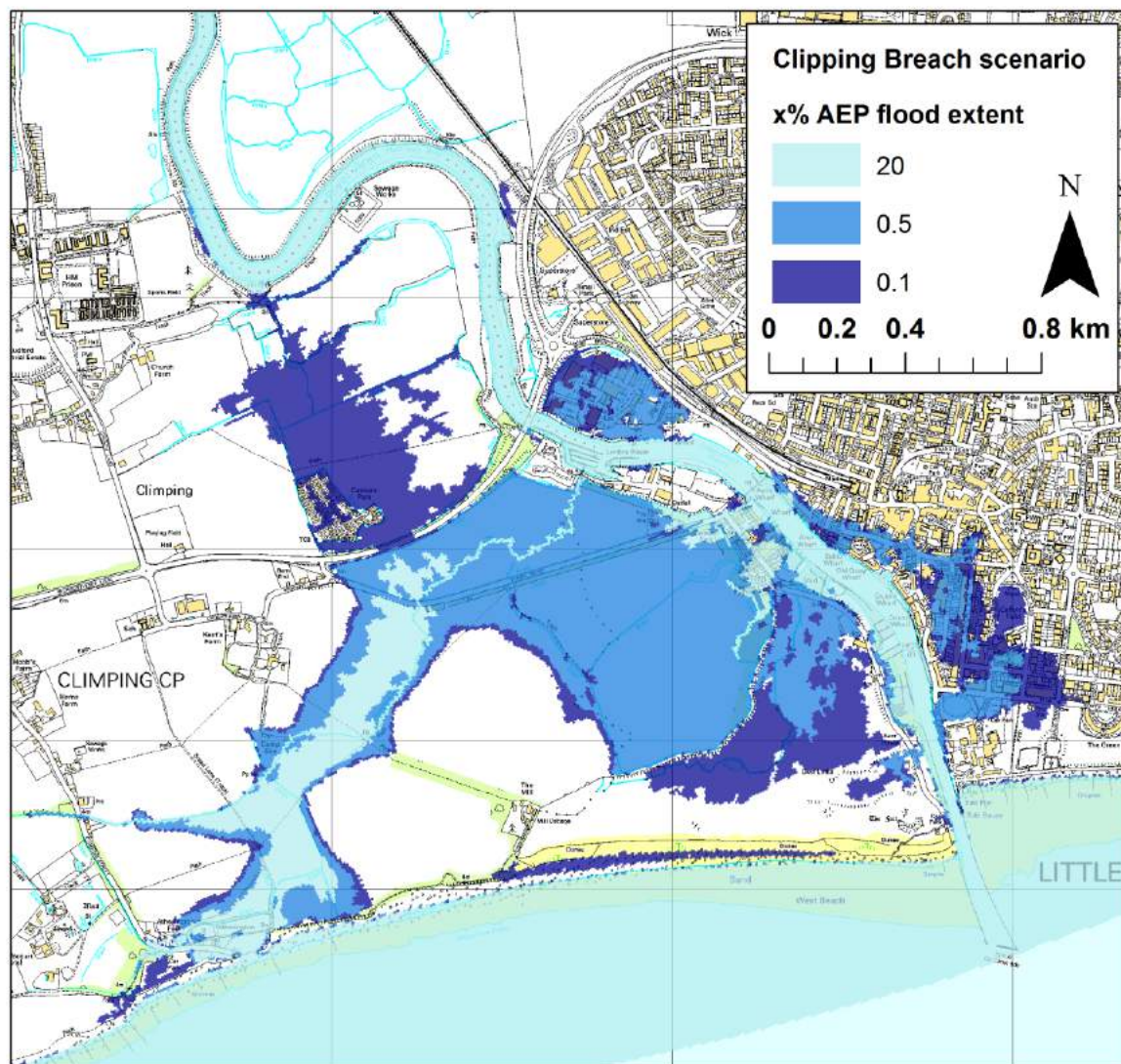
7.3 Breach scenarios

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

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

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

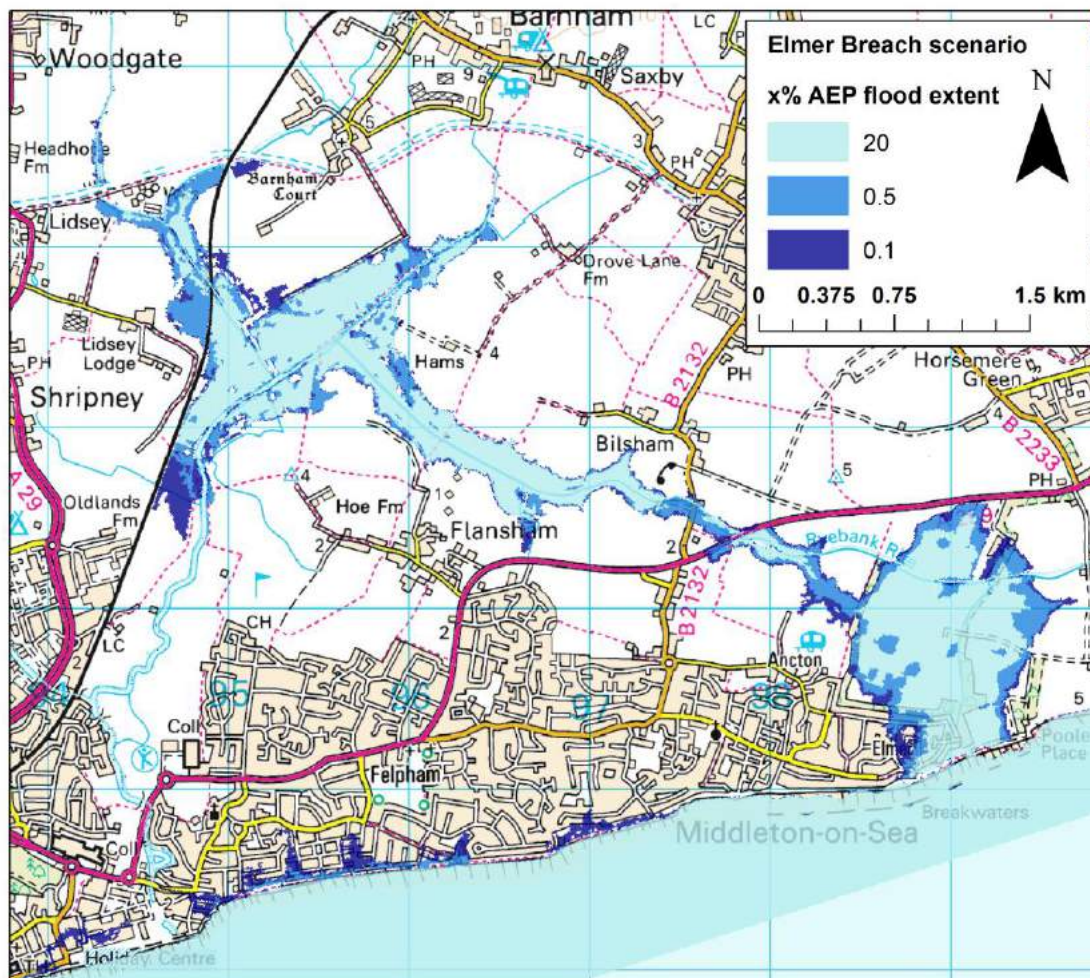
The breach at Climping was initially located on the western side of the car park but due to the high ground behind the defence this only resulted in limited flooding. The breach location was adjusted to tie into lower ground behind the defence near the entrance to the car park and this resulted in more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



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

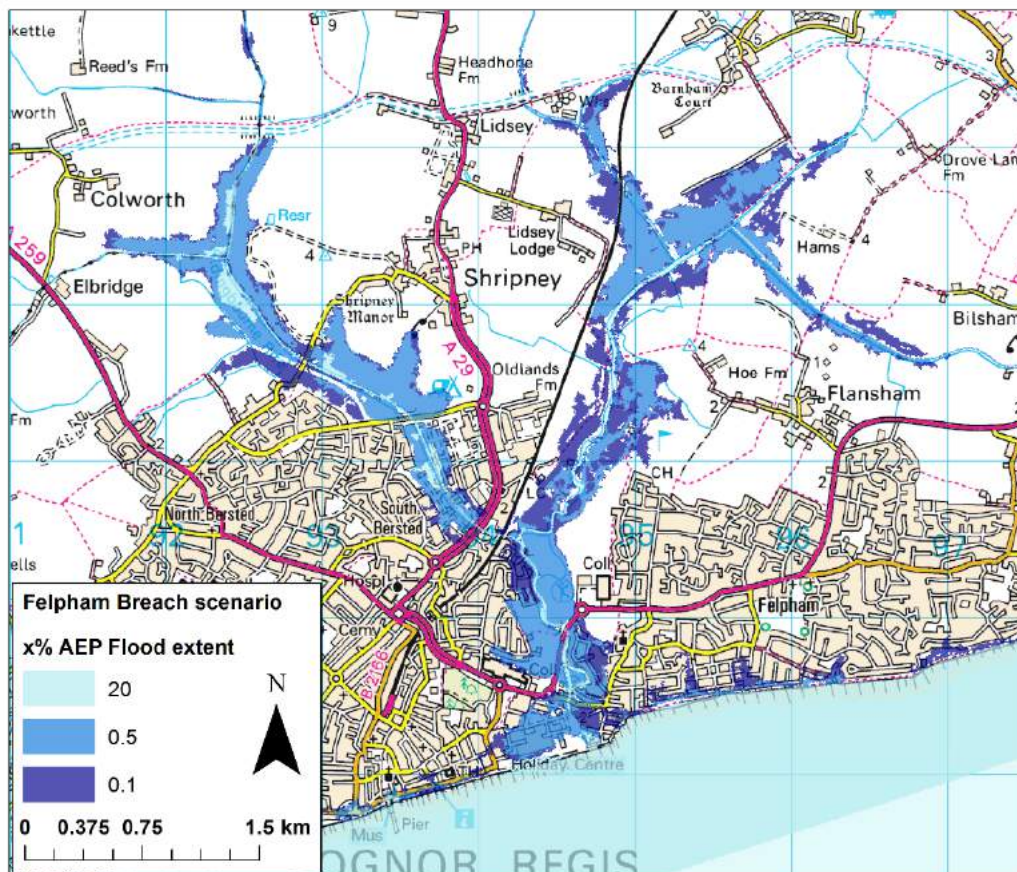
The breach at Elmer initially floods a large area behind the defence including over 100 properties. The flood water then spreads north-west along the channel of Ryebank Rife extending 6km inland during the 0.1% AEP AEP event. Figure 7-6 shows the breach extents for the 20, 0.5 and 0.1% AEP events at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



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

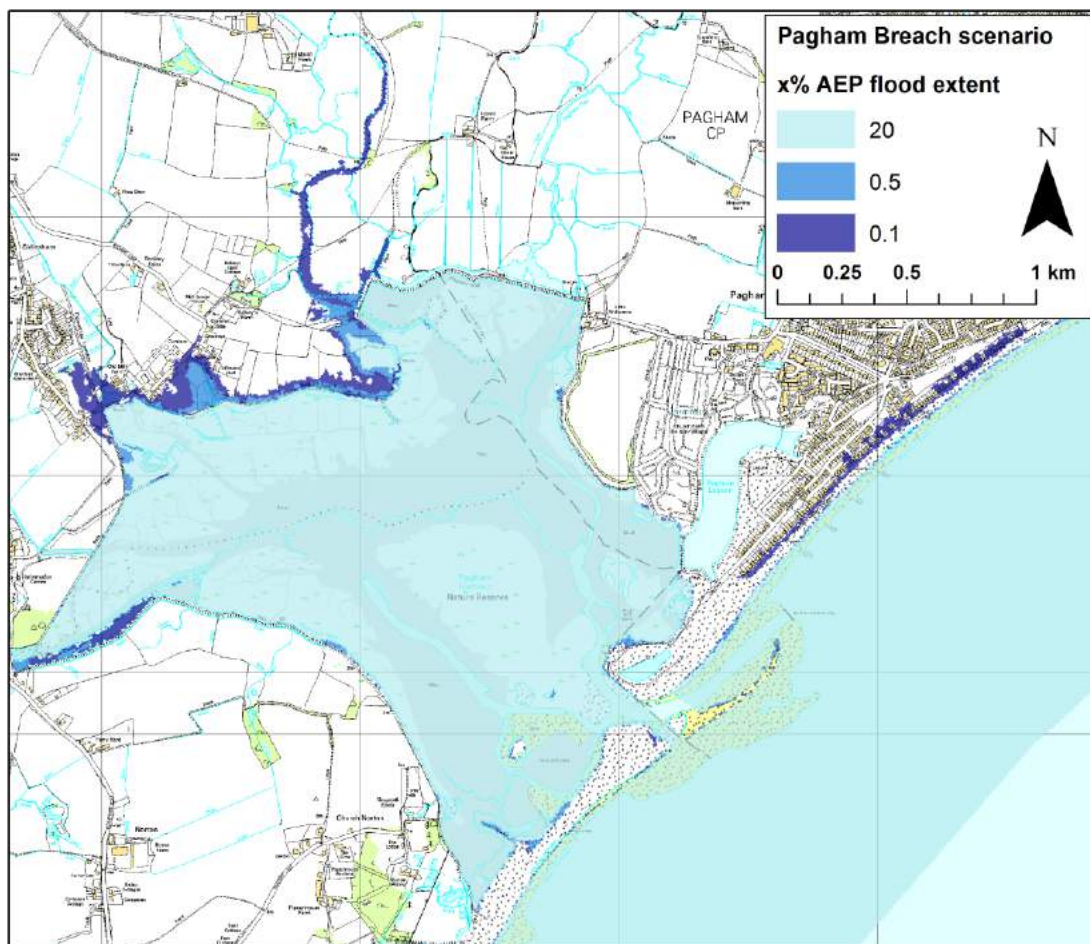
The breach at Felpham allowed water into the river channels of the Aldingbourne Rife and Lidsey Rife. Floodwater from the breach spreads several kilometres inland to the north-east and north-west within the river channels. Figure 7-7 shows the flood extents from the breach at Felpham for 20, 0.5, 0.1% AEP events.



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Figure 7-7: Flood extents from a breach at Felpham

The fourth breach was modelled at Pagham Harbour. A 40m breach was cut through the spit and the material was used to infill the existing channel to the east. The results from these simulations produced less flooding than the defended model simulations. This is in part due to the channel being blocked to the east and also due to the spit being removed from the harbour mouth to the end of the spit in the defended simulations, allowing a larger area for the water to flow through. The flood extents from the Pagham Harbour breach are shown in Figure 7-8.



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Figure 7-8: Flood extents from a breach at Pagham

7.4 Flooded properties

To provide an overview of the different model simulations, Table 7-1 below shows a comparison of the number of properties flooded during each defended and undefended simulation, including the 0.5% AEP events plus climate change for the years 2070 and 2115 based on both UKCP09 and NPPF guidance. These property counts were based on the National Receptors Database (NRD), which was edited to remove all properties which were listed in Appendix A of the 'Property Count Method - External Technical Note' as they were not required. There are clear differences between the defended and undefended simulations for all AEP events. For the 20% AEP defended event 222 properties are flooded compared with 2,565 during the undefended scenario. For the 0.5% AEP event, 968 properties flood in the defended simulation, most of which are on the River Arun and around Pagham Harbour. This count does however include some properties with Multi-Coloured Manual (MCM) code '999' and multiple points representing both ground floor and upper level flats. Thus the results are most appropriate for comparing the differences between different events and scenarios.

Table 7-1: Arun model flooded property count for all AEP events

Number of flooded properties			
AEP (%)	Flooded properties (Defended)	Flooded properties (Undefended)	Difference
20	222	2,565	2,343
5	378	3,011	2,633
3.33	415	3,092	2,677
1.33	584	3,314	2,730
1	640	3,397	2,757
0.5	968	3,575	2,607
0.2	1,562	3,880	2,318
0.1	2,199	4,144	1,945
0.5cc (2070 UKCP09)	3,006	4,711	1,705
0.5cc (2115 UKCP09)	5,115	5,972	857
0.5cc (2070NPPF)	3,537	5,077	1,540
0.5cc (2115 NPPF)	7,090	7,249	159

7.5 Onset and duration

Onset and duration of flooding grids were extracted from the 0.5% AEP model event simulation which detail the duration of inundation (duration grid) and the time of first inundation above a depth of 0.01m (onset grid).

7.6 Standard of protection

The SoP was calculated for all the coastal defences which overtopped during any of the model event simulations, excluding the climate change scenarios. Therefore, 32 SoP plots were created. The SoP was estimated based on various EurOtop wave overtopping tolerable limits. The tolerable discharge limits were obtained from the EurOtop Manual for various hazard types including trained staff, aware pedestrians, unaware pedestrians, fast/moderate moving vehicles and slow moving vehicles. Table 7-2 shows the SoP for the overtopped defences. For all overtopped defences, the SoP for slow moving vehicles is greater than 0.1% AEPs. Half of the defences also have a SoP for trained staff of more than 0.1% AEPs.

Wave overtopping profiles 38, 39 and 42 have a much lower SoP, with a 10% AEP standard of protection for 38 and 39 and less than 20% AEP SoP for trained staff at profile 42. These correspond to the areas that flooded during the 20% AEP event shown in Figure 13-1 at Bognor Regis pier and Aldwick. Wave overtopping profiles 24 and 46 have the highest SoP for overtopped defences in the region, with a 0.1% AEP SoP for all hazard types except fast moving vehicles, which reduces the SoP to 0.14% and 0.15% AEP respectively. All of the defences not included in Table 13-2 were not overtopped for the 20, 5, 3.33, 1.33, 1, 0.5, 0.2 or 0.1% AEP events and thus have a SoP greater than the 0.1% AEP event.

Table 7-2: Standard of protection for the 32 overtopped defences from Arun to East Head given as AEP (%)

WO profile	Threshold for fast moving vehicle 0.00001m ³ /s/m (SoP in AEP %)	Threshold for unaware pedestrian 0.00003m ³ /s/m (SoP in AEP %)	Threshold for aware pedestrian 0.0001m ³ /s/m (SoP in AEP %)	Threshold for trained staff 0.001m ³ /s/m (SoP in AEP %)	Threshold for slow moving vehicle 0.01m ³ /s/m (SoP in AEP %)
2	0.28	0.20	0.19	>0.1	>0.1
4	0.20	0.18	0.14	>0.1	>0.1
5	0.22	0.20	0.18	>0.1	>0.1
6	0.83	0.38	0.23	>0.1	>0.1
13	0.20	0.18	0.15	>0.1	>0.1
14	0.45	0.38	0.22	>0.1	>0.1

WO profile	Threshold for fast moving vehicle 0.00001m ³ /s/m (SoP in AEP %)	Threshold for unaware pedestrian 0.00003m ³ /s/m (SoP in AEP %)	Threshold for aware pedestrian 0.0001m ³ /s/m (SoP in AEP %)	Threshold for trained staff 0.001m ³ /s/m (SoP in AEP %)	Threshold for slow moving vehicle 0.01m ³ /s/m (SoP in AEP %)
15	0.48	0.45	0.38	0.12	>0.1
16	0.43	0.33	0.19	>0.1	>0.1
18	0.91	0.80	0.59	0.19	>0.1
19	0.91	0.80	0.59	0.19	>0.1
20	0.91	0.71	0.48	0.17	>0.1
21	0.91	0.71	0.48	0.17	>0.1
22	0.91	0.71	0.48	0.17	>0.1
23	0.20	0.19	0.17	>0.1	>0.1
24	0.14	>0.1	>0.1	>0.1	>0.1
25	0.71	0.50	0.43	0.15	>0.1
26	0.19	0.18	0.14	>0.1	>0.1
33	0.48	0.42	0.32	0.10	>0.1
34	1.18	0.91	0.77	0.23	>0.1
35	0.91	0.77	0.53	0.14	>0.1
36	0.91	0.77	0.53	0.17	>0.1
37	0.91	0.77	0.54	0.17	>0.1
38	<20	<20	<20	10.00	>0.1
39	<20	<20	<20	10.00	>0.1
42	<20	<20	<20	<20	>0.1
46	0.14	>0.1	>0.1	>0.1	>0.1
50	0.19	0.18	0.15	>0.1	>0.1
51	0.48	0.40	0.26	>0.1	>0.1
52	0.20	0.19	0.15	>0.1	>0.1
53	0.77	0.34	0.19	>0.1	>0.1
54	1.00	0.91	0.71	0.17	>0.1
55	3.03	2.50	1.43	0.40	>0.1

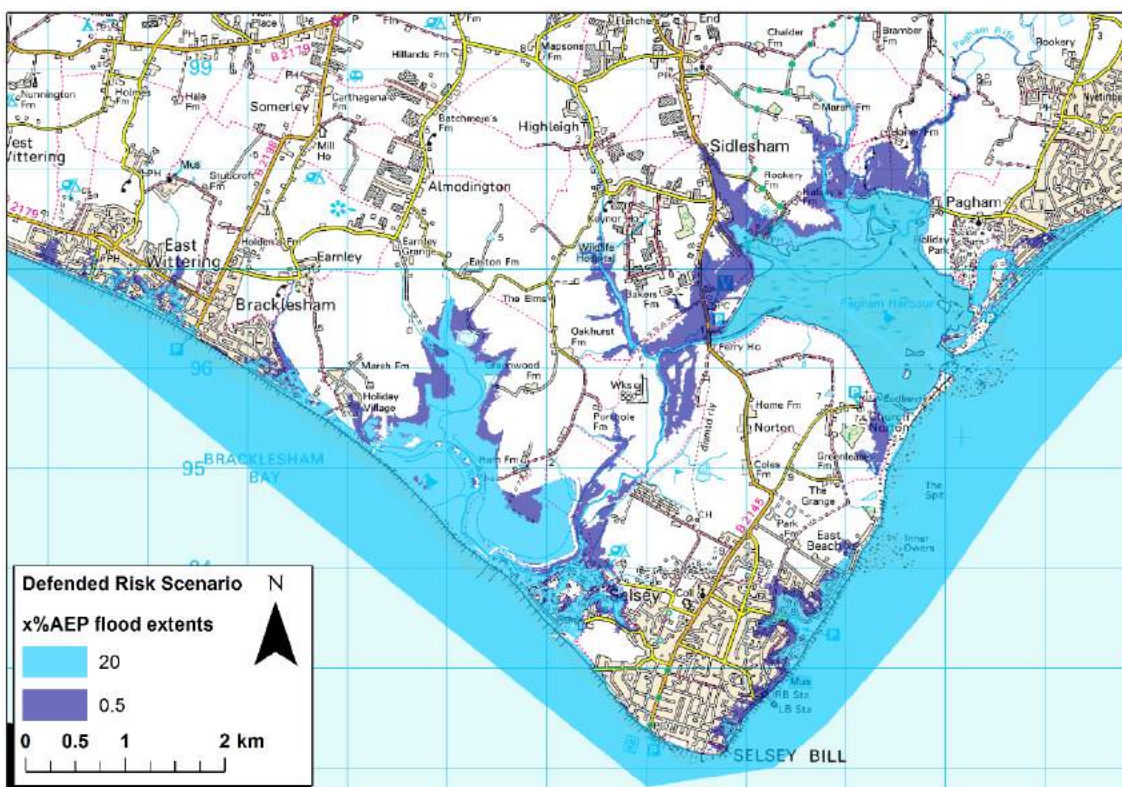
7.7 Risk model simulations

Wave overtopping calculations were completed for all joint probability scenarios for normal and risk profiles. The risk profiles were calculated for each coastal defence by analysing the CCO data for each area. The surveyed profile data was plotted and the worst case for each defence schematised in order to calculate the potential overtopping when the beach is in an eroded state. The model simulations were set up in exactly the same way as the defended models, but with the discharge rates from the risk profiles. Figure 7-9 and Figure 7-10 show the 20 and 0.5% AEP flood extents for the risk profile simulations at Bognor Regis, Littlehampton, Selsey and Wittering. When compared with the normal profile defended results for the 20 and 0.5% AEP flood extents, there is a distinctive increase in flooding, most notably around the river networks and coastal communities. There are significant increases around Bognor Regis, even during the 20% AEP event. Likewise, there is additional inundation of properties in the key communities of Selsey and Wittering during the 20% AEP event when using risk wave overtopping profiles.



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Figure 7-9: 20 and 0.5% AEP flood extents for Littlehampton and Bognor Regis using risk profiles



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Figure 7-10: 20 and 0.5% AEP flood extents for Selsey and Wittering using risk profiles

7.7.1 The impact of climate change

As with the defended and undefended scenarios, the risk profile simulations included assessing the impact of climate change, using both UKCP09 and NPPF guidance. Similarly to the lower AEP events, the risk profiles cause significant increases in flood extents compared with the defended climate change simulations. There is increased flooding at Wittering, Selsey, Bognor Regis, Felpham and Middleton-on-Sea. East Wittering in particular is almost entirely inundated during the 0.5% AEP plus climate change (2070 UKCP09) event, whereas almost no overtopping occurred during the defended equivalent.

7.7.2 Risk profile flooded properties

For the risk profile scenarios, there is increased property inundation for all model simulations when compared against the defended results. During higher AEP events, more properties flood than during the undefended scenarios. With the undefended simulations, defences are removed, allowing water to flow back out to sea in some coastal communities and disperse on the floodplains.

7.8 Chichester Harbour model

This model extended from West Wittering to Eastney on Portsea Island. This incorporates the communities of Itchenor, Bosham, Emsworth and Hayling Island. Wave overtopping does not have a significant impact on this harbour area, with flooding mainly caused by still water flooding. Several areas were schematised to include wave overtopping, including West Wittering and Emsworth. The model was built using information from six existing models in Chichester Harbour, extended west to Portsea Island and included updated bathymetry and DTM data. As with the Arun to East Head model, defended and undefended models were set up for the 20, 5, 3.33, 1.33, 1, 0.5, 0.5 (2070 UKCP09), 0.5 (2115 UKCP09), 0.5 (2070 NPPF), 0.5 (2115 NPPF), 0.2 and 0.1% AEP events. Risk profiles were not calculated for the area, as wave overtopping is limited in the harbour, with most flooding occurring as a result of still water inundation. The boundary conditions were updated and included new sea-level tidal graphs, with climate change increase calculated as +0.39m to 2070 and +0.78m to 2115 using UKCP09 guidance and +0.52m and +1.13m to 2070 and 2115 respectively using NPPF guidance methods.

Figure 7-11 illustrates the flood extents for the 20 and 0.5% AEP defended simulations in Chichester Harbour. In the harbour there is a noticeable increase in flooding during the 0.5% AEP event. The main differences are on Thorney Island and the land south of Fishbourne. Although there is increased flooding on Thorney Island, it does not extend as far as the urbanised areas, and instead mostly inundates fields.



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Figure 7-11: Chichester Harbour flood extents for the 20 and 0.5% AEP defended scenarios

The coastal community of Bosham is affected during the 20% AEP defended scenario, particularly near Bosham Quay and the properties along Shore Road. The 0.2% AEP event causes significantly more flooding in Bosham, extending inland to Taylor's Lane and Canute Road. Thorney Island is flooded during the 0.2% AEP event, resulting in flooding at Baker Barracks and an electrical sub-station. In Emsworth, properties surrounding Slipper Mill Pond and the A529 Hermitage Road Bridge are at risk of flooding during a 20% AEP event. During the 1.33% AEP event properties on Marina Close and King Street are affected. With increasing AEP, the flooded properties in Emsworth increase, most notably during the 0.2% AEP event when Bridge Road is flooded, north of the A259. The 0.1% AEP event floods to a similar extent in Emsworth, extending approximately 20m further inland to the east along Lumley Road.

In the region of 100 properties are flooded during the 0.2% AEP event to the east of Emsworth, with the majority of them overlooking the Mill Pond and Hermitage Bridge. To the west of Emsworth, over 60 properties are flooded during the same event, with most of the inundated properties situated on Bath Road.

The undefended events show increased flooding for all AEP events. During the 0.1% AEP event all areas of Bosham along the Channel are flooded and Canute Road, Harbour Road and Southwood Farm are inundated. The 20% AEP undefended event resulted in increased flooding in Emsworth, both west and east of Mill Pond near Hermitage Bridge, affecting properties on Queen Street, King Street and Slipper Road. Figure 7-12 shows the difference in flooding between the 20% AEP defended and undefended simulations. The main differences are on Thorney Island and along the eastern side of Hayling Island. With each increasing AEP, further properties are flooded up to the 0.1% AEP event when the A259 is affected and additional properties are flooded on Lumney Road, Pagham Close, Bath Road, Bridge Road and Bridgefoot Path.



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Figure 7-12: 20% AEP defended and undefended flood extents for Chichester Harbour

Figure 7-13 shows the 0.1% AEP flood extents for both the defended and undefended simulations. There are fewer differences between these two than during the 20% AEP events, as more low lying areas are completely inundated. The most noticeable difference is to the east of Chichester Marina, which is still protected in the defended simulation. Towards the north of the model, Southbourne also continues to remain protected during the defended event but not for the undefended scenario. The extents at Emsworth are the same for both simulations.



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Figure 7-13: 0.1% AEP defended and undefended flood extents for Chichester Harbour

The impact of climate change

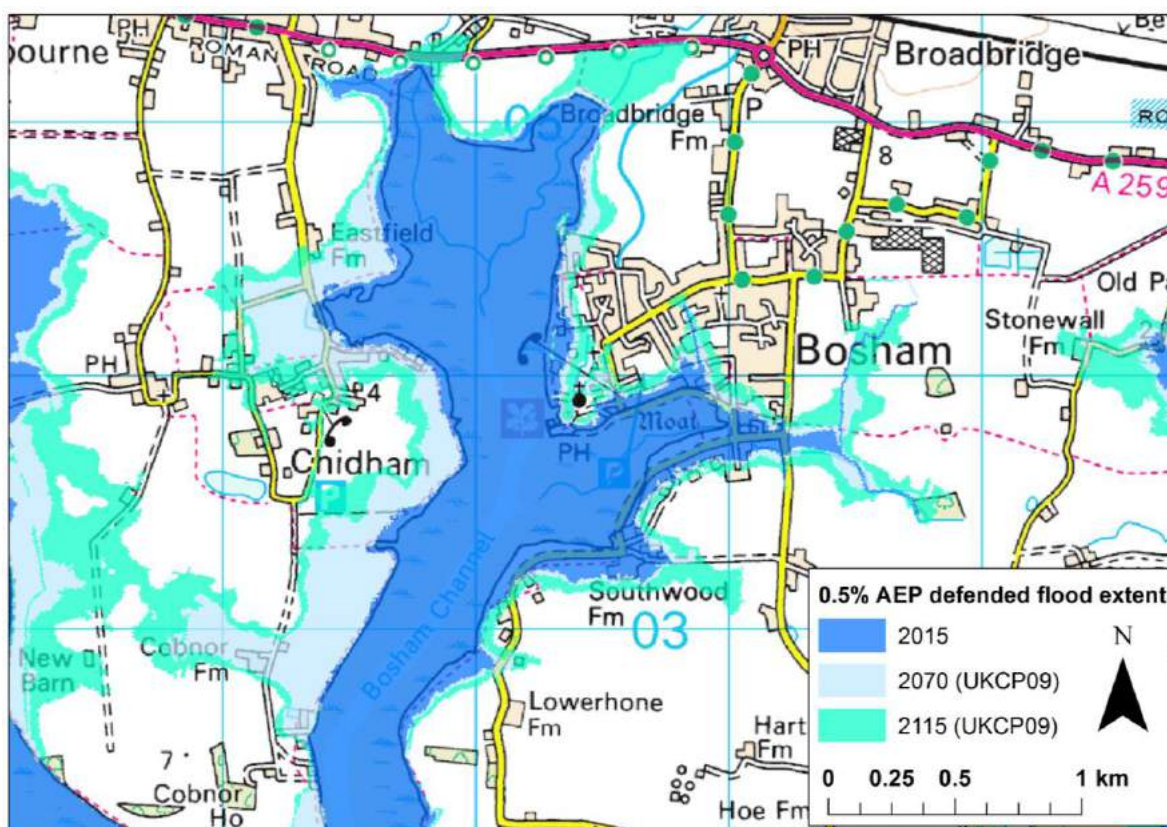
Figure 7-10 shows the flood extents in Chichester Harbour for the 0.5% AEP event and the 0.5% AEP events plus climate change to 2070 and 2115 based on the UKCP09 sea-level rise estimates. For both climate change simulations, there is increased flooding in Emsworth, Southbourne, Bosham, West Wittering and on Hayling and Portsea Islands. On Portsea Island, Eastney is affected by the 2115 climate change event, which extends 1km further inland to the west than the 0.5% AEP event.

In Emsworth there are fewer differences as a result of climate change, with additional flooding compared with the 0.5% AEP event mainly limited to the west, around Bridge Road. In Bosham, further properties are flooded during both climate change scenarios, although most of the additional flooding occurs only during the 2115 UKCP09 event. The additional flooding includes the area around the boat and car park, as well as Bosham Lane, Harbour Road and The Drive. Figure 7.14 illustrates the differences between the three events at Bosham. West Wittering remains defended during both the 0.5% and 0.5% AEP plus climate change to 2070 events. During the 0.5% AEP plus climate change to 2115 event, widespread flooding occurs affecting the Roman Landing area of West Wittering, including the church, National Trust site, school and properties around Rookwood Road. During the event, Rookwood Road has depths of up to 1.00m whilst the event flooded the nearby school to approximately 0.50m.



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Figure 7-14: 0.5% AEP defended and defended UKCP09 climate change comparison for the Chichester model



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Figure 7-15: 0.5% AEP defended and defended UKCP09 climate change comparison at Bosham

For both 2070 and 2115, climate change for the Chichester model was also calculated based on NPPF guidance. An increase of 0.52m and 1.13m was calculated for 2070 and 2115 respectively, the same increase as calculated for the Arun to East Head model. As would be expected, the increased still water level results in more widespread flooding based on NPPF guidance. Much of the coastline is affected, particularly during the 2115 NPPF event. During the 2070 NPPF event, flood extents are similar to the 2115 UKCP09 event. The 2115 NPPF event causes the largest amount of flooding. Emsworth and Bosham experience additional flooding, with Bosham the worst affected of the two, including depths of more than 0.60m on Bosham Lane.

Flooded properties

As with the Arun to East Head model, the flooded properties were calculated for the Chichester Harbour model to provide an overview of the different model simulations. Table 7-3 shows a comparison of the number of properties flooded during each defended and undefended simulation, including the 0.5% AEP events plus climate change for the years 2070 and 2115 based on both UKCP09 and NPPF guidance. These property counts were based on the same NRD as the Arun model, edited to remove all properties which were listed in Appendix A of the 'Property Count Method - External Technical Note'. There are clear differences between the defended and undefended simulations for all AEP events. For the 20% AEP defended event 375 properties are flooded compared with 2,195 during the undefended scenario. For the 0.5% AEP event, 1,581 properties flood in the defended simulation, most of which are around South Hayling, Emsworth and Bosham. The 0.2% AEP event causes 2,703 properties to flood in Chichester Harbour, which increases by 700 during the 0.1% AEP event to 3,438 properties. During the largest event, the 0.5% AEP event plus climate change (2115 NPPF), there is a smaller difference between the defended and undefended simulations, with only 2,045 more properties flooding during the undefended simulation. This count does however include some properties with MCM code '999' (84 properties in the 20% AEP extent). Thus the results are most appropriate for comparing the differences between different events and scenarios. When calculating the first, tenth and one hundredth properties to flood, which is discussed in the Chapter 8, this NRD was edited to remove such properties from the count.

Table 7-3: Chichester Harbour model flooded property count for all AEP events

Number of flooded properties			
AEP (%)	Flooded properties (Defended)	Flooded properties (Undefended)	Difference
20	375	2,195	1,820
5	894	3,323	2,429
3.33	1,001	3,620	2,619
1.33	1,331	4,390	3,059
1	1,415	4,582	3,167
0.5	1,581	5,116	3,535
0.2	2,703	5,823	3,120
0.1	3,438	6,478	3,040
0.5cc (2070 UKCP09)	4,823	8,345	3,522
0.5cc (2115 UKCP09)	9,190	12,647	3,457
0.5cc (2070NPPF)	6,636	9,800	3,164
0.5cc (2115 NPPF)	16,663	18,708	2,045

7.8.1 Onset and duration

Onset and duration of flooding grids were extracted from the 0.5% AEP model event simulation which detail the duration of inundation (duration grid) and the time of first inundation above a depth of 0.01m (onset grid) for Chichester Harbour.

7.8.2 Standard of protection

The SoP was to be calculated for all the coastal defences which overtopped during any of the model event simulations, excluding the climate change scenarios. Only four defences were schematised for this model, due to the sheltered nature of the harbour. Of the four defences, three

were at the eastern extent of the model near West Wittering and the other at Emsworth. All four defences were only overtopped during the climate change simulations and as such, SoP was not calculated. Therefore they all have a SoP greater than the 0.1% AEP event.

8 Flood warning updates

8.1 Update to existing Flood Alert and Flood Warnings Areas

New Flood Alert and Flood Warning areas were produced using the modelled flood extents. These were created for cell based areas and for individual community based areas.

8.1.1 Cell based Flood Alert and Flood Warning Areas

For the Arun model domain, the cell based Flood Alert and Flood Warning areas were based on the same six stretches of coastline that were covered in the existing Flood Warning areas. The six areas and their unique name codes are shown in Table 8-1. Similarly, for the Chichester Harbour model, three cell areas were used which are given in Table 8-2.

The Flood Alert Area was composed of the Flood Zone 2 outline. The Flood Warning Area A was composed of the 0.5% AEP defended flood extent and the Flood Warning Area B was composed of the area between the 0.5% AEP defended and Flood Zone 2 extents.

Table 8-1: Arun to East Head cell based Flood Alert codes and coverage

Flood Alert Code	Name
065WAF424	Lower Arun
065WAC405	Littlehampton Rope Walk
065WAC404	Climping
065WAC403	Selsey Bill to Elmer
065WAC402	Medmerry
065WAC401	Thorney Island to Bracklesham

Table 8-2: Chichester Harbour model cell based Flood Alert codes and coverage

Flood Alert Code	Name
065WAC401	Thorney Island to Bracklesham
065WAC155	Hayling Island
065WAC162	Langstone and Emsworth Harbour

8.1.2 Community based

Individual community based Flood Alert and Flood Warning areas were created to allow for more targeted Flood Alert and Flood Warnings along the coast. The format of these was the same as for the cell based output but clipped for specific communities rather than stretches of coastline. The communities that the targeted Flood Alert and Flood Warning areas were created for can be seen in Table 8-3 for the Arun model and Table 8-4 for the Chichester Harbour model. An additional Flood Warning area has also been recommended for East Wittering, which did not previously exist and therefore has no code. This is not in Table 8-3 as no code has been assigned.

Table 8-3: Arun to East Head community based Flood Alert and Flood Warning locations

Community	Flood Alert Area Code	Flood Warning, Area A Code	Flood Warning, Area B Code
Ford and Climping on the Arun	065FWF5006	065FWF5006A	065FWF5006B
Littlehampton Rope Walk	065FWC2601	065FWC2601A	065FWC2601B
Climping	065FWC2501	065FWC2501A	065FWC2501B
Elmer	065FWC2402	065FWC2402A	065FWC2402B
Bognor	065FWC2401	065FWC2401A	065FWC2401B
Pagham	065FWC2301	065FWC2301A	065FWC2301B
Sidlesham	065FWC2302	065FWC2302A	065FWC2302B
Selsey	065FWC2201	065FWC2201A	065FWC2201B
Medmerry	065FWC2101	065FWC2101A	065FWC2101B
Bracklesham	065FWC2002	n/a	065FWC2002B

Table 8-4: Chichester Harbour model community based Flood Alert and Flood Warning locations

Community	Flood Alert Area Code	Flood Warning, Area A Code	Flood Warning, Area B Code
West Wittering	065FWC2001	n/a	065FWC2001B
Bosham and West Itchenor	065FWC1901	065FWC1901A	065FWC1901B
Thorney Island, Southbourne and Nutbourne	065FWC1801	065FWC1801A	065FWC1801B
Eastoke Seafront and South Hayling	065FWC1602	065FWC1602A	065FWC1602B
Eastoke	065FWC1601	065FWC1601A	065FWC1601B
Northney to Selsmore, Hayling Island	065FWC1501	065FWC1501A	065FWC1501B
Langstone and Emsworth	065FWC1401	065FWC1401A	065FWC1401B

West Wittering did not have a Flood Warning Area A whilst Flood Warning Area B extends around Roman Landing and Rookwood Road. Where not necessary, small polygons were removed to create the final area A and B Flood Warnings. All polygons under 200m² were analysed to decide whether the area should be included in the warning areas based on location and the NRD.

8.2 Flooded properties

For the 0.5% AEP event, onset and duration grids were created and these were used to calculate the first, 10th and 100th property to flood. The first item of critical infrastructure to flood was also determined. The NRD was used to determine which properties flooded, together with the OS MasterMap data. Several modifications were made to the NRD. The first step was to reduce the size of the NRD by buffering the 0.1% AEP modelled flood extent and extracting the NRD points within it. This dataset was filtered to remove property types which were not required for analysis as recommended in Appendix A of the Environment Agency's 'Property Count Method - External Technical Note'. The filtered NRD was then compared against the OS MasterMap dataset and a series of manual edits were made to improve the way flooded properties would be calculated. This was completed separately for both models. The following sections discuss the methods and results for each model.

Arun model

The manual edits that were completed for the Arun model included:

- Removing upper level flats and accommodation from the dataset in areas where residential areas were above shops and other buildings.
- Editing areas on the River Arun. This included removing the Golf Club point from the flooded property count, leaving the Yacht Club. Both shared the same topographic ID in the NRD, but the Golf Club is a separate property, on the other side of the road and does not flood.
- Removing Bognor Regis Leisure Limited from the property count, which is the property on the pier. The onset grid suggested that this area was flooded after 5.5 hours from still water flooding, but this is without the pier being read into the model. Detailed analysis of the elevation in the area shows the road level to be 6.00mAOD, whereas the maximum SWL for the 0.5% AEP event was 4.04m. Wave overtopping was not present in the area until approximately 17 hours into the model simulation and even then, this remained on the road.
- Deleting some of the properties with an MCM code of '999', where the points were not individual dwellings and actually part of the same large property. For example, one particular property was listed as five buildings, one of which had an MCM code of '1' and the other four were listed as '999'. The adjacent building was listed as its own property, yet was a garage.

The first, 10th and 100th properties to flood during the 0.5% AEP event were all due to still water flooding on the River Arun. The first property to flood, based on the filtered NRD was the Arun Yacht Club on the west bank of the river on Rope Walk. The onset of flooding was at 5.88 hours resulting in a water level threshold of 3.69m. This threshold is based on the SWL at Littlehampton for the onset time of flooding. The tenth property to flood was also on Rope Walk; a semi-detached bungalow, 5 Riverside, postcode BN17 5DE. Further information about the flooded properties are provided in Table 8-5. The onset of flooding for the 10th property was after 6.30 hours, 25 minutes after the first property flooded. The 100th property to flood was in Littlehampton, again on Rope Walk. The property was a terraced house, 8 Tudor View, postcode BN17 5DE. This property flooded during the 0.5% AEP event after 15.41 hours, nine and a half hours after the first property.

Table 8-5: First, 10th and 100th property and first critical infrastructure to flood in the Arun model

Property number	Property information	Onset of flooding (hrs)	Toid	Water level threshold (mAOD)	WO volume threshold (m3/s/m)
First	Arun Yacht Club, Rope Walk, Littlehampton, BN17 5DL	5.88	osgb1000008159097	3.69	N/A
Tenth	5 Riverside, Rope Walk, Littlehampton, BN17 5DE	6.30	osgb1000008159484	3.83	N/A
One hundredth	8 Tudor View, Rope Walk, Littlehampton, BN17 5DE	15.41	osgb1000008159469	3.84	N/A
First critical infrastructure	Sewage pumping station, Rope Walk, Littlehampton, BN17 5DQ	6.70	osgb1000001824159274	3.84	N/A

8.2.1 Chichester model

The manual edits that were completed for the Chichester Harbour model included:

- Deleting all upper level flats and apartments.
- Removing of NRD points from Chichester, Emsworth and Langstone Channels, which had an MCM code of '526', labelled as harbours. These were flooded at all durations and would have an impact on the first, tenth and one hundredth property to flood. However, several buildings were also represented by the code '526' and these were included, as they were boatyards.
- Removing a boat from the property count in the yacht building yard on Hayling Island which was situated in the quay.
- Checking the first one hundred properties manually on Google Maps and deciding whether to include as a property. This included removing a shed from the NRD which was located at the quay near the yacht building yard.

Additional properties were examined, particularly those with an MCM code of '999' and removed where deemed necessary.

The first, 10th and 100th properties to flood during the 0.5% AEP event were all due to still water flooding in different parts of the harbour. The first property to flood, based on the filtered NRD, was a detached property on Harbour Way, Emsworth, with postcode PO10 7BE. The onset of flooding was at 5.52 hours resulting in a water level threshold of 2.69m. This threshold is based on the SWL at Wittering for the onset time of flooding. As the time of flooding was in-between the two model timesteps, the level was calculated by interpolating between the water levels at five and a half and five and three quarter hours. The 10th property to flood was also on Harbour Way, Emsworth, but a terraced house, with the same postcode PO10 7BE. The onset of flooding for the tenth property was after 5.79 hours with a water level threshold of 2.92m. The 100th property to flood was to the south of Bosham. The property was a detached house on Shore Road, Chichester, postcode PO18 8QL, with the property name Mermaid Cottage. This property flooded during the 0.5% AEP event after 6.18 hours with a water level threshold of 3.17m.

Table 8-6: First, tenth, one hundredth property and first critical infrastructure to flood in the Chichester model

Property number	Property information	Onset of flooding (hrs)	Toid	Water level threshold (mAOD)	WO volume threshold (m3/s/m)
First	14 Harbour Way, Emsworth, PO10 7BE	5.52	osgb1000002119006760	2.69	N/A
Tenth	10 Harbour Way, Emsworth, PO10 7BE	5.79	osgb1000002119007027	2.92	N/A
One hundredth	Mermaid Cotage, Shore Road, Chichester, PO11 0NQ	6.18	osgb1000002119074670	3.17	N/A
First critical infrastructure	Electricity sub-station, Hayling Island, PO11 0NQ	6.10	osgb5000000001290839	3.14	N/A

8.3 Critical infrastructure

8.3.1 Arun model

There were 10 critical infrastructure properties that flooded during the 0.5% AEP event. Critical infrastructure was extracted from the NRD based on the criteria for critical services in Table 6.1 of the 'EA Annexes for PFRA Guidance'. This included schools, hospitals, police stations, nursing homes, prisons, sewage treatment works and electricity installations.

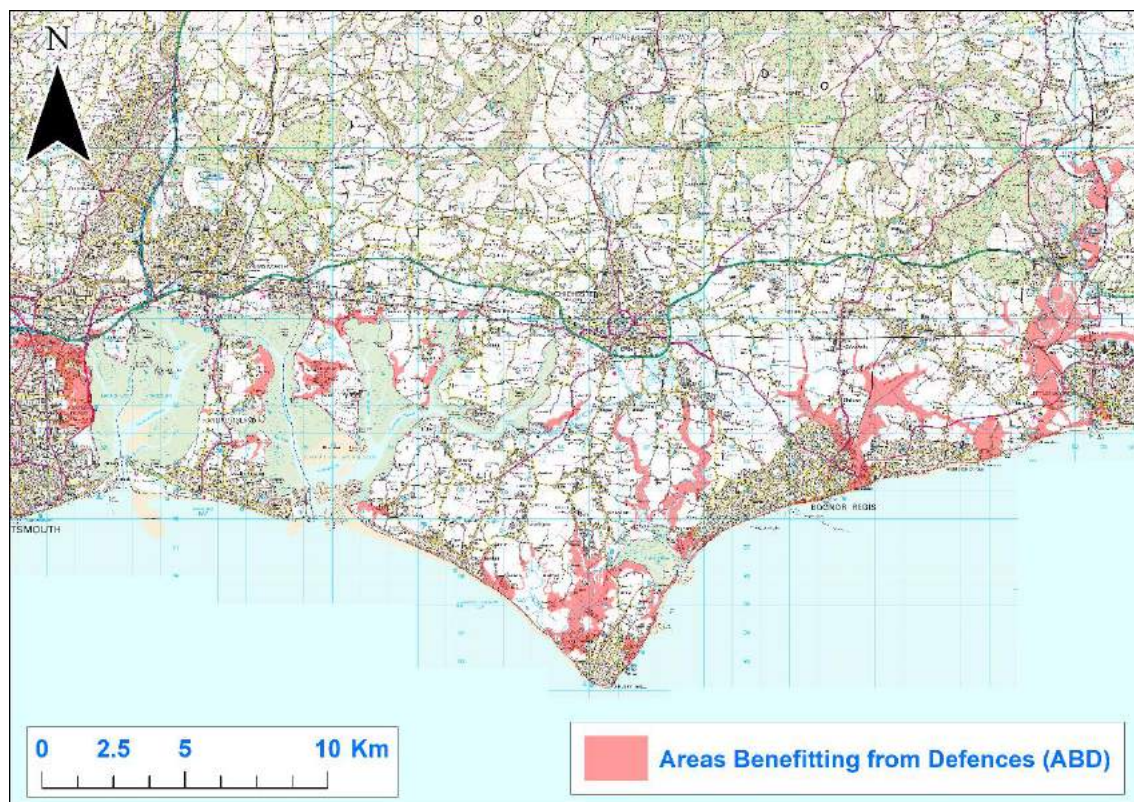
The critical infrastructure included six electricity sub-stations and four pumping stations. The sub-stations have an MCM code of '960' and the first sub-station to flood was the fourth critical infrastructure to flood, after the sewage pumping station and two additional pumping houses.

8.3.2 Chichester model

During the 0.5% AEP event in the Chichester model, much more critical infrastructure was affected, with 23 identified at flood risk. The first critical infrastructure to flood was an electricity sub-station on Hayling Island outside the Langstone Hotel, postcode PO11 0NQ. This sub-station was inundated after 6.10 hours, relating to a water level threshold of 3.14m at Wittering. Of the 23 critical services that were affected during the 0.5% AEP event, 19 of them were electricity sub-stations, two pumping stations, one education centre and a sewage treatment plant.

8.4 Areas benefitting from defences

ABDs were calculated by exporting the differences in area between the 0.5% AEP undefended and the 0.5% AEP defended modelled flood extents. In accordance with national guidance dry islands of less than 200m² were removed from the ABDs. In the Arun to East Head model domain the ABD totalled 27.19km², whilst the Chichester Harbour model ABD was 9.42km². To the east of the model domains, the main ABD is Climping and the nearby caravan park. Further west Elmer benefits from defences both on the coast and on the banks of Ryebank Rife. Central Bognor Regis also forms a significant area of the ABD, including Butlins Bognor Regis Resort, the University of Chichester and many properties either side of Aldingbourne Rife. Selsey benefits greatly from coastal defences, with the ABDs including much of East Selsey and both West Sands Caravan Park and Selsey Country Club. Sussex Beach Holiday Village also benefits from defences in the Arun model domain. The largest ABDs in the Chichester Harbour model are West Wittering, Thorney Island and East Hayling Island. Other areas benefitting from defences include the coastal frontages of Thornham, Southbourne and Nutbourne as well as small areas of West Itchenor and Bosham Hoe. Figure 8-1 shows a broad overview of both model domains with the areas benefitting from defences.



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Figure 8-1: ABDs in both Arun to East Head and Chichester Harbour model domains

9 Conclusions

9.1 Main findings

This project was commissioned by the Environment Agency to improve understanding of the area's exposure to coastal flood risk along the West Sussex coastline from Emsworth to Littlehampton's West Bank. The project focussed on the key communities of Littlehampton, Bognor Regis, Pagham, Selsey, Wittering, Bosham and Emsworth. The study used comprehensive extreme sea-level and wave information, improved quality topographic data and advanced inundation modelling techniques to produce revised tidal flood risk information, including Flood Zones 2 and 3. The revised Flood Zones were used to examine the potential impacts of climate change on flood risk, investigating flooding on a community based scale. Climate change simulations were completed using both NPPF and UKCP09 guidance, both to 2070 and 2115.

9.1.1 New flood map components

The new tidal Flood Zones derived for this study are similar to the previous Flood Zones, albeit slightly more extensive. The proposed Flood Warning thresholds are generally lower than the current Flood Alert and Warning thresholds, presumably due to the method of calculation using first property to flood rather than defence crest levels. In the Arun model, the proposed Flood Warning Area A includes significant inundation at Littlehampton West Bank from the River Arun as a result of still water flooding as well as coastal flooding at Bognor Regis and Pagham Harbour. Flood Warning Area B (the area between the 0.5% AEP defended flood extent and Flood Zone 2 extent) includes additional flooding at Climping, Elmer, Bognor Regis, Selsey and Bracklesham. Likewise in the Chichester Harbour model, the new Flood Zones are similar to the existing Flood Zones, with slight increases in extent for most regions. The most noticeable increases are at Birdham and on Thorney Island.

9.1.2 Impact of Climate Change

In addition to the present day flood simulations used to produce the revised Flood Zones for the study area, climate change simulations for the 0.5% AEP event were also carried out. These simulations indicate the expected increase in flood risk up to the year 2115 based on the latest UKCP09 and NPPF guidance on sea-level rise estimates. These simulations, run for the 0.5% AEP event, show that with the increases to still water level, the region and its current defence structures are vulnerable to flooding due to climate change. The defence network along the coast becomes heavily inundated as a result of increased wave overtopping of the current defences. In the Arun model, during the 0.5% AEP defended simulation, 968 properties were flooded, compared with 5,115 and 7,090 to 2115 based on UKCP09 and NPPF guidance respectively. Table 9-1 shows the number of flooded properties during climate change scenarios for both models compared against the 0.5% AEP property inundation.

Table 9-1: Property inundation for climate change simulations

Number of flooded properties		
AEP (%)	Flooded properties (Defended)	Model
0.5	968	Arun
0.5cc (2070 UKCP09)	3,006	Arun
0.5cc (2115 UKCP09)	5,115	Arun
0.5cc (2070NPPF)	3,537	Arun
0.5cc (2115 NPPF)	7,090	Arun
0.5	1,581	Chichester
0.5cc (2070 UKCP09)	4,823	Chichester
0.5cc (2115 UKCP09)	9,190	Chichester
0.5cc (2070NPPF)	6,636	Chichester
0.5cc (2115 NPPF)	16,663	Chichester

In the Chichester Harbour model 1,581 properties were flooded during the 0.5% AEP event, compared with 9,190 and 16,663 to 2115 based on UKCP09 and NPPF guidance respectively.

The climate change wave overtopping scenario suggests that the effects of climate change could make wave overtopping an increasing hazard, which threatens large areas of the coastal frontages around Bognor Regis, Selsey, Bracklesham and East Wittering in the Arun to East Head model and West Wittering, West Itchenor, Bosham, Emsworth, Thorney Island and Hayling Island in the Chichester Harbour model. Inland areas which are not affected by wave overtopping are shown to be at considerable risk of still water flooding when climate change sea-level rise is taken into account. The impacts of climate change on the West Sussex coastline is going to be an ever increasing hazard to both residential and commercial property and infrastructure.

9.1.3 Impact on critical infrastructure

As part of the study, the first critical infrastructure to flood for both Arun and Chichester models were identified. The first critical infrastructure to flood in the Arun to East Head model domain was a sewage pumping station, with a water level threshold of 3.84m relating to the water level at Littlehampton gauge. In the Chichester Harbour model, the first critical infrastructure to flood was also an electricity sub-station, on Hayling Island with postcode PO11 0NQ. The water level threshold for this sub-station was 3.14m which related to Wittering tidal curve.

9.1.4 Impact on coastal communities

In the Arun to East Head model domain, the major flooding during the defended scenarios occurs at Littlehampton West Bank, Felpham, Bognor Regis, Pagham and Selsey. Littlehampton West Bank was inundated by still water flooding from the 20% AEP defended event, whereas other coastal communities were generally not affected by overtopping until a 1% AEP event or greater.

Felpham coastal communities were inundated during the 1% AEP event, whilst a small area near Bognor Regis pier was flooded during the 5% AEP event. However, excluding this approximately 300m breach, Bognor Regis remains defended throughout all AEP events up to the 0.1% AEP scenario. Aldwick experienced minor flooding due to a breach of the sea wall at Dark Lane which is maintained by the Local Authority. Properties were affected by flooding around Pagham including Church Farm Holiday Village and the Mill Lane area to the west of the harbour. Excluding an area at Selsey Bill, Selsey coastal defence network mitigated the risk up to the 0.1% AEP event, when an area of East Selsey was inundated near the museum. Elsewhere to the west of the model, no communities were inundated with East Wittering and Bracklesham remaining defended.

The Chichester model differed from the Arun to East Head model in that inundation was mainly as a result of still water flooding, with only four wave overtopping discharges calculated for the whole domain. The main communities affected by flooding here were Bosham, West Itchenor, Emsworth, Hayling Island and Thorney Island. To the eastern extent of the model, West Wittering remained defended throughout all simulations from 100 to 0.1% AEP events. Properties in West Itchenor were largely defended until the 1% to 0.5% AEP events. Several properties were affected in Birdham during the 0.1% AEP event only. In the area surrounding Bosham Quay, localised coastal flooding occurred during the 3.33% AEP event, with more widespread flooding occurring further inland during the 0.2 and 0.1% AEP events. Emsworth was mainly inundated in the area surrounding Hermitage Bridge and Quay Mill. To the south of Emsworth, in front of the Oyster Beds, the defences mitigated the risk of flooding for all AEP events except to the east of King Street. Although Hayling Island was heavily flooded to the north and east near Northney and North Hayling, most of the large communities on the island remained protected. To the south, more communities were affected by flooding including Selmore, Mengham and Eastoke. From the 20% AEP event onwards, properties at Sandy Point became affected by flooding, whilst Selmore was flooded during the 5% AEP event. On Thorney Island, which was heavily inundated, the two most densely populated areas remained largely protected with flooding more restricted to the area around Baker Barracks, which flooded during the 0.2% AEP event.

9.2 Limitations and recommendations

The approaches taken in this study include the most appropriate 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. The joint probability approaches for coastal studies may be in line for review due to the methods being applied for the SotN study. For this study the joint probability is based on the Defra desk based approach. This simplified joint probability method, originally developed by HR Wallingford, creates contours of the extreme variables that have an equal likelihood of exceedence. Combinations of the variables that lie on a contour are used to provide the joint probability combinations of two dependent variables. The dependence factors used within this study were taken from the Technical report FD2308/TR1¹⁶ for the closest gauged location at Portsmouth and then applied across the modelled domain. There are limitations in that it can only be applied to calculate the joint probability between two variables and assumptions have to be made to calculate the associated conditions for the variables that are not included in the calculations. It is also well-known that this method contains a known error that results in an underestimation of overtopping rates (Hawkes et al 2002). This method has been best practice for a number of years but new approaches are now coming into use to calculate the joint probability across multiple variables using an application of the extreme value method developed by Heffernan and Tawn (2004) utilising a more robust full Monte-Carlo method.

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 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 but where this data is not available the survey was supplemented with information from AIMS and point heights extracted from LIDAR data.
- It has just come to light that any LIDAR data flown since 2008 in the modelled area may not have been converted from a digital surface model (DSM) to a DTM. This means that the DTM used in the model may be from 2008 despite more recent surveys being available in DSM format.
- For the wave overtopping calculations the flood defence profiles, toe levels and crest heights remain the same throughout all of the modelled scenarios.
- The model grid size of 8m 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 drainage channel networks have been modelled through the use of gully lines which may not always represent the channel conveyance accurately.
- Infiltration losses into the beaches from wave overtopping have been assumed to equal 1.5L/s based on the results of the model calibration.
- Infiltration losses into the ground within the flood inundation model have not been considered.
- 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 NPPF average figures for the south of England.
- The discharge rates for the Felpham pumping station are assumed to be constant throughout the defended model simulations but are not accounted for in the undefended scenarios due to instabilities from the large volumes of water reaching the station when the defences are removed.

¹⁶ EA Defra (2005). Joint Probability: Dependence Mapping and Best Practice R&D Technical Report FD2308/TR1. HR Wallingford
2014s0753_Emsworth to Littlehampton_Coastal final report_v2.docx

- No account of the operation control rules from structures such as the Ferry and Felpham pumping stations are included in the model. The most recent versions of TUFLOW now allow the incorporation of control rules in the 2D domains.

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.

It is recommend that the joint probability approach is reviewed once new practices come into use.

Following the winter storms of 2013/14 it is recommend that the CFB data are updated to account for the additional gauged data and the notable extremes that occurred.

The model should be updated to use the latest version of the TUFLOW software to allow incorporation of the operational control rules.

As many of the beaches are shingle beach defences it is recommend that an alternative method is used to calculate the wave overtopping, such as X-Beach-G, to take account of the dynamic response of the beach throughout the course of the event and compare these to the rates used within this study.

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.

Appendices

A Joint probability conditions

Wave type (Swell/total sea)	Joint exceedance probability (AEP %)	Water level (AEP %)	Water level at Littlehampton (mAOD)	Offshore wave height (m)	Offshore wave period (seconds)	Offshore wave direction (°)	Wind speed (m/s)	Wind direction (°)
Total	20	100	3.35	5.81	8.76	225	19.16	225
Total	20	100	3.35	4.97	7.99	225	17.11	225
Total	20	100	3.35	3.84	6.95	225	14.34	225
Total	20	100	3.35	3.25	6.40	225	12.88	225
Total	20	50	3.43	2.65	5.86	225	11.43	225
Total	20	20	3.52	1.87	5.13	225	9.50	225
Total	20	100	3.35	4.94	7.27	180	21.67	180
Total	20	100	3.35	4.51	6.93	180	20.08	180
Total	20	100	3.35	3.69	6.28	180	17.03	180
Total	20	100	3.35	3.25	5.94	180	15.41	180
Total	20	50	3.43	2.82	5.59	180	13.79	180
Total	20	20	3.52	2.24	5.14	180	11.65	180
Total	20	100	3.35	2.76	5.30	135	14.60	135
Total	20	100	3.35	2.40	4.97	135	13.14	135
Total	20	100	3.35	1.76	4.37	135	10.52	135
Total	20	100	3.35	1.42	4.06	135	9.13	135
Total	20	50	3.43	1.07	3.75	135	7.74	135
Total	20	20	3.52	0.62	3.33	135	5.91	135
Total	5	100	3.35	7.10	9.94	225	22.32	225
Total	5	100	3.35	6.42	9.32	225	20.65	225
Total	5	100	3.35	5.35	8.34	225	18.03	225
Total	5	100	3.35	4.75	7.79	225	16.58	225
Total	5	50	3.43	4.16	7.24	225	15.12	225
Total	5	20	3.52	3.38	6.52	225	13.20	225
Total	5	5	3.67	2.19	5.43	225	10.29	225
Total	5	100	3.35	5.52	7.73	180	23.85	180
Total	5	100	3.35	5.42	7.65	180	23.48	180
Total	5	100	3.35	4.85	7.20	180	21.33	180
Total	5	100	3.35	4.48	6.90	180	19.96	180
Total	5	50	3.43	4.04	6.56	180	18.36	180
Total	5	20	3.52	3.47	6.11	180	16.22	180
Total	5	5	3.67	2.60	5.42	180	12.98	180
Total	5	100	3.35	3.30	5.80	135	16.79	135
Total	5	100	3.35	3.21	5.71	135	16.42	135
Total	5	100	3.35	2.68	5.23	135	14.29	135
Total	5	100	3.35	2.37	4.94	135	13.03	135
Total	5	50	3.43	2.04	4.63	135	11.65	135
Total	5	20	3.52	1.58	4.22	135	9.81	135
Total	5	5	3.67	0.90	3.59	135	7.04	135
Total	3.33	100	3.35	7.45	10.27	225	23.17	225
Total	3.33	100	3.35	6.78	9.65	225	21.53	225
Total	3.33	100	3.35	5.79	8.74	225	19.11	225
Total	3.33	100	3.35	5.20	8.20	225	17.66	225
Total	3.33	50	3.43	4.60	7.65	225	16.21	225
Total	3.33	20	3.52	3.82	6.93	225	14.28	225
Total	3.33	5	3.67	2.63	5.83	225	11.37	225
Total	3.33	3.33	3.71	2.28	5.52	225	10.52	225
Total	3.33	100	3.35	5.66	7.84	180	24.38	180
Total	3.33	100	3.35	5.64	7.82	180	24.29	180
Total	3.33	100	3.35	5.11	7.40	180	22.30	180
Total	3.33	100	3.35	4.78	7.14	180	21.09	180
Total	3.33	50	3.43	4.40	6.85	180	19.69	180
Total	3.33	20	3.52	3.83	6.39	180	17.55	180
Total	3.33	5	3.67	2.96	5.70	180	14.31	180
Total	3.33	3.33	3.71	2.70	5.50	180	13.36	180
Total	3.33	100	3.35	3.44	5.93	135	17.37	135
Total	3.33	100	3.35	3.42	5.91	135	17.27	135
Total	3.33	100	3.35	2.92	5.44	135	15.23	135

Wave type (Swell/total sea)	Joint exceedance probability (AEP %)	Water level (AEP %)	Water level at Littlehampton (mAOD)	Offshore wave height (m)	Offshore wave period (seconds)	Offshore wave direction (°)	Wind speed (m/s)	Wind direction (°)
Total	3.33	100	3.35	2.63	5.18	135	14.06	135
Total	3.33	50	3.43	2.32	4.89	135	12.79	135
Total	3.33	20	3.52	1.87	4.47	135	10.96	135
Total	3.33	5	3.67	1.18	3.85	135	8.18	135
Total	3.33	3.33	3.71	0.98	3.66	135	7.37	135
Total	1.33	100	3.35	8.13	10.90	225	24.85	225
Total	1.33	100	3.35	7.56	10.37	225	23.45	225
Total	1.33	100	3.35	6.67	9.55	225	21.27	225
Total	1.33	100	3.35	6.15	9.08	225	20.01	225
Total	1.33	50	3.43	5.60	8.57	225	18.65	225
Total	1.33	20	3.52	4.81	7.84	225	16.73	225
Total	1.33	5	3.67	3.63	6.75	225	13.82	225
Total	1.33	3.33	3.71	3.28	6.43	225	12.96	225
Total	1.33	2	3.77	2.84	6.03	225	11.89	225
Total	1.33	1.33	3.81	2.50	5.71	225	11.04	225
Total	1.33	100	3.35	5.95	8.06	180	25.42	180
Total	1.33	100	3.35	5.95	8.06	180	25.42	180
Total	1.33	100	3.35	5.63	7.81	180	24.24	180
Total	1.33	100	3.35	5.36	7.60	180	23.24	180
Total	1.33	50	3.43	5.06	7.37	180	22.15	180
Total	1.33	20	3.52	4.61	7.01	180	20.46	180
Total	1.33	5	3.67	3.77	6.34	180	17.32	180
Total	1.33	3.33	3.71	3.51	6.14	180	16.38	180
Total	1.33	2	3.77	3.19	5.89	180	15.18	180
Total	1.33	1.33	3.81	2.94	5.69	180	14.24	180
Total	1.33	100	3.35	3.74	6.20	135	18.57	135
Total	1.33	100	3.35	3.74	6.20	135	18.57	135
Total	1.33	100	3.35	3.41	5.90	135	17.23	135
Total	1.33	100	3.35	3.15	5.66	135	16.18	135
Total	1.33	50	3.43	2.88	5.41	135	15.08	135
Total	1.33	20	3.52	2.49	5.05	135	13.48	135
Total	1.33	5	3.67	1.82	4.43	135	10.76	135
Total	1.33	3.33	3.71	1.62	4.25	135	9.95	135
Total	1.33	2	3.77	1.37	4.02	135	8.93	135
Total	1.33	1.33	3.81	1.17	3.83	135	8.12	135
Total	1	100	3.35	8.33	11.08	225	25.34	225
Total	1	100	3.35	7.78	10.57	225	23.99	225
Total	1	100	3.35	6.92	9.78	225	21.88	225
Total	1	100	3.35	6.44	9.34	225	20.71	225
Total	1	50	3.43	5.90	8.84	225	19.38	225
Total	1	20	3.52	5.13	8.13	225	17.49	225
Total	1	5	3.67	3.94	7.04	225	14.58	225
Total	1	3.33	3.71	3.59	6.72	225	13.73	225
Total	1	2	3.77	3.16	6.32	225	12.66	225
Total	1	1.33	3.81	2.81	6.00	225	11.81	225
Total	1	1	3.84	2.56	5.77	225	11.20	225
Total	1	100	3.35	6.02	8.13	180	25.71	180
Total	1	100	3.35	6.02	8.13	180	25.71	180
Total	1	100	3.35	5.76	7.92	180	24.74	180
Total	1	100	3.35	5.53	7.73	180	23.87	180
Total	1	50	3.43	5.24	7.50	180	22.78	180
Total	1	20	3.52	4.83	7.18	180	21.26	180
Total	1	5	3.67	4.02	6.55	180	18.27	180
Total	1	3.33	3.71	3.77	6.34	180	17.32	180
Total	1	2	3.77	3.45	6.09	180	16.13	180
Total	1	1.33	3.81	3.19	5.89	180	15.18	180
Total	1	1	3.84	3.01	5.75	180	14.51	180
Total	1	100	3.35	3.83	6.28	135	18.93	135
Total	1	100	3.35	3.83	6.28	135	18.93	135
Total	1	100	3.35	3.54	6.02	135	17.78	135
Total	1	100	3.35	3.31	5.80	135	16.82	135
Total	1	50	3.43	3.04	5.55	135	15.72	135
Total	1	20	3.52	2.67	5.21	135	14.22	135
Total	1	5	3.67	2.02	4.61	135	11.58	135
Total	1	3.33	3.71	1.82	4.43	135	10.76	135
Total	1	2	3.77	1.57	4.20	135	9.74	135

Wave type (Swell/total sea)	Joint exceedance probability (AEP %)	Water level (AEP %)	Water level at Littlehampton (mAOD)	Offshore wave height (m)	Offshore wave period (seconds)	Offshore wave direction (°)	Wind speed (m/s)	Wind direction (°)
Total	1	1.33	3.81	1.37	4.02	135	8.93	135
Total	1	1	3.84	1.23	3.89	135	8.35	135
Total	0.5	100	3.35	8.70	11.42	225	26.25	225
Total	0.5	100	3.35	8.28	11.03	225	25.21	225
Total	0.5	100	3.35	7.51	10.33	225	23.34	225
Total	0.5	100	3.35	7.05	9.90	225	22.20	225
Total	0.5	50	3.43	6.57	9.46	225	21.03	225
Total	0.5	20	3.52	5.87	8.82	225	19.31	225
Total	0.5	5	3.67	4.69	7.73	225	16.43	225
Total	0.5	3.33	3.71	4.35	7.41	225	15.58	225
Total	0.5	2	3.77	3.91	7.01	225	14.51	225
Total	0.5	1.33	3.81	3.56	6.69	225	13.66	225
Total	0.5	1	3.84	3.32	6.47	225	13.05	225
Total	0.5	0.5	3.93	2.72	5.92	225	11.60	225
Total	0.5	100	3.35	6.19	8.26	180	26.34	180
Total	0.5	100	3.35	6.19	8.26	180	26.34	180
Total	0.5	100	3.35	6.04	8.14	180	25.78	180
Total	0.5	100	3.35	5.85	7.99	180	25.07	180
Total	0.5	50	3.43	5.63	7.81	180	24.24	180
Total	0.5	20	3.52	5.26	7.52	180	22.88	180
Total	0.5	5	3.67	4.61	7.01	180	20.45	180
Total	0.5	3.33	3.71	4.38	6.83	180	19.60	180
Total	0.5	2	3.77	4.06	6.58	180	18.41	180
Total	0.5	1.33	3.81	3.80	6.37	180	17.46	180
Total	0.5	1	3.84	3.62	6.23	180	16.79	180
Total	0.5	0.5	3.93	3.19	5.89	180	15.17	180
Total	0.5	100	3.35	4.02	6.46	135	19.72	135
Total	0.5	100	3.35	4.02	6.46	135	19.72	135
Total	0.5	100	3.35	3.85	6.30	135	19.01	135
Total	0.5	100	3.35	3.64	6.11	135	18.15	135
Total	0.5	50	3.43	3.41	5.89	135	17.22	135
Total	0.5	20	3.52	3.06	5.58	135	15.82	135
Total	0.5	5	3.67	2.48	5.05	135	13.47	135
Total	0.5	3.33	3.71	2.30	4.87	135	12.72	135
Total	0.5	2	3.77	2.05	4.64	135	11.70	135
Total	0.5	1.33	3.81	1.85	4.46	135	10.88	135
Total	0.5	1	3.84	1.71	4.33	135	10.31	135
Total	0.5	0.5	3.93	1.37	4.01	135	8.92	135
Total	0.2	100	3.35	9.12	11.81	225	27.29	225
Total	0.2	100	3.35	8.85	11.56	225	26.63	225
Total	0.2	100	3.35	8.19	10.95	225	25.00	225
Total	0.2	100	3.35	7.80	10.59	225	24.03	225
Total	0.2	50	3.43	7.37	10.20	225	22.99	225
Total	0.2	20	3.52	6.74	9.62	225	21.45	225
Total	0.2	5	3.67	5.69	8.65	225	18.88	225
Total	0.2	3.33	3.71	5.34	8.33	225	18.02	225
Total	0.2	2	3.77	4.91	7.93	225	16.95	225
Total	0.2	1.33	3.81	4.56	7.61	225	16.10	225
Total	0.2	1	3.84	4.31	7.38	225	15.50	225
Total	0.2	0.5	3.93	3.72	6.84	225	14.04	225
Total	0.2	0.2	4.04	2.93	6.12	225	12.12	225
Total	0.2	100	3.35	6.38	8.41	180	27.05	180
Total	0.2	100	3.35	6.38	8.41	180	27.05	180
Total	0.2	100	3.35	6.33	8.37	180	26.86	180
Total	0.2	100	3.35	6.19	8.26	180	26.32	180
Total	0.2	50	3.43	6.02	8.12	180	25.69	180
Total	0.2	20	3.52	5.75	7.91	180	24.69	180
Total	0.2	5	3.67	5.22	7.49	180	22.73	180
Total	0.2	3.33	3.71	5.05	7.36	180	22.09	180
Total	0.2	2	3.77	4.81	7.16	180	21.19	180
Total	0.2	1.33	3.81	4.59	6.99	180	20.38	180
Total	0.2	1	3.84	4.43	6.87	180	19.81	180
Total	0.2	0.5	3.93	4.00	6.53	180	18.19	180
Total	0.2	0.2	4.04	3.42	6.07	180	16.05	180
Total	0.2	100	3.35	4.26	6.68	135	20.68	135
Total	0.2	100	3.35	4.26	6.68	135	20.68	135

Wave type (Swell/total sea)	Joint exceedance probability (AEP %)	Water level (AEP %)	Water level at Littlehampton (mAOD)	Offshore wave height (m)	Offshore wave period (seconds)	Offshore wave direction (°)	Wind speed (m/s)	Wind direction (°)
Total	0.2	100	3.35	4.20	6.62	135	20.43	135
Total	0.2	100	3.35	4.02	6.46	135	19.70	135
Total	0.2	50	3.43	3.82	6.27	135	18.90	135
Total	0.2	20	3.52	3.53	6.01	135	17.73	135
Total	0.2	5	3.67	3.02	5.54	135	15.66	135
Total	0.2	3.33	3.71	2.86	5.40	135	15.02	135
Total	0.2	2	3.77	2.65	5.20	135	14.15	135
Total	0.2	1.33	3.81	2.47	5.03	135	13.42	135
Total	0.2	1	3.84	2.34	4.91	135	12.89	135
Total	0.2	0.5	3.93	2.00	4.60	135	11.50	135
Total	0.2	0.2	4.04	1.55	4.18	135	9.67	135
Total	0.1	100	3.35	9.41	12.08	225	28.00	225
Total	0.1	100	3.35	9.25	11.92	225	27.59	225
Total	0.1	100	3.35	8.64	11.37	225	26.11	225
Total	0.1	100	3.35	8.29	11.04	225	25.25	225
Total	0.1	50	3.43	7.91	10.69	225	24.30	225
Total	0.1	20	3.52	7.35	10.17	225	22.93	225
Total	0.1	5	3.67	6.39	9.29	225	20.58	225
Total	0.1	3.33	3.71	6.07	9.00	225	19.80	225
Total	0.1	2	3.77	5.66	8.62	225	18.80	225
Total	0.1	1.33	3.81	5.31	8.30	225	17.95	225
Total	0.1	1	3.84	5.07	8.08	225	17.35	225
Total	0.1	0.5	3.93	4.47	7.53	225	15.89	225
Total	0.1	0.2	4.04	3.69	6.81	225	13.97	225
Total	0.1	0.1	4.13	3.10	6.26	225	12.51	225
Total	0.1	100	3.35	6.51	8.51	180	27.51	180
Total	0.1	100	3.35	6.51	8.51	180	27.51	180
Total	0.1	100	3.35	6.51	8.51	180	27.51	180
Total	0.1	100	3.35	6.39	8.42	180	27.08	180
Total	0.1	50	3.43	6.25	8.30	180	26.55	180
Total	0.1	20	3.52	6.03	8.13	180	25.75	180
Total	0.1	5	3.67	5.61	7.80	180	24.19	180
Total	0.1	3.33	3.71	5.46	7.68	180	23.62	180
Total	0.1	2	3.77	5.25	7.51	180	22.82	180
Total	0.1	1.33	3.81	5.07	7.38	180	22.19	180
Total	0.1	1	3.84	4.95	7.28	180	21.73	180
Total	0.1	0.5	3.93	4.59	6.99	180	20.37	180
Total	0.1	0.2	4.04	4.04	6.56	180	18.33	180
Total	0.1	0.1	4.13	3.60	6.21	180	16.71	180
Total	0.1	100	3.35	4.42	6.83	135	21.34	135
Total	0.1	100	3.35	4.42	6.83	135	21.34	135
Total	0.1	100	3.35	4.42	6.83	135	21.34	135
Total	0.1	100	3.35	4.27	6.69	135	20.72	135
Total	0.1	50	3.43	4.09	6.52	135	20.00	135
Total	0.1	20	3.52	3.84	6.29	135	18.97	135
Total	0.1	5	3.67	3.39	5.88	135	17.17	135
Total	0.1	3.33	3.71	3.25	5.75	135	16.57	135
Total	0.1	2	3.77	3.05	5.56	135	15.76	135
Total	0.1	1.33	3.81	2.89	5.42	135	15.12	135
Total	0.1	1	3.84	2.78	5.31	135	14.66	135
Total	0.1	0.5	3.93	2.47	5.03	135	13.41	135
Total	0.1	0.2	4.04	2.03	4.63	135	11.62	135
Total	0.1	0.1	4.13	1.69	4.31	135	10.24	135
Swell	20	100	3.35	2.38	14.84	225	10.5	225
Swell	20	100	3.35	2.02	13.92	225	10.5	225
Swell	20	100	3.35	1.54	12.67	225	10.5	225
Swell	20	100	3.35	1.29	12.01	225	10.5	225
Swell	20	50	3.43	1.04	11.36	225	10.5	225
Swell	20	20	3.52	0.70	10.49	225	10.5	225
Swell	5	100	3.35	2.92	10.25	225	10.5	225
Swell	5	100	3.35	2.63	10.25	225	10.5	225
Swell	5	100	3.35	2.18	14.33	225	10.5	225
Swell	5	100	3.35	1.93	13.68	225	10.5	225
Swell	5	50	3.43	1.68	13.02	225	10.5	225
Swell	5	20	3.52	1.34	12.16	225	10.5	225
Swell	5	5	3.67	0.84	10.85	225	10.5	225

Wave type (Swell/total sea)	Joint exceedance probability (AEP %)	Water level (AEP %)	Water level at Littlehampton (mAOD)	Offshore wave height (m)	Offshore wave period (seconds)	Offshore wave direction (°)	Wind speed (m/s)	Wind direction (°)
Swell	3.33	100	3.35	3.07	10.25	225	10.5	225
Swell	3.33	100	3.35	2.79	10.25	225	10.5	225
Swell	3.33	100	3.35	2.37	14.81	225	10.5	225
Swell	3.33	100	3.35	2.12	14.16	225	10.5	225
Swell	3.33	50	3.43	1.86	13.51	225	10.5	225
Swell	3.33	20	3.52	1.53	12.64	225	10.5	225
Swell	3.33	5	3.67	1.03	11.33	225	10.5	225
Swell	3.33	3.33	3.71	0.88	10.95	225	10.5	225
Swell	1.33	100	3.35	3.35	10.25	225	10.5	225
Swell	1.33	100	3.35	3.11	10.25	225	10.5	225
Swell	1.33	100	3.35	2.74	10.25	225	10.5	225
Swell	1.33	100	3.35	2.52	10.25	225	10.5	225
Swell	1.33	50	3.43	2.29	14.61	225	10.5	225
Swell	1.33	20	3.52	1.95	13.74	225	10.5	225
Swell	1.33	5	3.67	1.45	12.43	225	10.5	225
Swell	1.33	3.33	3.71	1.30	12.05	225	10.5	225
Swell	1.33	0.5	3.93	1.12	11.57	225	10.5	225
Swell	1.33	1.33	3.81	0.97	11.19	225	10.5	225
Swell	1	100	3.35	3.44	10.25	225	10.5	225
Swell	1	100	3.35	3.21	10.25	225	10.5	225
Swell	1	100	3.35	2.85	10.25	225	10.5	225
Swell	1	100	3.35	2.64	10.25	225	10.5	225
Swell	1	50	3.43	2.41	14.94	225	10.5	225
Swell	1	20	3.52	2.09	14.09	225	10.5	225
Swell	1	5	3.67	1.58	12.78	225	10.5	225
Swell	1	3.33	3.71	1.44	12.40	225	10.5	225
Swell	1	0.5	3.93	1.25	11.91	225	10.5	225
Swell	1	1.33	3.81	1.10	11.53	225	10.5	225
Swell	1	1	3.84	1.00	11.26	225	10.5	225
Swell	0.5	100	3.35	3.59	10.25	225	10.5	225
Swell	0.5	100	3.35	3.41	10.25	225	10.5	225
Swell	0.5	100	3.35	3.09	10.25	225	10.5	225
Swell	0.5	100	3.35	2.90	10.25	225	10.5	225
Swell	0.5	50	3.43	2.70	10.25	225	10.5	225
Swell	0.5	20	3.52	2.40	14.91	225	10.5	225
Swell	0.5	5	3.67	1.90	13.61	225	10.5	225
Swell	0.5	3.33	3.71	1.76	13.23	225	10.5	225
Swell	0.5	0.5	3.93	1.57	12.75	225	10.5	225
Swell	0.5	1.33	3.81	1.42	12.36	225	10.5	225
Swell	0.5	1	3.84	1.32	12.09	225	10.5	225
Swell	0.5	0.5	3.93	1.07	11.44	225	10.5	225
Swell	0.2	100	3.35	3.76	10.25	225	10.5	225
Swell	0.2	100	3.35	3.65	10.25	225	10.5	225
Swell	0.2	100	3.35	3.38	10.25	225	10.5	225
Swell	0.2	100	3.35	3.21	10.25	225	10.5	225
Swell	0.2	50	3.43	3.03	10.25	225	10.5	225
Swell	0.2	20	3.52	2.77	10.25	225	10.5	225
Swell	0.2	5	3.67	2.33	14.71	225	10.5	225
Swell	0.2	3.33	3.71	2.18	14.33	225	10.5	225
Swell	0.2	0.5	3.93	1.99	13.84	225	10.5	225
Swell	0.2	1.33	3.81	1.85	13.46	225	10.5	225
Swell	0.2	1	3.84	1.74	13.19	225	10.5	225
Swell	0.2	0.5	3.93	1.49	12.54	225	10.5	225
Swell	0.2	0.2	4.04	1.16	11.67	225	10.5	225
Swell	0.1	100	3.35	3.88	10.25	225	10.5	225
Swell	0.1	100	3.35	3.82	10.25	225	10.5	225
Swell	0.1	100	3.35	3.57	10.25	225	10.5	225
Swell	0.1	100	3.35	3.42	10.25	225	10.5	225
Swell	0.1	50	3.43	3.26	10.25	225	10.5	225
Swell	0.1	20	3.52	3.02	10.25	225	10.5	225
Swell	0.1	5	3.67	2.62	10.25	225	10.5	225
Swell	0.1	3.33	3.71	2.49	15.12	225	10.5	225
Swell	0.1	0.5	3.93	2.31	14.68	225	10.5	225
Swell	0.1	1.33	3.81	2.17	14.29	225	10.5	225
Swell	0.1	1	3.84	2.06	14.02	225	10.5	225
Swell	0.1	0.5	3.93	1.81	13.37	225	10.5	225

Wave type (Swell/ total sea)	Joint exceedance probability (AEP %)	Water level (AEP %)	Water level at Littlehampton (mAOD)	Offshore wave height (m)	Offshore wave period (seconds)	Offshore wave direction (°)	Wind speed (m/s)	Wind direction (°)
Swell	0.1	0.2	4.04	1.48	12.50	225	10.5	225
Swell	0.1	0.1	4.13	1.22	11.85	225	10.5	225

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APPENDIX 3

Ground and Water Ltd							Trialpit No TP1 Sheet 1 of 1	
Project Name Fowley Cottage			Project No. GWPR1704		Co-ords: - Level: -		Date 20/06/2016	
Location: 46 Warblington Road, Emsworth, Hampshire PO10 7HD					Dimensions: -		Scale 1:25	
Client: Charles Glanville c/o Re-Format Ltd					Depth 4.30m		Logged By JL	
Samples & In Situ Testing			Depth (m)	Level (m AOD)	Legend	Stratum Description		
Depth (m)	Type	Results						
0.20	D		0.30			TOPSOIL: Mid brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare to occasional, sub-angular to sub-rounded, fine to coarse flint.		
0.50	D					RIVER TERRACE DEPOSITS: Mottled light/mid brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare to occasional, fine to medium, sub-angular to sub-rounded flint.		
0.80	D							
1.20	D		1.30			RIVER TERRACE DEPOSITS: Mottled light brown/orange silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine, sub-angular to sub-rounded flint and chalk fragments.		
1.50	D							
1.80	D							
2.20	D		2.00			RIVER TERRACE DEPOSITS: Mottled light brown/grey/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is abundant, fine, sub-angular to sub-rounded chalk fragments.		
2.50	D							
2.80	D							
3.00	D					RIVER TERRACE DEPOSITS: Light brown/off white sandy gravelly CLAY. Sand is fine to coarse grained. Gravel is abundant, fine to medium, sub-angular to sub-rounded flint and chalk.		
3.40	D							
3.80	D							
4.30	D		4.30			Trialpit Complete at 4.30 m		
Remarks: Roots noted to 1.80m bgl.								
Groundwater: No groundwater encountered.								



Ground and Water Ltd						Trialpit No TP2 Sheet 1 of 1	
Project Name Fowley Cottage			Project No. GWPR1704		Co-ords: - Level: -		Date 20/06/2016
Location: 46 Warblington Road, Emsworth, Hampshire PO10 7HD					Dimensions: - Depth 3.40m		Scale 1:25
Client: Charles Glanville c/o Re-Format Ltd							Logged By JL
Samples & In Situ Testing			Depth (m)	Level (m AOD)	Legend	Stratum Description	
Depth (m)	Type	Results					
0.20	D		0.30			TOPSOIL: Mid brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare, sub-angular to sub-rounded, fine to coarse flint.	1
0.50	D					RIVER TERRACE DEPOSITS: Mottled light brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare, fine to medium, sub-angular to sub-rounded flint.	
0.80	D						
1.20	D		1.30			RIVER TERRACE DEPOSITS: Mottled light brown/orange silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine, sub-angular to sub-rounded flint and chalk.	2
1.50	D						
1.80	D						
2.20	D		2.10			RIVER TERRACE DEPOSITS: Light brown/orange/grey silty sandy gravelly CLAY. Sand is fine grained. Gravel is abundant, fine to medium, sub-angular to sub-rounded flint and chalk.	3
2.50	D						
2.80	D						
3.40	D		3.40				4
Trialpit Complete at 3.40 m							
Remarks: Roots noted to 0.80m bgl							
Groundwater: No groundwater encountered.							



Ground and Water Ltd						Trialpit No TP3 Sheet 1 of 1		
Project Name Fowley Cottage			Project No. GWPR1704		Co-ords: - Level: -		Date 20/06/2016	
Location: 46 Warblington Road, Emsworth, Hampshire PO10 7HD					Dimensions: -		Scale 1:25	
Client: Charles Glanville c/o Re-Format Ltd					Depth 3.00m		Logged By JL	
Samples & In Situ Testing			Depth (m)	Level (m AOD)	Legend	Stratum Description		
Depth (m)	Type	Results						
0.10	D		0.15			TOPSOIL: Mid brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare to occasional, fine to coarse, sub-angular to sub-rounded flint.		
0.30	D							
0.50	D							
1.00	D		1.50			RIVER TERRACE DEPOSITS: Light brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare, fine to medium, sub-angular to sub-rounded flint.	1	
1.40	D							
1.80	D							
			2.10			RIVER TERRACE DEPOSITS: Mottled light brown/orange silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine, sub-angular to sub-rounded flint and chalk.	2	
			2.40			RIVER TERRACE DEPOSITS: Light brown/orange/grey silty sandy gravelly CLAY. Sand is fine to coarse grained. Gravel is abundant, fine to coarse, sub-angular to sub-rounded flint and chalk.		
2.50	D		3.00			LEWES NODULAR CHALK FORMATION: Structureless CHALK composed of cream gravelly sandy SILT. Gravels are moderately weak, high density, white and sub-angular. Locally white/light brown with a trace of light brown clay.	3	
2.80	D							
						Trialpit Complete at 3.00 m		4
Remarks: Roots noted to 0.70m bgl.								
Groundwater: No groundwater encountered.								

Ground and Water Ltd							Trialpit No TP4 Sheet 1 of 1	
Project Name Fowley Cottage			Project No. GWPR1704		Co-ords: - Level: -		Date 20/06/2016	
Location: 46 Warblington Road, Emsworth, Hampshire PO10 7HD					Dimensions: -		Scale 1:25	
Client: Charles Glanville c/o Re-Format Ltd					Depth 3.10m		Logged By JL	
Samples & In Situ Testing			Depth (m)	Level (m AOD)	Legend	Stratum Description		
Depth (m)	Type	Results						
0.15	D		0.15			TOPSOIL: Mid brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare, fine to coarse, sub-angular to sub-rounded flint.		
0.30	D					RIVER TERRACE DEPOSITS: Light brown/mid-brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare to occasional, fine to medium, sub-angular to sub-rounded flint.		
0.90	D							
1.20	D							
1.50	D							
1.80	D		1.70			RIVER TERRACE DEPOSITS: Light brown/orange silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine, sub-angular to sub-rounded flint and chalk.		
2.20	D		2.10			RIVER TERRACE DEPOSITS: Light brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is abundant, fine to coarse, flint and chalk.		
2.50	D							
3.00	D		2.90			RIVER TERRACE DEPOSITS: Light grey/off white sandy gravelly CLAY. Sand is fine to coarse grained. Gravel is abundant, sub-angular to sub-rounded, fine to coarse flint and chalk.		
			3.10			Trialpit Complete at 3.10 m		
Remarks: Roots noted to 1.00m bgl.								
Groundwater: No groundwater encountered.								

Ground and Water Ltd							Trialpit No TP5 Sheet 1 of 1	
Project Name Fowley Cottage			Project No. GWPR1704		Co-ords: - Level: -		Date 20/06/2016	
Location: 46 Warblington Road, Emsworth, Hampshire PO10 7HD					Dimensions: -		Scale 1:25	
Client: Charles Glanville c/o Re-Format Ltd					Depth 3.90m		Logged By JL	
Samples & In Situ Testing			Depth (m)	Level (m AOD)	Legend	Stratum Description		
Depth (m)	Type	Results						
0.20	D		0.20			TOPSOIL: Mid brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare, fine to medium, sub-angular to sub-rounded flint.		
0.50	D					RIVER TERRACE DEPOSITS: Mid brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare, fine to medium, sub-angular to sub-rounded flint.		
0.80	D							
1.20	D		1.10			RIVER TERRACE DEPOSITS: Light brown/orange/grey silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine to coarse, sub-angular to sub-rounded flint and chalk. Coarse chalk fragments noted from 2.40m bgl.		
1.50	D							
1.80	D							
2.20	D							
2.70	D							
3.40	D							
3.80	D		3.90					
						Trialpit Complete at 3.90 m		
Remarks: Roots to 1.30m bgl.								
Groundwater: No groundwater encountered.								

Ground and Water Ltd						Trialpit No TP6 Sheet 1 of 1	
Project Name Fowley Cottage			Project No. GWPR1704		Co-ords: - Level: -		Date 20/06/2016
Location: 46 Warblington Road, Emsworth, Hampshire PO10 7HD					Dimensions: - Depth 3.70m		Scale 1:25
Client: Charles Glanville c/o Re-Format Ltd							Logged By JL
Samples & In Situ Testing			Depth (m)	Level (m AOD)	Legend	Stratum Description	
Depth (m)	Type	Results					
0.20	D		0.20			TOPSOIL: Mid to dark brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare, fine to coarse, sub-angular to sub-rounded flint.	
0.50	D					MADE GROUND: Light brown silty sandy gravelly clay. Sand is fine to medium grained. Gravel is fine to abundant, angular to sub-rounded, fine to coarse flint, chalk and brick.	
0.80	D		0.80			RIVER TERRACE DEPOSITS: Mottled light brown/orange silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine to medium, sub-angular to sub-rounded flint and chalk.	1
1.20	D						
1.50	D						
1.80	D						
2.20	D						2
2.70	D						
			2.90			RIVER TERRACE DEPOSITS: Mottled light brown/orange silty sandy gravelly CLAY. Sand is fine grained. Gravel is abundant, sub-angular to sub-rounded, fine to medium chalk and flint.	3
3.20	D						
3.70	D		3.70				
						Trialpit Complete at 3.70 m	4
Remarks: Roots noted to 1.80m bgl.							
Groundwater: No groundwater encountered.							



CLIENT	Charles Glanville c/o Re-Format Limited		
SITE ADDRESS	Fowley Cottage, 46 Warblington Road, Emsworth, Hampshire PO10 7HD		
REPORT REFERENCE	GWPR1704		
ENGINEER	Joshua Laite, Ground and Water Limited		
INVESTIGATION	Please see Figure 1 attached. Site works were undertaken on 20 th June 2016 and comprised the excavation of six		
LOCATIONS	trial pits (TP1-TP6) with soakaway testing in accordance with the principles of BRE365 in TP1-TP3 and TP5-TP6.		
GROUND CONDITIONS ENCOUNTERED	Summary of Strata Encountered (TP1-TP6)		
	Strata	Depth Encountered (m bgl)	Thickness (m)
	TOPSOIL: Mid brown silty sandy gravelly clay. Sand is fine grained. Gravel is rare to occasional, sub-angular to sub-rounded, fine to coarse flint.	GL	0.15 – 0.30
	MADE GROUND (TP6): Light brown silty sandy gravelly clay. Sand is fine to medium grained. Gravel is fine to abundant, fine to coarse, angular to sub-rounded flints, chalk and bricks.	0.20	0.60
	RIVER TERRACE DEPOSITS: Mottled light / mid brown / orange, silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare to occasional, fine to medium, sub-angular to sub-rounded flint.	0.15 – 0.30	0.90 – 1.55
	RIVER TERRACE DEPOSITS: Mottled light / mid brown / orange silty sandy gravelly CLAY. Sand is fine to medium grained. Gravel is abundant, fine to coarse, sub-rounded to sub-angular flints and chalk.	0.80 – 1.70	0.90 - 2.90
	RIVER TERRACE DEPOSITS (TP1 and TP4): Light brown / off white sandy gravelly CLAY. Sand is fine to coarse grained. Gravel is abundant, fine to medium, sub-angular to sub-rounded, flint and chalk.	2.90 - 3.90	0.20 – 0.40
	LEWES NODULAR CHALK FORMATION (TP3): Structureless CHALK composed of cream gravelly sandy SILT. Gravel are moderately weak, high density, white and sub-angular. Locally white / light brown with a trace of light brown clay.	2.40	0.60
ROOTS	Roots were noted between 0.70 - 1.80m bgl.		
GROUNDWATER	Groundwater was not encountered during this investigation.		

INFILTRATION RATE

Soakaway tests, in accordance with the principles of BRE365, were undertaken at depths from 3.10m bgl – 4.30m bgl (TP1, TP3, TP5 and TP6) within the soils of the 1st River Terrace Deposits, comprising a silty sandy gravelly clay.

No infiltration was recorded and therefore soakaways constructed within the soils of the 1st River Terraced Deposits are **not** likely to perform to a satisfactory standard.

Soakaway Testing (TP1, TP2, TP5 and TP6)				
Trial Hole	Starting Depth (m bgl)	Final Depth (m bgl)	Time Elapsed (mins)	Infiltration Rate (m/sec)
TP1	2.50	2.49	125	0
TP2	2.03	2.01	156	0
TP5	1.82	1.80	154	0
TP6	2.10	2.20	190	0

A soakaway test, in accordance with the principles of BRE365, was undertaken at a depth of 3.00m bg in TP3 within the soils of the Lewes Nodular Chalk Formation, comprising a cream gravelly sandy SILT. No infiltration was noted and therefore the soils of the Lewes Nodular Chalk Formation are also **not** likely to perform to a satisfactory standard.

Soakaway Testing (TP3)				
Trial Hole	Starting Depth	Final Depth	Time Elapsed	Infiltration Rate
TP3	1.10m bgl	1.10m bgl	116 min	0 m/sec

The site was located within a **Secondary A Aquifer** relating to superficial deposits of the River Terrace Deposits, beneath a **Principal Aquifer** comprising the bedrock deposits of Lewes Nodular Chalk Formation is present.

Examination of the Environment Agency records showed that the site **was not** located within a **Groundwater Source Protection Zone** as classified in the Policy and Practice for the Protection of Groundwater.

Consultation with the Environment Agency must be sought regarding any use that may have an impact on groundwater resources.

ANTICIPATED VOLUME CHANGE POTENTIAL

RIVER TERRACE DEPOSITS: Likely to have moderate to high volume change potential.

LEWES NODULAR CHALK FORMATION: Likely to have no volume change potential.

All in accordance with NHBC Standards Chapter 4.2 and BRE240. Subject to confirmation of results of geotechnical classification testing.

FOUNDATION RECOMMENDATIONS

At the time of reporting, June 2016, it is our understanding that the proposed development will incorporate the retention of the existing property and construction of 6No. detached properties with gardens in the surrounding grounds. Ancillary structures, such as site access, parking areas and private gardens will also be formed.

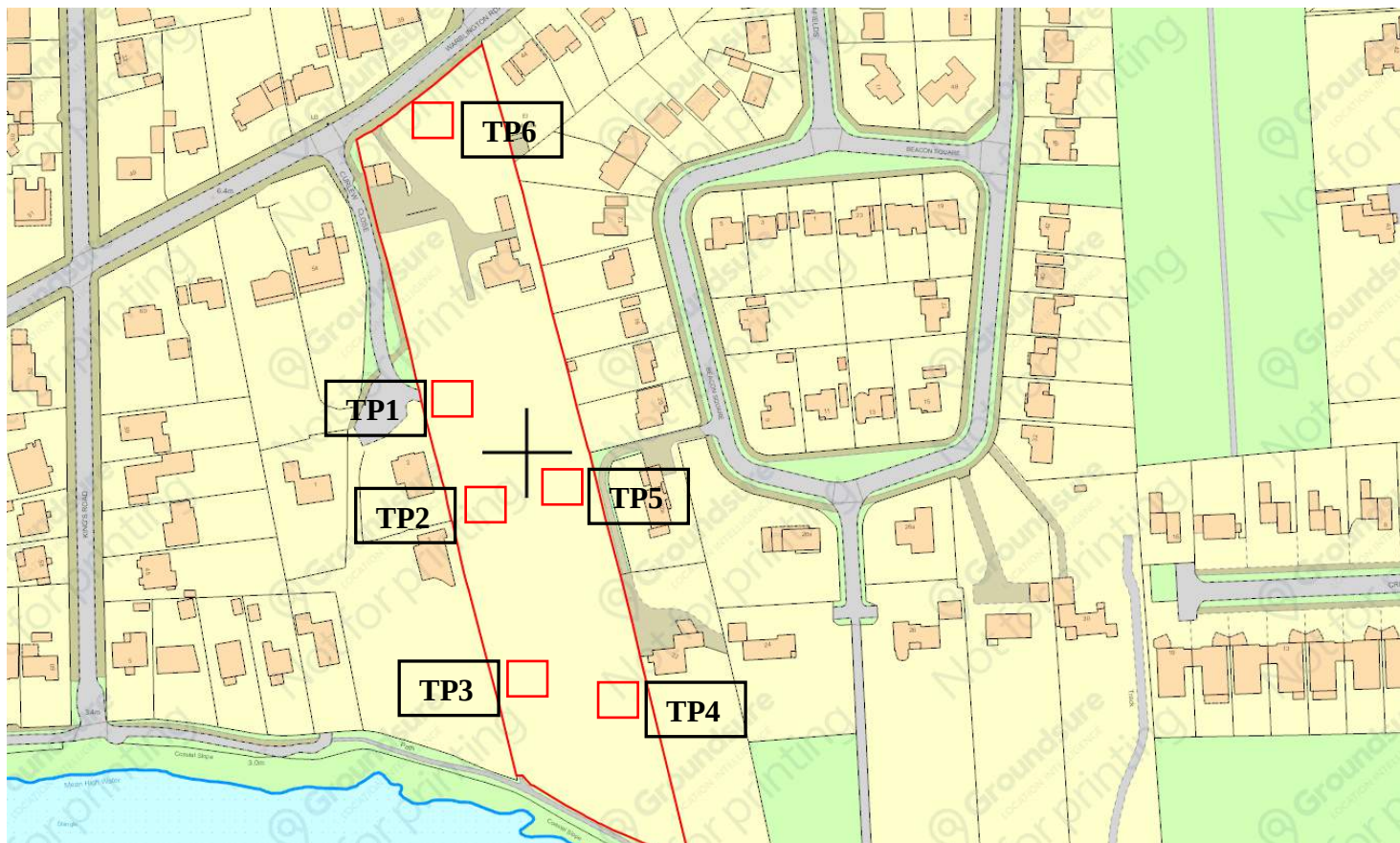
Foundations must not be placed within cohesive root penetrated and/or desiccated soils and the influence of the trees surrounding the site must be taken into account. The base of foundation excavations must extend at least 300mm into non-root penetrated soils. Should trees be removed from footprint of proposed development then a piled foundation should be considered.

Roots were generally noted ranging between 0.70 - 1.80m bgl, foundations minimum foundations depths rare likely to range from 1.00 - 2.10m bgl into the River Terrace Deposits.

Foundations constructed on the River Terrace Deposits at 1.00m bgl can be designed based on a presumed safe bearing capacity of 75kN/m², increasing to 100kN/m² at 2.10m bgl. This is based on trial hole records, inspection of samples recovered, geotechnical laboratory results, and referral to BS 8004:1986, Code of Practice for Foundations, the results of the insitu testing, and based on a 5m long by 1.0m wide foundation and a maximum settlement of ~25mm.

Given that the cohesive soils of the 1st River Terrace Deposits are likely to have a volume change potential in accordance with BRE Digest 240 and NHBC Chapter 4.2, a suspended slab should be considered. The volume change potential of the shallow surface soils must be taken into account in final design (underfloor void diameter/compressible material/void formers etc).

This preliminary information may be subject to amendment in the final report and no liability can be accepted for any actions based on this preliminary information.



NOT TO SCALE

 APPROXIMATE SITE BOUNDARY

Project:

Fowley Cottage, 46 Warblington Road, Emsworth, Hampshire PO10 7HD

Client:

Charles Glanville c/o Re-Format Limited

Date:

July 2016

Trial Hole Location Plan

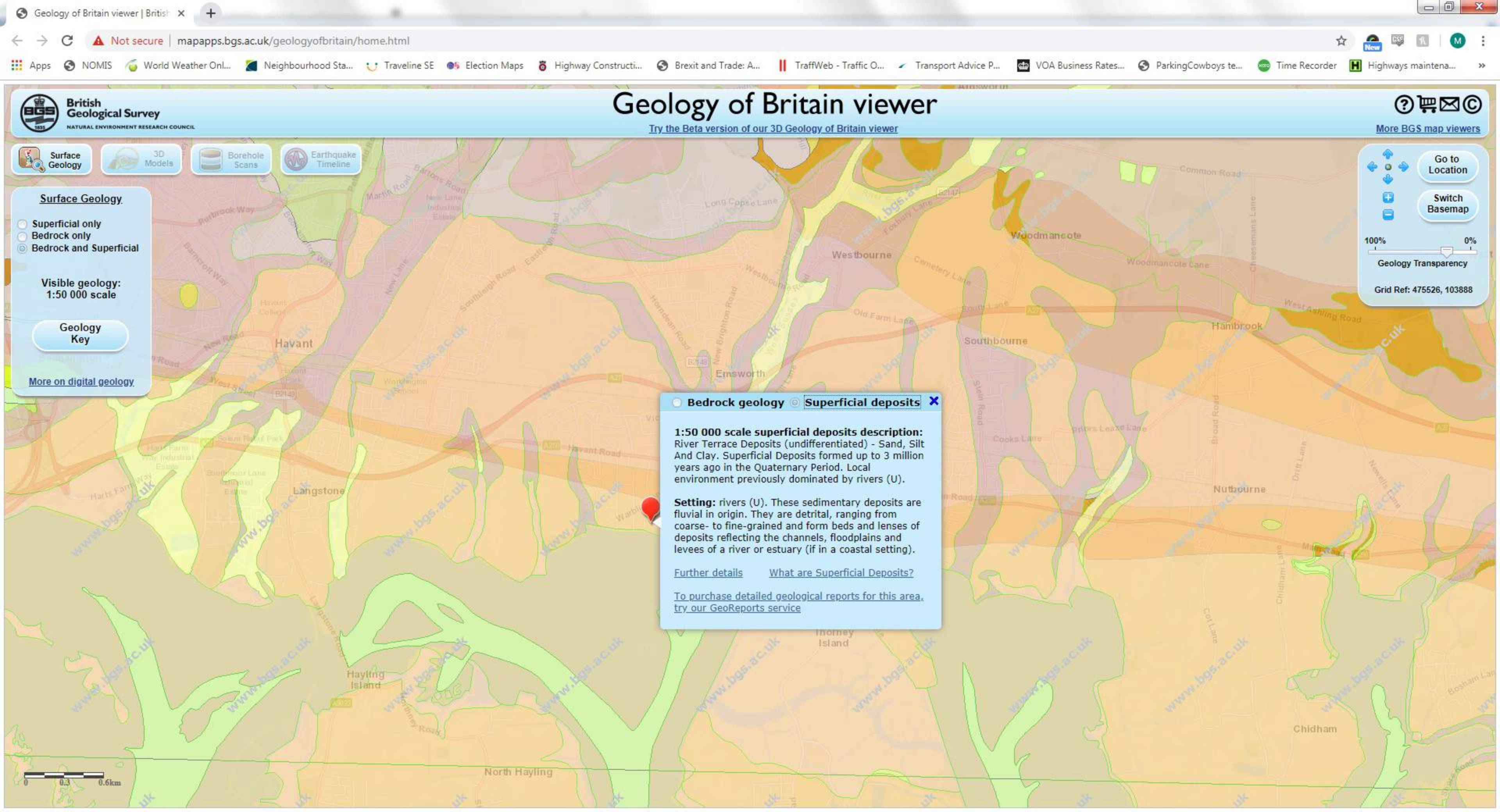
Ref:

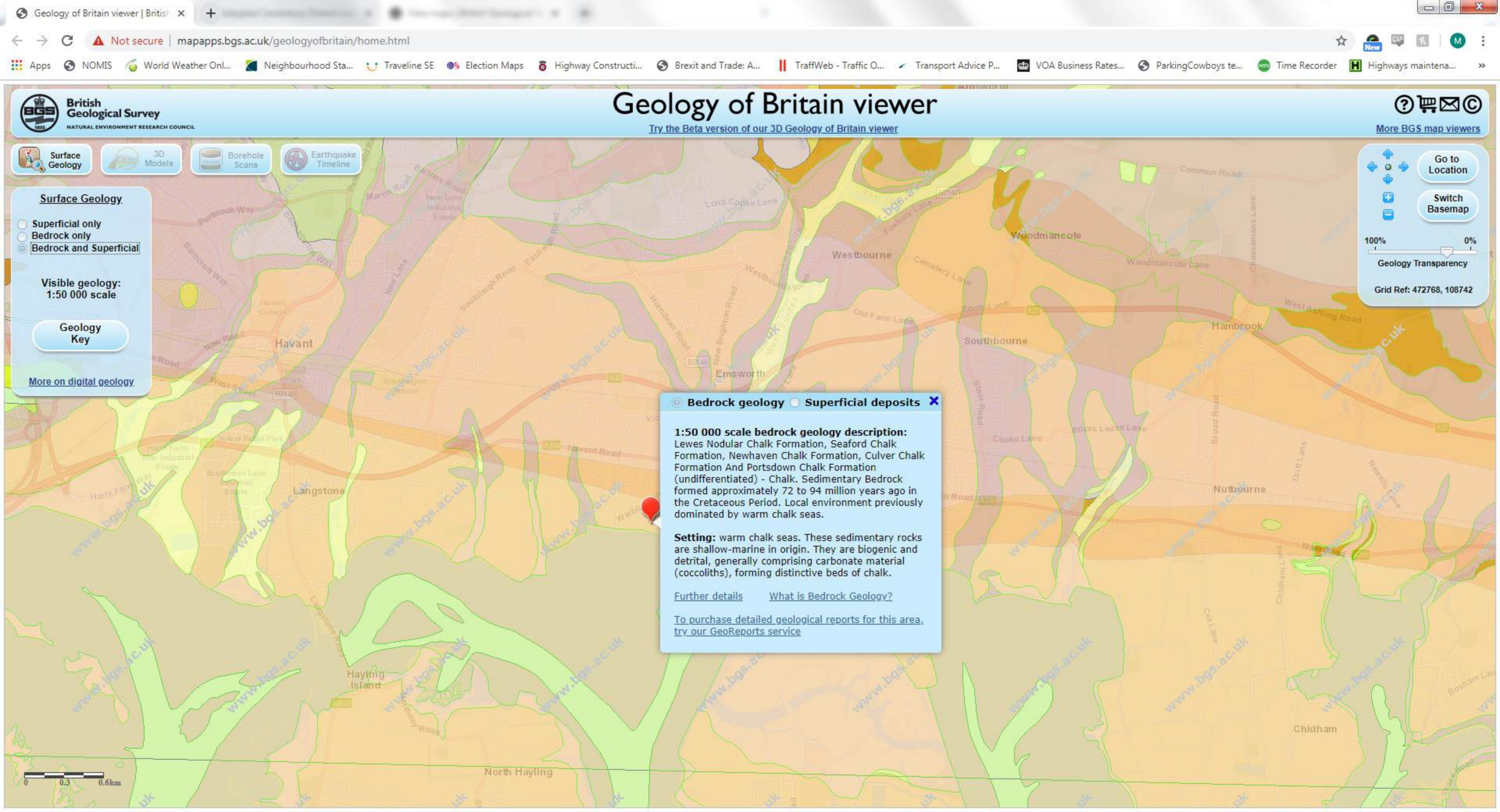
GWPR1704

Figure 1

ground&water

APPENDIX 4





APPENDIX 5

Bellamy Roberts		Page 1
Clover House Western Lane Odiham Hants RG29 1TU		
Date 14/01/2020 12:33 File	Designed by jasonbrown Checked by	
XP Solutions Source Control 2015.1		

ICP SUDS Mean Annual Flood

Input

Return Period (years)	100	Soil	0.400
Area (ha)	1.000	Urban	0.000
SAAR (mm)	700	Region Number	Region 7

Results l/s

QBAR Rural	3.4
QBAR Urban	3.4
Q100 years	10.9
Q1 year	2.9
Q30 years	7.7
Q100 years	10.9

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APPENDIX 6



Atkins Global
The Hub, 500
Park Avenue
Bristol
BS32 4RZ

Your ref 54886
Our ref 245484
Date 22 May 2017
Contact searches@southernwater.co.uk
Tel 0845 272 0845
0330 303 0276
Fax 01634 844514

Attention: Andrew Aitken

Dear Customer

Re: Provision of public sewer record extract

Location: Site surrounding Warblington Road, Emsworth, PO10 7HD

Thank you for your order regarding the provision of extracts of our sewer and/or water main records. Please find enclosed the extracts from Southern Water's records for the above location.

We confirm payment of your fee in the sum of £49.92 and enclose a VAT receipt for your records.

Customers should be aware that there are areas within our region in which there are neither sewers nor water mains. Similarly, whilst the enclosed extract may indicate the approximate location of our apparatus in the area of interest, it should not be relied upon as showing that further infrastructure does not exist and may subsequently be found following site investigation. Actual positions of the disclosed (and any undisclosed) infrastructure should therefore be determined on site, because Southern Water does not accept any responsibility for inaccuracy or omission regarding the enclosed plan. Accordingly it should not be considered to be a definitive document.

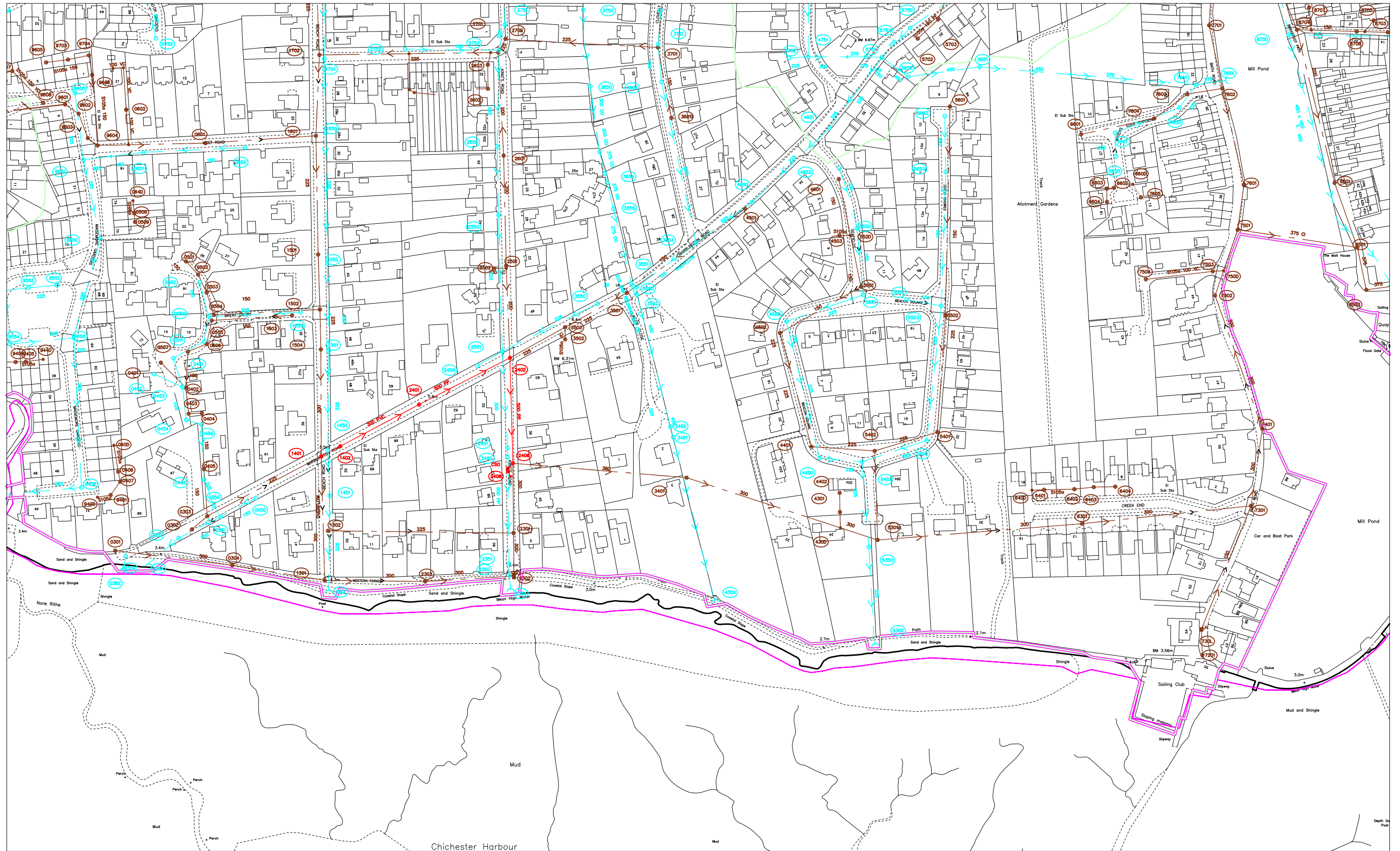
Should you require any further assistance regarding this matter, please contact the LandSearch team.

Yours faithfully

LandSearch

SEWER RECORDS PAGE 1 OF 2

105739



105161

473929

O.S.	REF.

SU7405SW

Title: 245484_Site surrounding Warbli

Drawn by: yadavsa

Scale: 1:2500

Date: 22/05/2017

The positions of pipes shown on this plan are believed to be correct, but Southern Water Services Ltd accept no responsibility in the event of inaccuracy.

WARNING: BAC pipes are constructed of Bonded Asbestos Cement
WARNING: Unknown (UNK) materials may include Bonded Asbestos Cement

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Licence No. WU 298530.



Southern
Water

174074

SEWER RECORDS PAGE 2 OF 2

Node	Cover	Invert	Size	Material	Shape	Node	Cover	Invert	Size	Material	Shape	Node	Cover	Invert	Size	Material	Shape	Node	Cover	Invert	Size	Material	Shape	
0301X	3.761	2.554	300	UNK	CIRC	2350X	3.269	1.749	UNK	UNK	CIRC	4652X	6.132	5.232	150	UNK	CIRC	8707Y			UNK	UNK	UNK	
0302X			UNK	UNK	CIRC	2401X	5.779	2.487	500	PP	CIRC	4751X	6.436		300	UNK	CIRC	870DX			150	UNK	CIRC	
0303X	4.279		225	UNK	CIRC	2402X	6.144		500	PP	CIRC	4752X	6.61		225	UNK	CIRC	871DY			150	UNK	CIRC	
0304X	3.429	2.478	300	UNK	CIRC	2406Y	4.75		300	UNK	CIRC	*	5301X	3.761		300	UNK	CIRC	8750X	3.035		450	UNK	RECT
0350X	4.382	3.312	225	UNK	CIRC	2406X	4.75	2.065	500	PP	CIRC	*	5350X	3.558	2.478	225	UNK	CIRC	9401X			UNK	UNK	CIRC
0351X	4.281		300	UNK	CIRC	2408X	4.61	2.628	500	PP	CIRC		5401X	5.069	3.094	225	UNK	CIRC	9404X			UNK	UNK	CIRC
0352X	3.562	2.652	300	UNK	CIRC	2450X			UNK	UNK	CIRC		5402X	4.721		UNK	UNK	CIRC	9405X			UNK	UNK	CIRC
0354X	2.62	2.17	UNK	UNK	CIRC	2451X	4.8	3.277	500	PP	CIRC	*	5450X	4.82	3.43	225	UNK	CIRC	9450X			225	UNK	CIRC
0401X	5.599	4.229	UNK	UNK	CIRC	2451Y	4.8	3.254	300	UNK	CIRC	*	5451X	4.536		225	UNK	CIRC	9550X	4.952	3.512	225	UNK	CIRC
0402X	5.583	4.033	UNK	UNK	CIRC	2452X	4.61	2.796	500	PP	CIRC		5501X	5.974	4.325	150	UNK	CIRC	9551X	4.471	3.701	225	UNK	CIRC
0403X	5.363	3.933	UNK	UNK	CIRC	2501X	6.334		200	UNK	CIRC		5502X	5.63	3.572	225	UNK	CIRC	9552X	4.469	3.619	225	UNK	CIRC
0404X	5.397	3.907	150	UNK	CIRC	2502X			100	VC	CIRC		552DX			150	UNK	CIRC	9554X	4.054	3.478	225	UNK	CIRC
0405X	4.398	3.258	150	UNK	CIRC	250DX			200	UNK	CIRC		5550X	6.155	5.055	150	UNK	CIRC	9555X	4.368		225	UNK	CIRC
0406X			UNK	UNK	CIRC	2550X	6.489	4.519	225	UNK	CIRC		5551X	5.922	4.792	150	UNK	CIRC	9601X			UNK	UNK	CIRC
0407X			UNK	UNK	CIRC	2551X	6.244		300	UNK	CIRC		5552X	5.777	4.387	150	UNK	CIRC	9602X			UNK	UNK	CIRC
040DX			UNK	UNK	CIRC	2601X	6.507	5.09	200	UNK	CIRC	*	5553X	5.624		225	UNK	CIRC	9603X			UNK	UNK	CIRC
0450X	4.919	3.109	150	UNK	CIRC	2601Y	6.507		200	UNK	CIRC	*	5552X			225	UNK	CIRC	9604Y			UNK	UNK	CIRC
0451X	5.558	4.518	225	UNK	CIRC	2602X			150	VC	CIRC		5601X	5.352	4.59	150	UNK	CIRC	9605X			150	UNK	CIRC
0452X	5.642	4.362	225	UNK	CIRC	2603X			150	VC	CIRC		5650X	5.568		450	UNK	CIRC	9606X			150	UNK	CIRC
0453X	5.758		225	UNK	CIRC	263DX			150	VC	CIRC		5651X	4.815	3.935	450	UNK	CIRC	9650X			225	UNK	CIRC
0454X	5.33	4.18	UNK	UNK	CIRC	264DX			150	VC	CIRC		5652X	5.352	4.812	150	UNK	CIRC	9651X	5.669	4.109	225	UNK	CIRC
0455X	4.635	3.835	225	UNK	CIRC	2650X	6.516	4.916	UNK	UNK	CIRC		5653X	5.902	4.602	150	UNK	CIRC	9652X			225	UNK	CIRC
0456X	4.379	3.409	225	UNK	CIRC	2701Y	6.571	4.596	225	UNK	CIRC		565ZX			450	UNK	CIRC	9703X			150	UNK	CIRC
0501X	5.7	4.15	150	UNK	CIRC	2702X	6.958		225	UNK	CIRC		5702X			150	PF	CIRC	9704X			150	UNK	CIRC
0502X	5.61	3.99	150	UNK	CIRC	270DX			225	UNK	CIRC		5703X			150	PF	CIRC						
0503X	5.69	3.84	150	UNK	CIRC	271DX			225	UNK	CIRC		5750X	5.196	4.536	225	UNK	CIRC						
0504X	5.55	3.77	150	UNK	CIRC	2751X	6.938	6.288	225	UNK	CIRC		5751X	5.608	4.308	375	UNK	CIRC	*					
0505X	5.865	4.565	UNK	UNK	CIRC	2752X	7.156	5.876	150	UNK	CIRC		5752X	5.231		225	UNK	CIRC	*					
0506X	5.785	4.335	UNK	UNK	CIRC	2753X	6.541	5.301	225	UNK	CIRC		5752Y	5.231		150	UNK	CIRC	*					
0507X	5.525	4.205	150	UNK	CIRC	3401X	4.596	1.694	300	UNK	CIRC		575ZX			375	UNK	CIRC						
0508X			UNK	UNK	CIRC	3450X			450	UNK	CIRC		6301X	3.529	0.884	300	UNK	CIRC						
0509X			UNK	UNK	CIRC	3451X			450	UNK	CIRC		6401X			UNK	UNK	CIRC						
0550X	5.994		225	UNK	CIRC	3501X	6.489	4.413	225	UNK	CIRC		6402X			UNK	UNK	CIRC						
0551X	5.826	4.816	225	UNK	CIRC	3502X			UNK	VC	CIRC		6403X			UNK	UNK	CIRC						
0552X			150	UNK	CIRC	3503X			225	UNK	CIRC		640DX			UNK	UNK	CIRC						
0601X			UNK	UNK	CIRC	3550X	6.478	4.998	225	UNK	CIRC		6601X			UNK	UNK	CIRC						
0602X			100	VC	CIRC	3551Z	6.56	5.36	225	CO	CIRC	*	6603X			UNK	PF	CIRC						
060DX			100	VC	CIRC	3551Y	6.56	5.36	225	CO	CIRC	*	6604X			UNK	PF	CIRC						
061DX			100	VC	CIRC	3551X	6.56	5.36	300	CO	CIRC	*	6650X	3.474	2.604	225	UNK	CIRC						
063DX			UNK	UNK	CIRC	3552Y	6.564		450	UNK	CIRC		6651X	3.287	2.291	225	UNK	CIRC						
0650X	6.35	5.52	150	UNK	CIRC	3553Y	6.477		225	UNK	CIRC		6652X			375	UNK	CIRC						
0651X	5.81	5.2	150	UNK	CIRC	3554X	6.693		150	UNK	CIRC		7201X	3.017	1.493	UNK	UNK	CIRC						
0752X	7.642	6.382	225	UNK	CIRC	3555X			375	CO	CIRC		7301X	2.975	0.637	300	UNK	CIRC						
1301X	4.514	2.39	300	UNK	CIRC	3601X	6.431	5.294	150	UNK	CIRC		730LX			150	UNK	CIRC						
1302X	4.803	3.514	225	UNK	CIRC	3650Y	6.546		300	CO	CIRC	*	7401X	2.929	0.515	300	UNK	CIRC						
1401Y	5.443	2.649	300	PVC	CIRC	3650X	6.546	6.046	300	CO	CIRC	*	7501X	3.011		375	CI	CIRC						
1401Y	5.443		300	UNK	CIRC	3651X	6.364	5.544	225	UNK	CIRC		7502X	3.006	0.283	300	UNK	CIRC						
1401X	5.443	3.58	300	PVC	CIRC	3652X			375	CO	CIRC		7503X			UNK	UNK	CIRC						
1450X	5.445	4.195	UNK	UNK	CIRC	3653X	6.529		375	CO	CIRC		7504X			100	VC	CIRC						
1451X	5.095	3.875	225	UNK	CIRC	3654X			375	CO	CIRC		750DX			300	UNK	CIRC						
1501X	6.431	3.222	225	UNK	CIRC	3701X	6.495		225	UNK	CIRC		7601X			UNK	UNK	CIRC						
1502X	6.15		225	UNK	CIRC	3751X	6.517	5.867	150	UNK	CIRC		7602X			UNK	UNK	CIRC						
1503X	6.343	5.083	150	UNK	CIRC	3753X			300	CO	CIRC		7603X			UNK	UNK	CIRC						
1504X	6.047	2.972	300	UNK	CIRC	4301X			UNK	UNK	CIRC		7604X			UNK	UNK	CIRC						
1550X	6.391	5.741	225	UNK	CIRC	430DX			300	UNK	CIRC		7605X			UNK	UNK	CIRC						
1551X	5.993	4.683	225	UNK	CIRC	4401X	4.88	3.612	225	UNK	CIRC		7650X	3.005	2.245	225	UNK	CIRC						
1552X	6.366	5.156	225	UNK	CIRC	4402X			UNK	UNK	CIRC		7651X	3.082	2.182	375	UNK	CIRC						
1601X	6.849		225	UNK	CIRC	4450X	5.677	4.447	225	UNK	CIRC		7701X	3.017	0.384	UNK								

LINE STYLES / COLOURS	
Brown	— Foul
Red	— Foul Syphon Sewer
Red	— Foul Vacuum Main
Red	— Foul Rising Main
Red	— Combined
Red	— Combined Syphon Sewer
Red	— Combined Rising Main
Orange	— Lateral Drain
Orange	— Building Over Agreement Area
Dark Blue	— Treated Effluent
Purple	— Sludge
Purple	— Sewer Catchment
Purple	— Section 104 Area
Light Blue	— Surface Water
Light Blue	— Surface Water Rising Main
Yellow	— Private
Green	— Access Shaft
Green	— Decommissioned

MATERIALS	
AK	Alkathene
BAC	Bonded Asbestos Cement
BRC	Brick (Common)
BRE	Brick (Engineering)
CC	Concrete Box Culvert
CI	Cast Iron
CO	Concrete (In-Situ)
CP	Concrete (Pre-Cast)
CSB	Concrete Segments (bolted)
CSU	Concrete Segments (unbolted)
DI	Ductile Iron
GRC	Glass Reinforced Concrete
GRP	Glass Reinforced Plastic
MAC	Masonry in regular Courses
MAR	Masonry in random Courses
PE	Polyethylene
PF	Pitch Fibre
PP	Polypropylene
PVC	Polyvinyl Chloride
RPM	Reinforced Plastic Matrix
SI	Spun Iron
ST	Steel
VC	Vertical Clay
XXX	Other
ZZZ	Unknown

Manhole (SW)	Manhole (F&C)	Lamp hole (SW)	Lamp hole (F&C)	Pumping Station (SW)
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APPENDIX 7

APPENDIX 8

Bellamy Roberts				Page 1	
Clover House Western Lane Odiham Hants RG29 1TU			Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 1		
Date 23/04/2020 File 100YRS ST1 REV B.SRCX			Designed by JCB Checked by MT		
XP Solutions			Source Control 2017.1		
Summary of Results for 100 year Return Period (+40%)					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	5.054	0.554	2.7	15.5	O K
30 min Summer	5.200	0.700	3.0	19.6	O K
60 min Summer	5.298	0.798	3.2	22.3	O K
120 min Summer	5.339	0.839	3.2	23.5	O K
180 min Summer	5.326	0.826	3.2	23.1	O K
240 min Summer	5.295	0.795	3.1	22.3	O K
360 min Summer	5.226	0.726	3.0	20.3	O K
480 min Summer	5.163	0.663	2.9	18.6	O K
600 min Summer	5.106	0.606	2.8	17.0	O K
720 min Summer	5.055	0.555	2.7	15.5	O K
960 min Summer	4.970	0.470	2.5	13.1	O K
1440 min Summer	4.847	0.347	2.2	9.7	O K
2160 min Summer	4.734	0.234	1.8	6.6	O K
2880 min Summer	4.667	0.167	1.6	4.7	O K
4320 min Summer	4.595	0.095	1.2	2.7	O K
5760 min Summer	4.565	0.065	1.0	1.8	O K
7200 min Summer	4.555	0.055	0.9	1.5	O K
8640 min Summer	4.548	0.048	0.7	1.3	O K
10080 min Summer	4.544	0.044	0.7	1.2	O K
15 min Winter	5.125	0.625	2.8	17.5	O K
30 min Winter	5.295	0.795	3.1	22.3	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer	126.074	0.0	17.2	17	
30 min Summer	84.745	0.0	23.2	31	
60 min Summer	54.368	0.0	29.8	54	
120 min Summer	33.674	0.0	36.9	88	
180 min Summer	25.065	0.0	41.2	122	
240 min Summer	20.180	0.0	44.2	156	
360 min Summer	14.839	0.0	48.7	224	
480 min Summer	11.923	0.0	52.2	290	
600 min Summer	10.054	0.0	55.0	356	
720 min Summer	8.741	0.0	57.4	420	
960 min Summer	7.003	0.0	61.3	542	
1440 min Summer	5.114	0.0	67.2	782	
2160 min Summer	3.727	0.0	73.4	1144	
2880 min Summer	2.973	0.0	78.1	1500	
4320 min Summer	2.159	0.0	85.1	2208	
5760 min Summer	1.718	0.0	90.3	2936	
7200 min Summer	1.438	0.0	94.5	3672	
8640 min Summer	1.245	0.0	98.1	4400	
10080 min Summer	1.102	0.0	101.4	5112	
15 min Winter	126.074	0.0	19.3	17	
30 min Winter	84.745	0.0	26.0	31	
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Bellamy Roberts				Page 2	
Clover House Western Lane Odiham Hants RG29 1TU		Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 1			
Date 23/04/2020 File 100YRS ST1 REV B.SRCX		Designed by JCB Checked by MT			
XP Solutions		Source Control 2017.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	5.414	0.914	3.4	25.6	O K
120 min Winter	5.451	0.951	3.4	26.6	O K
180 min Winter	5.426	0.926	3.4	25.9	O K
240 min Winter	5.376	0.876	3.3	24.5	O K
360 min Winter	5.272	0.772	3.1	21.6	O K
480 min Winter	5.178	0.678	2.9	19.0	O K
600 min Winter	5.097	0.597	2.8	16.7	O K
720 min Winter	5.028	0.528	2.6	14.8	O K
960 min Winter	4.916	0.416	2.3	11.7	O K
1440 min Winter	4.772	0.272	1.9	7.6	O K
2160 min Winter	4.658	0.158	1.5	4.4	O K
2880 min Winter	4.600	0.100	1.2	2.8	O K
4320 min Winter	4.559	0.059	0.9	1.7	O K
5760 min Winter	4.548	0.048	0.7	1.3	O K
7200 min Winter	4.542	0.042	0.6	1.2	O K
8640 min Winter	4.539	0.039	0.5	1.1	O K
10080 min Winter	4.536	0.036	0.5	1.0	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
60 min Winter	54.368	0.0	33.3	58	
120 min Winter	33.674	0.0	41.3	92	
180 min Winter	25.065	0.0	46.1	132	
240 min Winter	20.180	0.0	49.5	168	
360 min Winter	14.839	0.0	54.6	240	
480 min Winter	11.923	0.0	58.5	308	
600 min Winter	10.054	0.0	61.6	374	
720 min Winter	8.741	0.0	64.3	440	
960 min Winter	7.003	0.0	68.7	566	
1440 min Winter	5.114	0.0	75.3	808	
2160 min Winter	3.727	0.0	82.3	1164	
2880 min Winter	2.973	0.0	87.5	1524	
4320 min Winter	2.159	0.0	95.3	2184	
5760 min Winter	1.718	0.0	101.1	2872	
7200 min Winter	1.438	0.0	105.8	3672	
8640 min Winter	1.245	0.0	109.9	4344	
10080 min Winter	1.102	0.0	113.5	4984	
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Bellamy Roberts		Page 3
Clover House Western Lane Odiham Hants RG29 1TU	Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 1	
Date 23/04/2020 File 100YRS ST1 REV B.SRCX	Designed by JCB Checked by MT	
XP Solutions	Source Control 2017.1	

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.200	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.073

Time (mins)		Area
From:	To:	(ha)
0	4	0.073

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Clover House Western Lane Odiham Hants RG29 1TU	Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 1	
Date 23/04/2020 File 100YRS ST1 REV B.SRCX	Designed by JCB Checked by MT	
XP Solutions Source Control 2017.1		

Model Details

Storage is Online Cover Level (m) 6.500

Tank or Pond Structure

Invert Level (m) 4.500

Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)
0.000	28.0	1.000	28.0	1.100	0.0	2.000	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SCU-0058-3500-1000-3500
Design Head (m)	1.000
Design Flow (l/s)	3.5
Flush-Flo™	Calculated
Objective	Linear discharge profile
Application	Surface
Sump Available	Yes
Diameter (mm)	58
Invert Level (m)	4.500
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.5
Flush-Flo™	0.078	1.2
Kick-Flo®	0.086	1.2
Mean Flow over Head Range	-	2.4

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.2	1.200	3.8	3.000	5.9	7.000	8.8
0.200	1.7	1.400	4.1	3.500	6.3	7.500	9.1
0.300	2.0	1.600	4.4	4.000	6.7	8.000	9.3
0.400	2.3	1.800	4.6	4.500	7.1	8.500	9.6
0.500	2.5	2.000	4.8	5.000	7.5	9.000	9.9
0.600	2.8	2.200	5.1	5.500	7.8	9.500	10.1
0.800	3.2	2.400	5.3	6.000	8.1		
1.000	3.5	2.600	5.5	6.500	8.5		

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Clover House Western Lane Odiham Hants RG29 1TU			Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 2		
Date 23/04/2020 File 100YRS ST2 REV B.SRCX			Designed by JCB Checked by MT		
XP Solutions			Source Control 2017.1		
Summary of Results for 100 year Return Period (+40%)					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	3.419	0.619	2.8	12.4	O K
30 min Summer	3.568	0.768	3.1	15.4	O K
60 min Summer	3.656	0.856	3.3	17.1	O K
120 min Summer	3.680	0.880	3.3	17.6	O K
180 min Summer	3.647	0.847	3.2	16.9	O K
240 min Summer	3.598	0.798	3.2	16.0	O K
360 min Summer	3.506	0.706	3.0	14.1	O K
480 min Summer	3.427	0.627	2.8	12.5	O K
600 min Summer	3.359	0.559	2.7	11.2	O K
720 min Summer	3.301	0.501	2.5	10.0	O K
960 min Summer	3.208	0.408	2.3	8.2	O K
1440 min Summer	3.083	0.283	2.0	5.7	O K
2160 min Summer	2.979	0.179	1.6	3.6	O K
2880 min Summer	2.922	0.122	1.3	2.4	O K
4320 min Summer	2.868	0.068	1.1	1.4	O K
5760 min Summer	2.854	0.054	0.8	1.1	O K
7200 min Summer	2.846	0.046	0.7	0.9	O K
8640 min Summer	2.842	0.042	0.6	0.8	O K
10080 min Summer	2.839	0.039	0.5	0.8	O K
15 min Winter	3.500	0.700	3.0	14.0	O K
30 min Winter	3.674	0.874	3.3	17.5	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer	126.074	0.0	14.2	17	
30 min Summer	84.745	0.0	19.1	31	
60 min Summer	54.368	0.0	24.5	50	
120 min Summer	33.674	0.0	30.3	84	
180 min Summer	25.065	0.0	33.8	118	
240 min Summer	20.180	0.0	36.3	152	
360 min Summer	14.839	0.0	40.1	218	
480 min Summer	11.923	0.0	42.9	282	
600 min Summer	10.054	0.0	45.2	346	
720 min Summer	8.741	0.0	47.2	406	
960 min Summer	7.003	0.0	50.4	530	
1440 min Summer	5.114	0.0	55.2	766	
2160 min Summer	3.727	0.0	60.4	1124	
2880 min Summer	2.973	0.0	64.2	1472	
4320 min Summer	2.159	0.0	69.9	2200	
5760 min Summer	1.718	0.0	74.2	2896	
7200 min Summer	1.438	0.0	77.6	3672	
8640 min Summer	1.245	0.0	80.7	4384	
10080 min Summer	1.102	0.0	83.3	5064	
15 min Winter	126.074	0.0	15.9	17	
30 min Winter	84.745	0.0	21.3	31	
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Clover House Western Lane Odiham Hants RG29 1TU		Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 2			
Date 23/04/2020 File 100YRS ST2 REV B.SRCX		Designed by JCB Checked by MT			
XP Solutions		Source Control 2017.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	3.775	0.975	3.5	19.5	O K
120 min Winter	3.790	0.990	3.5	19.8	O K
180 min Winter	3.733	0.933	3.4	18.7	O K
240 min Winter	3.658	0.858	3.3	17.2	O K
360 min Winter	3.522	0.722	3.0	14.4	O K
480 min Winter	3.410	0.610	2.8	12.2	O K
600 min Winter	3.318	0.518	2.6	10.4	O K
720 min Winter	3.243	0.443	2.4	8.9	O K
960 min Winter	3.131	0.331	2.1	6.6	O K
1440 min Winter	3.000	0.200	1.7	4.0	O K
2160 min Winter	2.908	0.108	1.3	2.2	O K
2880 min Winter	2.867	0.067	1.0	1.3	O K
4320 min Winter	2.849	0.049	0.8	1.0	O K
5760 min Winter	2.842	0.042	0.6	0.8	O K
7200 min Winter	2.837	0.037	0.5	0.7	O K
8640 min Winter	2.834	0.034	0.4	0.7	O K
10080 min Winter	2.832	0.032	0.4	0.6	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
60 min Winter	54.368	0.0	27.4	52	
120 min Winter	33.674	0.0	33.9	90	
180 min Winter	25.065	0.0	37.9	126	
240 min Winter	20.180	0.0	40.7	162	
360 min Winter	14.839	0.0	44.9	232	
480 min Winter	11.923	0.0	48.1	298	
600 min Winter	10.054	0.0	50.7	362	
720 min Winter	8.741	0.0	52.9	424	
960 min Winter	7.003	0.0	56.5	546	
1440 min Winter	5.114	0.0	61.9	780	
2160 min Winter	3.727	0.0	67.6	1128	
2880 min Winter	2.973	0.0	71.9	1468	
4320 min Winter	2.159	0.0	78.3	2204	
5760 min Winter	1.718	0.0	83.1	2936	
7200 min Winter	1.438	0.0	87.0	3544	
8640 min Winter	1.245	0.0	90.3	4392	
10080 min Winter	1.102	0.0	93.3	5096	
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Clover House Western Lane Odiham Hants RG29 1TU	Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 2	
Date 23/04/2020 File 100YRS ST2 REV B.SRCX	Designed by JCB Checked by MT	
XP Solutions	Source Control 2017.1	

Model Details

Storage is Online Cover Level (m) 4.800

Tank or Pond Structure

Invert Level (m) 2.800

Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)
0.000	20.0	1.000	20.0	1.100	0.0	2.000	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SCU-0058-3500-1000-3500
Design Head (m)	1.000
Design Flow (l/s)	3.5
Flush-Flo™	Calculated
Objective	Linear discharge profile
Application	Surface
Sump Available	Yes
Diameter (mm)	58
Invert Level (m)	2.800
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.5
Flush-Flo™	0.078	1.2
Kick-Flo®	0.086	1.2
Mean Flow over Head Range	-	2.4

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.2	1.200	3.8	3.000	5.9	7.000	8.8
0.200	1.7	1.400	4.1	3.500	6.3	7.500	9.1
0.300	2.0	1.600	4.4	4.000	6.7	8.000	9.3
0.400	2.3	1.800	4.6	4.500	7.1	8.500	9.6
0.500	2.5	2.000	4.8	5.000	7.5	9.000	9.9
0.600	2.8	2.200	5.1	5.500	7.8	9.500	10.1
0.800	3.2	2.400	5.3	6.000	8.1		
1.000	3.5	2.600	5.5	6.500	8.5		

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Clover House Western Lane Odiham Hants RG29 1TU		Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 3			
Date 23/04/2020 File 100YRS ST3.SRCX		Designed by JCB Checked by MT			
XP Solutions		Source Control 2017.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	2.528	0.528	3.0	26.4	O K
30 min Summer	2.681	0.681	3.3	34.1	O K
60 min Summer	2.806	0.806	3.6	40.3	O K
120 min Summer	2.870	0.870	3.7	43.5	O K
180 min Summer	2.878	0.878	3.8	43.9	O K
240 min Summer	2.867	0.867	3.7	43.3	O K
360 min Summer	2.827	0.827	3.7	41.3	O K
480 min Summer	2.781	0.781	3.6	39.1	O K
600 min Summer	2.736	0.736	3.5	36.8	O K
720 min Summer	2.694	0.694	3.4	34.7	O K
960 min Summer	2.618	0.618	3.2	30.9	O K
1440 min Summer	2.496	0.496	2.9	24.8	O K
2160 min Summer	2.369	0.369	2.5	18.4	O K
2880 min Summer	2.283	0.283	2.2	14.2	O K
4320 min Summer	2.180	0.180	1.8	9.0	O K
5760 min Summer	2.122	0.122	1.5	6.1	O K
7200 min Summer	2.084	0.084	1.4	4.2	O K
8640 min Summer	2.070	0.070	1.2	3.5	O K
10080 min Summer	2.062	0.062	1.1	3.1	O K
15 min Winter	2.594	0.594	3.1	29.7	O K
30 min Winter	2.770	0.770	3.5	38.5	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer	126.074	0.0	28.3	18	
30 min Summer	84.745	0.0	38.1	32	
60 min Summer	54.368	0.0	48.9	60	
120 min Summer	33.674	0.0	60.6	100	
180 min Summer	25.065	0.0	67.6	132	
240 min Summer	20.180	0.0	72.6	166	
360 min Summer	14.839	0.0	80.1	234	
480 min Summer	11.923	0.0	85.8	304	
600 min Summer	10.054	0.0	90.4	370	
720 min Summer	8.741	0.0	94.4	436	
960 min Summer	7.003	0.0	100.8	568	
1440 min Summer	5.114	0.0	110.4	820	
2160 min Summer	3.727	0.0	120.7	1188	
2880 min Summer	2.973	0.0	128.4	1532	
4320 min Summer	2.159	0.0	139.8	2248	
5760 min Summer	1.718	0.0	148.4	2952	
7200 min Summer	1.438	0.0	155.3	3672	
8640 min Summer	1.245	0.0	161.3	4400	
10080 min Summer	1.102	0.0	166.6	5136	
15 min Winter	126.074	0.0	31.7	18	
30 min Winter	84.745	0.0	42.6	32	
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Clover House Western Lane Odiham Hants RG29 1TU		Premier Coastal Properties Ltd Warblington Road, Emsworth Storage Tank 3			
Date 23/04/2020 File 100YRS ST3.SRCX		Designed by JCB Checked by MT			
XP Solutions		Source Control 2017.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	2.918	0.918	3.8	45.9	O K
120 min Winter	2.992	0.992	4.0	49.6	O K
180 min Winter	2.997	0.997	4.0	49.9	O K
240 min Winter	2.977	0.977	4.0	48.9	O K
360 min Winter	2.915	0.915	3.8	45.7	O K
480 min Winter	2.845	0.845	3.7	42.3	O K
600 min Winter	2.778	0.778	3.6	38.9	O K
720 min Winter	2.716	0.716	3.4	35.8	O K
960 min Winter	2.608	0.608	3.2	30.4	O K
1440 min Winter	2.446	0.446	2.8	22.3	O K
2160 min Winter	2.294	0.294	2.3	14.7	O K
2880 min Winter	2.204	0.204	1.9	10.2	O K
4320 min Winter	2.111	0.111	1.5	5.5	O K
5760 min Winter	2.070	0.070	1.2	3.5	O K
7200 min Winter	2.058	0.058	1.0	2.9	O K
8640 min Winter	2.052	0.052	0.9	2.6	O K
10080 min Winter	2.047	0.047	0.8	2.4	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
60 min Winter	54.368	0.0	54.8	60	
120 min Winter	33.674	0.0	67.9	112	
180 min Winter	25.065	0.0	75.8	140	
240 min Winter	20.180	0.0	81.3	178	
360 min Winter	14.839	0.0	89.7	254	
480 min Winter	11.923	0.0	96.1	326	
600 min Winter	10.054	0.0	101.3	398	
720 min Winter	8.741	0.0	105.7	464	
960 min Winter	7.003	0.0	112.9	598	
1440 min Winter	5.114	0.0	123.7	852	
2160 min Winter	3.727	0.0	135.2	1212	
2880 min Winter	2.973	0.0	143.8	1560	
4320 min Winter	2.159	0.0	156.6	2288	
5760 min Winter	1.718	0.0	166.2	2936	
7200 min Winter	1.438	0.0	173.9	3672	
8640 min Winter	1.245	0.0	180.7	4368	
10080 min Winter	1.102	0.0	186.6	5056	
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Date 23/04/2020 File 100YRS ST3.SRCX	Designed by JCB Checked by MT	
XP Solutions Source Control 2017.1		

Model Details

Storage is Online Cover Level (m) 4.000

Tank or Pond Structure

Invert Level (m) 2.000

Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)
0.000	50.0	1.000	50.0	1.100	0.0	2.000	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SCU-0062-4000-1000-4000
Design Head (m)	1.000
Design Flow (l/s)	4.0
Flush-Flo™	Calculated
Objective	Linear discharge profile
Application	Surface
Sump Available	Yes
Diameter (mm)	62
Invert Level (m)	2.000
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	4.0
Flush-Flo™	0.089	1.4
Kick-Flo®	0.093	1.4
Mean Flow over Head Range	-	2.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.4	1.200	4.4	3.000	6.7	7.000	10.0
0.200	1.9	1.400	4.7	3.500	7.2	7.500	10.4
0.300	2.3	1.600	5.0	4.000	7.7	8.000	10.7
0.400	2.6	1.800	5.3	4.500	8.1	8.500	11.0
0.500	2.9	2.000	5.5	5.000	8.5	9.000	11.3
0.600	3.2	2.200	5.8	5.500	8.9	9.500	11.6
0.800	3.6	2.400	6.0	6.000	9.3		
1.000	4.0	2.600	6.3	6.500	9.7		

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