Meridian Solar

Meridian Solar Farm

EN010169

Volume 6

Environmental Statement

6.3 ES Appendix 11-3: Flood Risk Assessment - Annex B

APFP Regulation 5(2)(a)

Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009

March 2026

Annex B - Hydraulic Modelling Reports

Postland Catchment Pump Failure Analysis

River Welland Hydraulic Modelling

South Holland Main Drain Modelling Report

Technical Note

AECOM

September 2025

Postland Catchment Pump Failure Analysis

Project	Meridian Solar Hydraulic Modelling
Project Number	WHS10217
Title	Postland Catchment Pump Failure Analysis
Description	This technical note details the pump failure analysis for the Postland catchment based on the existing 2023 2D hydraulic model of the Postland drainage catchment.
Prepared by	AJ *(Consultant)*
Reviewed by	SP *(Senior Consultant)*
Date	12th September 2025
Version	1.0

1 Introduction

1.1 Background and Scope

Wallingford HydroSolutions Ltd (WHS) has been commissioned by AECOM to undertake detailed hydraulic modelling to support the scheme design for the proposed Meridian Solar Farm (E: 527196, N: 313799), following agreement with the Environment Agency (EA). The modelling aims to define the flood levels for land parcels A, B, C and D to inform the elevation of flood-sensitive infrastructure.

As part of this work, three hydraulic models will be reviewed or built to assess realistic 'worst-case' flood risk scenarios. One of these is the Postland Pumped Catchment (PPC) model, which covers land parcels A, B and C of the site. This technical note presents the methodology and results from the pump failure scenario for the Postland drainage catchment, assuming the Internal Drainage Board (IDB) pumping station is offline throughout the design flood event.

1.2 Methodology

A data request for the PPC model[1] was submitted to the North Level District Internal Drainage Board (NLDIDB), and the model was received in June 2025. Its use for simulating a pump failure scenario for the Postland drainage catchment was agreed upon with both the NLDIDB and EA.

The PPC model is a 2D ESTRY-TUFLOW model and was updated and rerun to simulate the pump failure scenario during the design flood event. The analysis involved the following key tasks:

- Assessment of catchment hydrology to derive model inflows for the 0.1% AEP plus climate change design event, as this scenario was not previously included in the assessment undertaken on behalf of the IDB.
- Undertaking additional model runs with the pumps switched off (pump failure) for the entire duration of the design flood event.

The 0.1% AEP event, plus an allowance for climate change, was chosen as the design event to determine suitable levels for flood-sensitive infrastructure, in line with the South East Lincolnshire Strategic Flood Risk Assessment (SFRA)[2]. The climate change allowance has been applied following the EA's guidance[3] for using peak river flow allowances for flood risk assessments. A higher-end climate change allowance of 28% for the 2080s epoch was applied for the Welland Management Catchment, which the PPC falls within.

1.3 Data Sources

The data sources used to inform the pump failure analysis are as follows:

- Postland Pumped Catchment – Hydraulic Model[1]
- South East Lincolnshire Strategic Flood Risk Assessment (SFRA)[2]
- Climate change allowances for peak river flow[3]
- LiDAR Data[4] flown in 2022

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[1] Postland Pumped Catchment – Hydraulic Modelling. JBA Consulting. March 2024.
[2] South East Lincolnshire Strategic Flood Risk Assessment (SFRA). South East Lincolnshire Joint Strategic Planning Committee. March 2017
[3] Peak river flow climate change allowances by management catchment. Environment Agency. February 2022. Available at: https://environment.data.gov.uk/hydrology/climate-change-allowances/river-flow
[4] LiDAR Composite. Environment Agency. 2022. Available at: https://environment.data.gov.uk/survey

1.4 Assumptions

The hydrology for the PPC model does not need updating due to its relevant recency, and only the 0.1% AEP hydrograph must be multiplied by the relevant climate change factor.

2 Site Description

2.1 Site Location

Figure 1 shows the location and the land use for the proposed Meridian Solar Farm development in Lincolnshire, England, primarily within the South Holland District. The proposed solar farm development is spread across four main land parcels, labelled A, B, C and D. The site is located east of the River Welland, with the solar farm infrastructure situated between the towns of Spalding (to the northwest) and Holbeach (to the northeast).

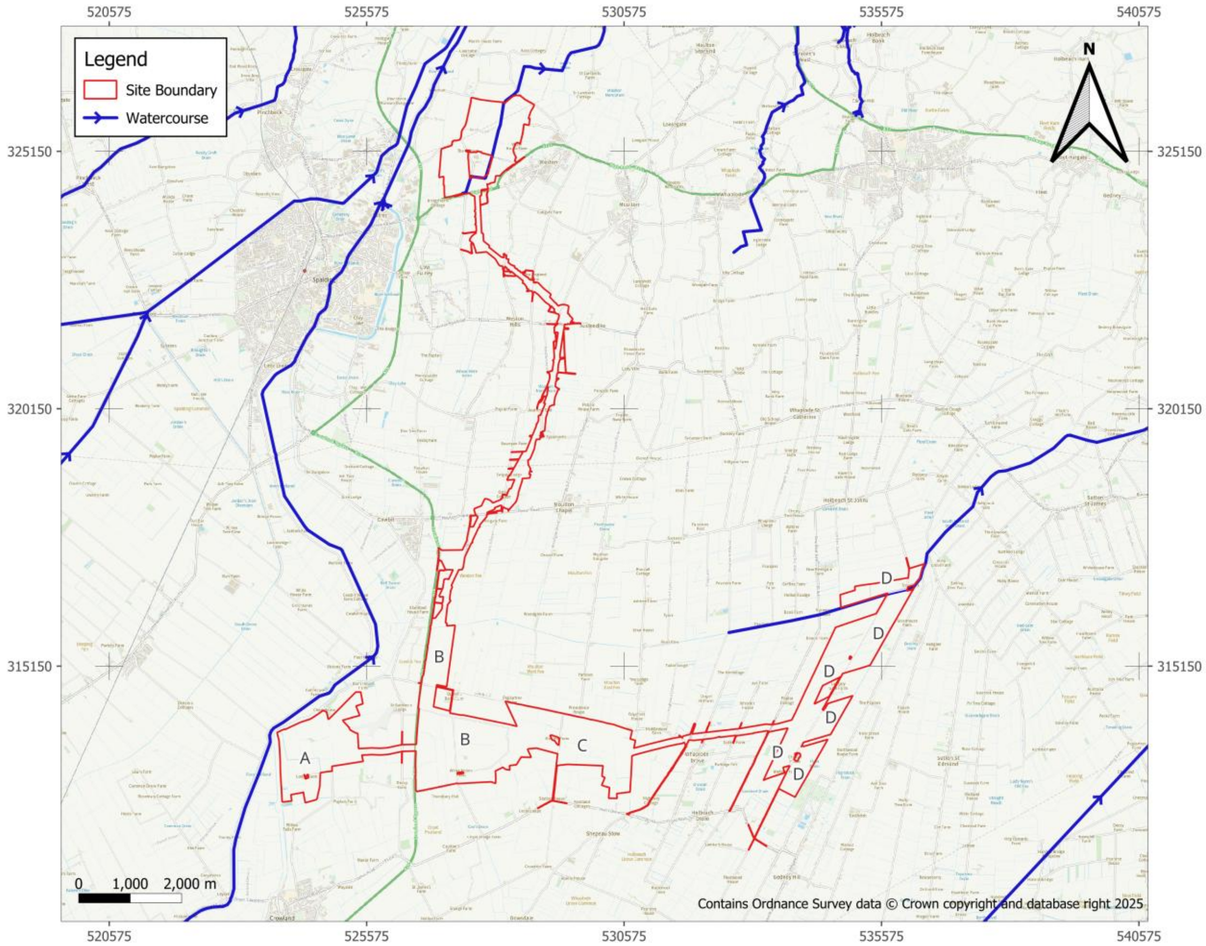


Figure 1 – Site Location

The Postland Pumped Catchment, which lies within the North Level District Internal Drainage Board (NLIDB) management area and contains land parcels A, B and C of the proposed development, is shown in Figure 2. It also shows the pumping station located at the north end (E: 523876, N: 313891) of the PPC. The pumping station consists of three 1.2 m^3/s rated pumps, which pump water from the drainage catchment into the elevated River Welland.

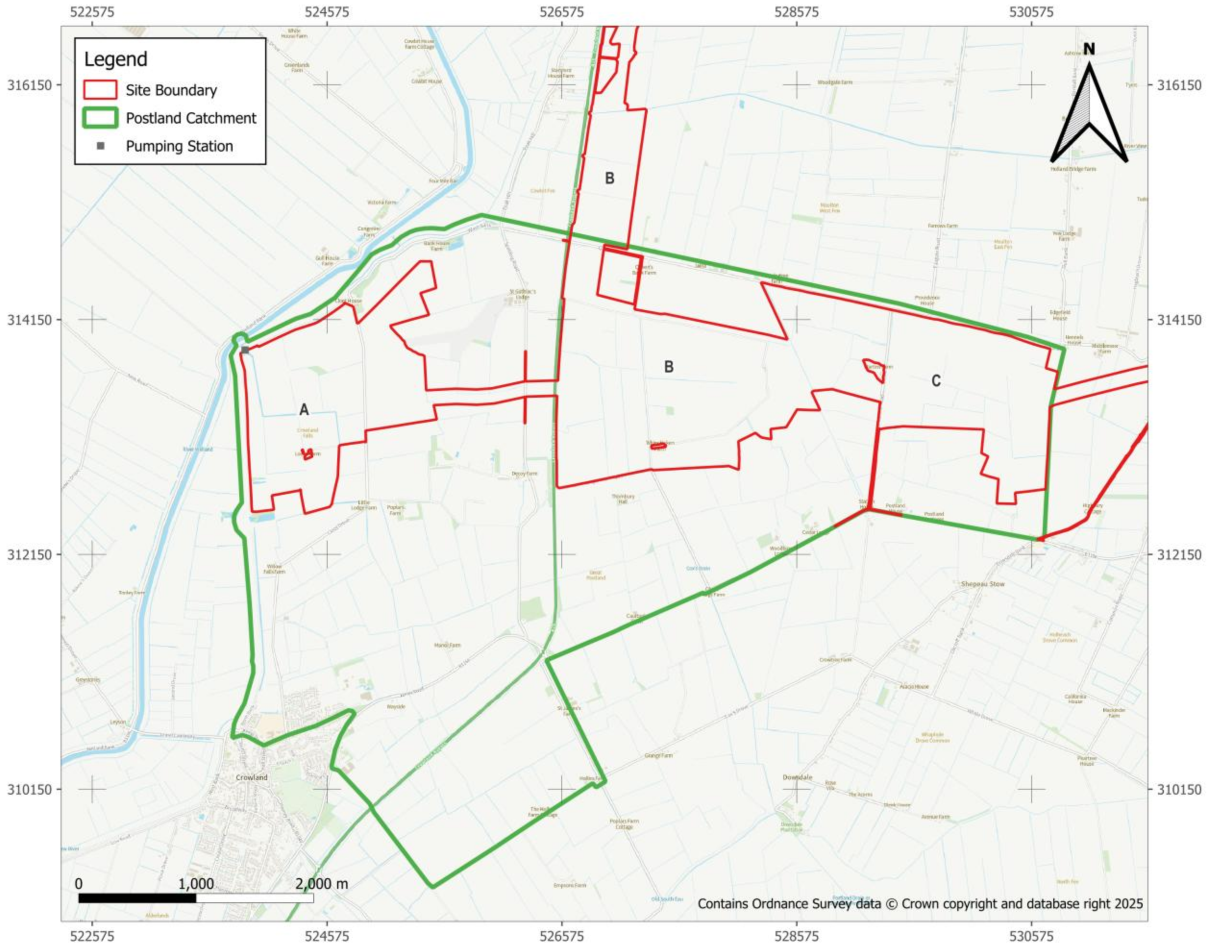


Figure 2 – Proposed Development within Parcels A, B and C

2.2 Site Topography

The topography of the site based on 1m national LiDAR data[4] is shown in Figure 3. The LiDAR shows the flat topography of the pumped catchment as ground levels within the site generally ranging between -1.0 and 2.0 m AOD. However, there are some localised high points throughout the site with levels above 2.0 m AOD.

Within Parcel A, the topography is predominantly low-lying with large areas of depressions (around -1.0 to 1.0 m AOD), particularly in the western and central sections. Therefore, this area is susceptible to ponding.

Ground levels in Parcel B are slightly more varied than in Parcel A, with areas reaching above 2.0 m AOD, especially towards the northern boundary of the parcel. As such, this parcel has better natural drainage than Parcel A, though large portions still lie below 2.0 m AOD.

Parcel C has generally higher elevations compared to A and B. Elevations are mostly above 1.0 m AOD, reaching up to 3.0 m AOD towards the eastern boundary of the parcel. Therefore, drainage in Parcel C is comparatively better with less potential for pooling.

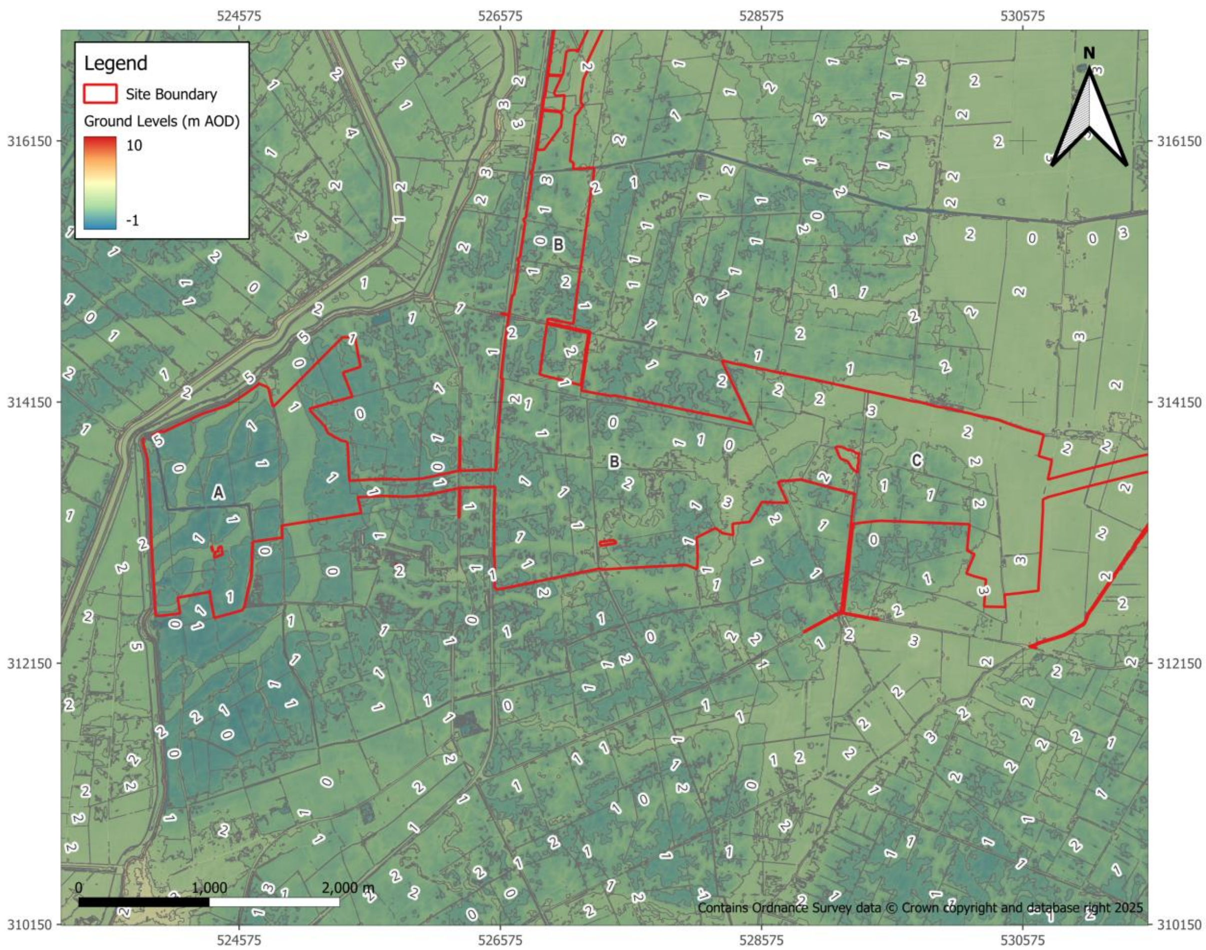


Figure 3 – Site Topography

3 Pump Failure Analysis

3.1 Model Overview

The PPC hydraulic model[1] is a 1D/2D ESTRY-TUFLOW model, where the NLIDIDB drainage network is represented in the 1D domain based on channel survey data, and the 2D domain represents the floodplain within the pumped catchment. The length of the modelled 1D network is 23.4 km, including 11 reaches with open channel cross sections and culverts and pumps as 1D structures. The 2D model domain covers an area of 23 km^2, i.e., the extent of the PPC.

The Postland PS consists of three individual pumps (1.2 m^3/s) with a cumulative capacity of 3.6m^3/s. The three pumps draw from the downstream node of the PPC, and the outfall of the pumps is into the River Welland. Their operation is controlled using a set of rules based on their pumping capacity and activation (when the pipe is half full) and deactivation (just above the bed level) thresholds. As outlined in section 1.2, the pump failure analysis assesses a worst-case scenario within the PPC, where the pumps are non-operational for the entire duration of the 0.1% AEP design flood event, including an allowance for climate change. Therefore, the model scenarios used for the analysis are:

- 0.1% AEP fluvial event with climate change:
  This scenario represents the baseline conditions within the pumped catchment during a 0.1% AEP fluvial event, with a 28% uplift applied to account for the peak river flow climate change allowance. In this scenario, the pumps are operational throughout the design event.
- The 'PUMP_OFF' scenario:
  This scenario simulates a failure at the pumping station during the 0.1% AEP + climate change fluvial event. All three pumps are switched off to represent pump failure. To model this, the activation and deactivation thresholds for the pumps have been set above the maximum flood level within the pipe, ensuring that pumping will not be triggered during the design event.
- Post-development scenario:
  This scenario simulates a failure at the pumping station during the 0.1% AEP + climate change fluvial event with the proposed solar farm infrastructure in place. Similar to the PUMP-OFF scenario, all three pumps are switched off to represent pump failure, and the proposed infrastructure has been incorporated into the model.

3.2 Model Updates

Deriving 0.1% AEP + climate change event

Model updates were undertaken for the PPC model to enable the additional runs required to establish baseline conditions for the design event and simulate the worst-case pump failure scenario. The updates were necessary as the 0.1% AEP + climate change event was not included in the previous assessment undertaken on behalf of the IDB.

For the model update, it was assumed that the hydrology within the PPC model remained valid due to its recency. Therefore, the hydrograph for the design event was derived by applying an appropriate climate change allowance to the existing 0.1% AEP hydrograph. In line with the EA's guidance[3], a higher-end climate change allowance of 28% was applied for the 2080s epoch, corresponding to the Welland Management Catchment, within which the PPC is located. The hydrographs for the existing 0.1% AEP event and the design 0.1% AEP + climate change event are shown in Figure 4.

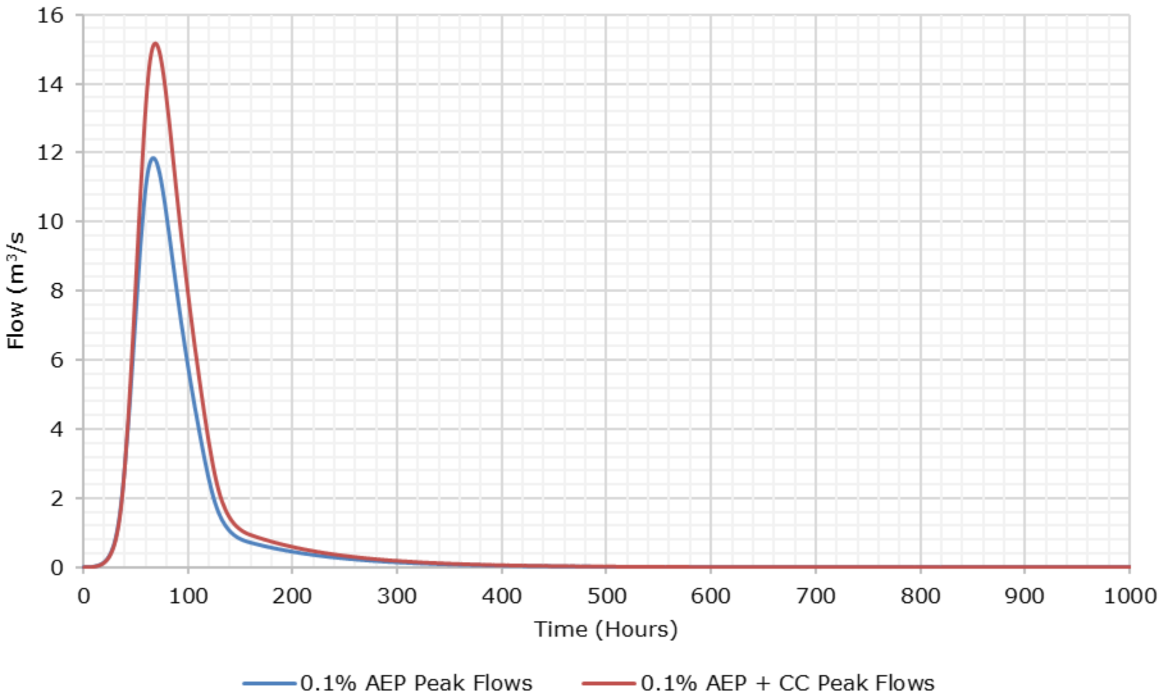


Figure 4 – Hydrographs for the 0.1% AEP and 0.1% AEP + CC events

3.3 Post-development Model Build

3.3.1 Post-development Model Overview

Figure 5 shows the sections of the proposed development located within Parcels A, B and C. These were incorporated into the model to create the post-development scenario to determine the impacts on flood risk as a result of the proposed development during a pump failure event. The post-development infrastructure incorporated into the model includes the following:

- Solar Tables
- Solar Stations
- Substations

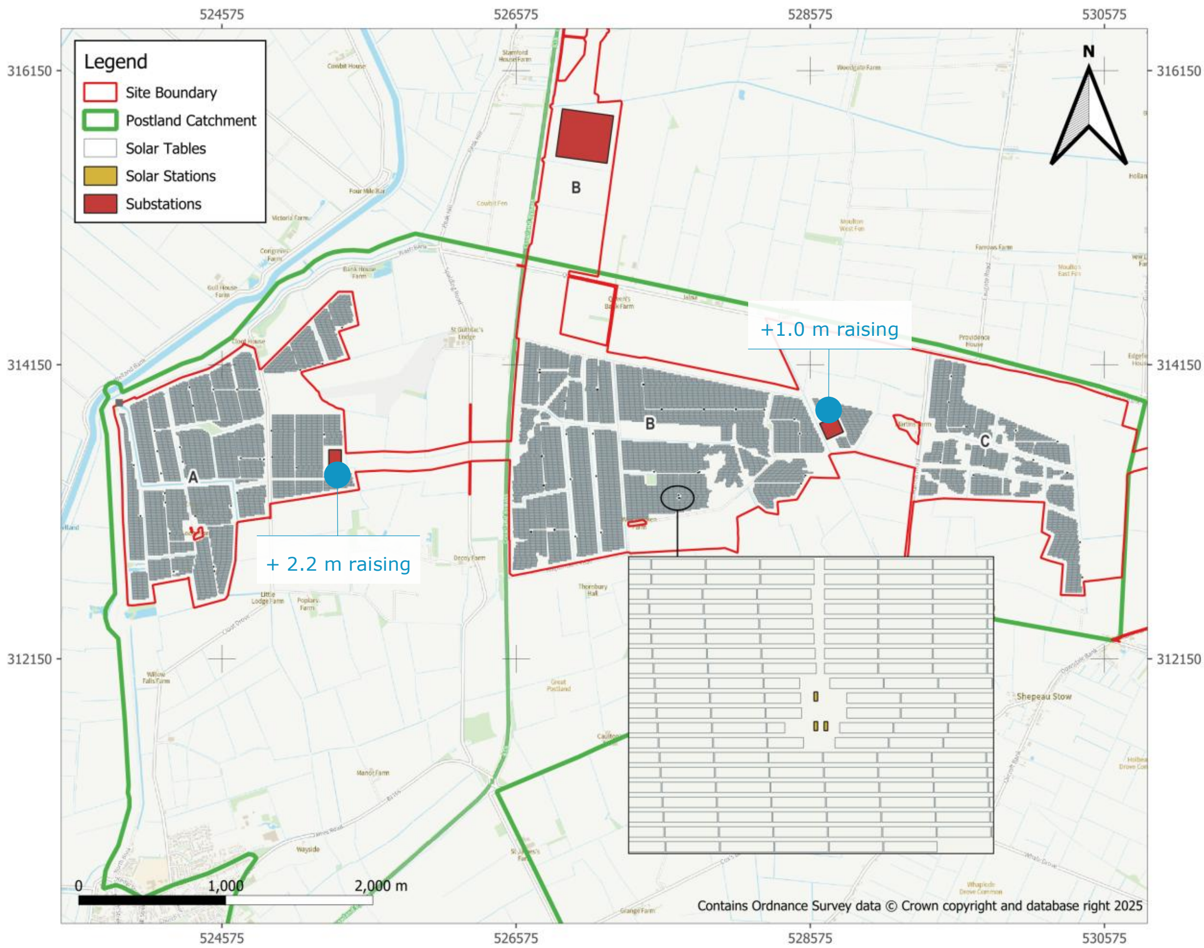


Figure 5 – Proposed infrastructure within land parcels A, B and C (i.e., in the Postland Pumped Catchment)

3.3.2 Modifications for the Post-Development Scenario

Solar Tables

The solar tables will be raised on supports above ground level, with the lower edge of the panels positioned at least 0.8 m above the ground. To assess the impact that the proposed Solar Tables will have on flood risk they have been included in the model as form loss coefficients, based on the proposed dimensions and number of support legs. Therefore, the supports for the panels have been represented using a layered flow constriction layer (2d_lfcsh) to assess their impact on flow routes within the PPC. Though consideration was given to modifying the underlying DTM to explicitly represent each support, this approach was deemed inappropriate due to the small size of the supports relative to the model grid resolution.

To represent the supports as a flow constriction layer, two key attributes are required – the Form Loss Coefficient (FLC) and Blockage Fraction (J) associated with the supports. The FLC used for the post-development scenario was derived using the method for applying losses from Hydraulics of Bridge and Waterways[5], which uses the width of the supports normal to the flow. The procedure set out in Hydraulics of Bridge and Waterways applied to derive the FLC is outlined below:

1. The average width of the supports was taken as 135 mm, as the width of the supports throughout the development will range between 80-150 mm.
2. The flow width was calculated for each support by assessing the flow area blocked on a span-by-span approach. For this approach, the flow width from mid-support to mid-support was calculated using an approximate distance of 3 m between the supports.
3. The average width of the supports and the flow width were used to calculate the blockage fraction (J value) for each support.
4. Using the Hydraulics of Bridge and Waterways chart (Figure 6) and assuming a single rounded shape support, the FLC (ΔKp) was determined.

Table 1 – Parameters used to derive FLC

Parameter	Description	Value
b	Flow width assessed on a span-by-span approach	3.000 m
W_p	Support with normal to flow	0.135 m
J	Blockage fraction (b/W_p)	0.045 (4.5%)
Δkp	FLC (from graph based on J value)	0.051

[5] Hydraulics of Bridge and Waterways. Hydraulics Design Series No. 1. Bradley, J. 1978.

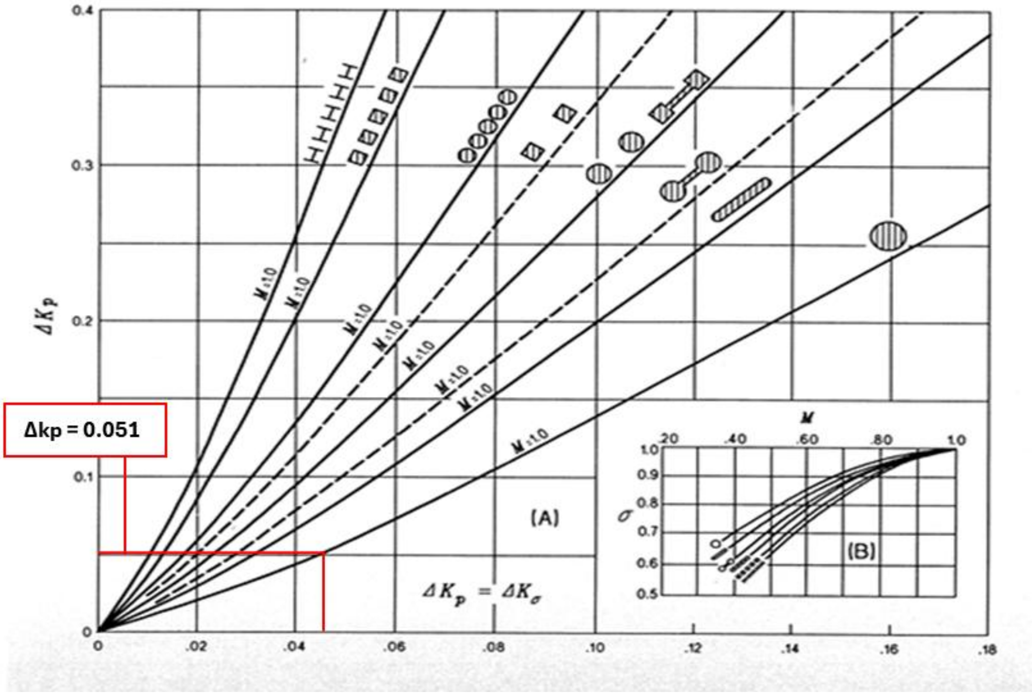


Figure 6 – Method to derive FLC, Hydraulics of Bridge and Waterways[5]

Several limitations were encountered when deriving the FLC, which related to the proximity of the rows for the supports for each solar table, as TuFLOW does not allow multiple FLC lines within the same model cell. Therefore, a single FLC line was used for each solar table within the 2d_lfcsh layer, and the Δkp value was taken as 0.102, i.e., the combined Δkp value for both rows of supports.

The layered flow constriction layer (2d_lfcsh) layer was used to allow losses and blockages to be varied with water depth. This allowed:

- Layer 1 to represent the supports beneath the panels, where the supports are 0.8 m above the ground (L1_Obvert), with the 4.5% Blockage Fraction and 0.102 combined Δkp applied.
- Layer 2 to represent the inclined solar at a height of 3.5 m (L2_Depth), i.e., 4.3 m above ground level, including the 0.8 m height of the supports. This layer will be 100% blocked and has an increased Δkp of 1.56 due to the additional energy losses associated with the flow surcharging the panels. The Δkp value of 1.56 was adopted from the approach used for modelling bridge decks and represents form loss when the flood level is above the soffit level of the bridge, i.e when it is surcharged.

Solar Stations

The solar stations have been represented in the post-development scenario by a direct modification of the underlying DTM using a z-shape layer (2d_zsh). The z-shape layer was used to define the area for the bunds around the solar station, and the terrain elevations in the 2D domain were modified based on this defined area.

A Z value of 2.3 m and the 'ADD' shape option were used for the z-shape layer, which instructed TuFLOW to increase the ground levels within the defined area by 2.3 m. This created discrete structures, representing the worst-case bund level around the solar stations. The z-shape features were produced by buffering the solar stations by 20 m, providing the flood protection zone for the worst-case flood depths.

Substations

Similar to the solar stations, bunds for the substations were represented using a z-shape layer. The area for the layer was informed using the footprint of the substations provided in the masterplan for the development[6]. The levels of the bunds for the substations are outlined below:

- 400 kV substation and BESS Compound: 1.7 m above ground
- 132 kV substation on Parcel A: 2.2 m above ground
- 132 kV substation on Parcel B: 1 m above ground
- 132 kV substation on Parcel D: no bund

[6] Meridian Illustrative Site Layout, Constraints and Layout Overview. Downing. Dwg. No. DOW-2025-01-DCO.

3.4 Model Results

3.4.1 Baseline Scenario

Figure 7 shows the modelled maximum flood depths for the 0.1% AEP plus climate change design event for the baseline scenario when the pumps are operational. The modelled flood extent indicates flooding is concentrated in lower-lying areas throughout the site, with localised pooling mainly in Parcels A and B, based on the topography of these parcels as outlined in section 2.2. Flood depths resulting from the design event generally remain below 0.5 m, with some deeper areas (up to approximately 1.2 m) near drains and low-lying topography.

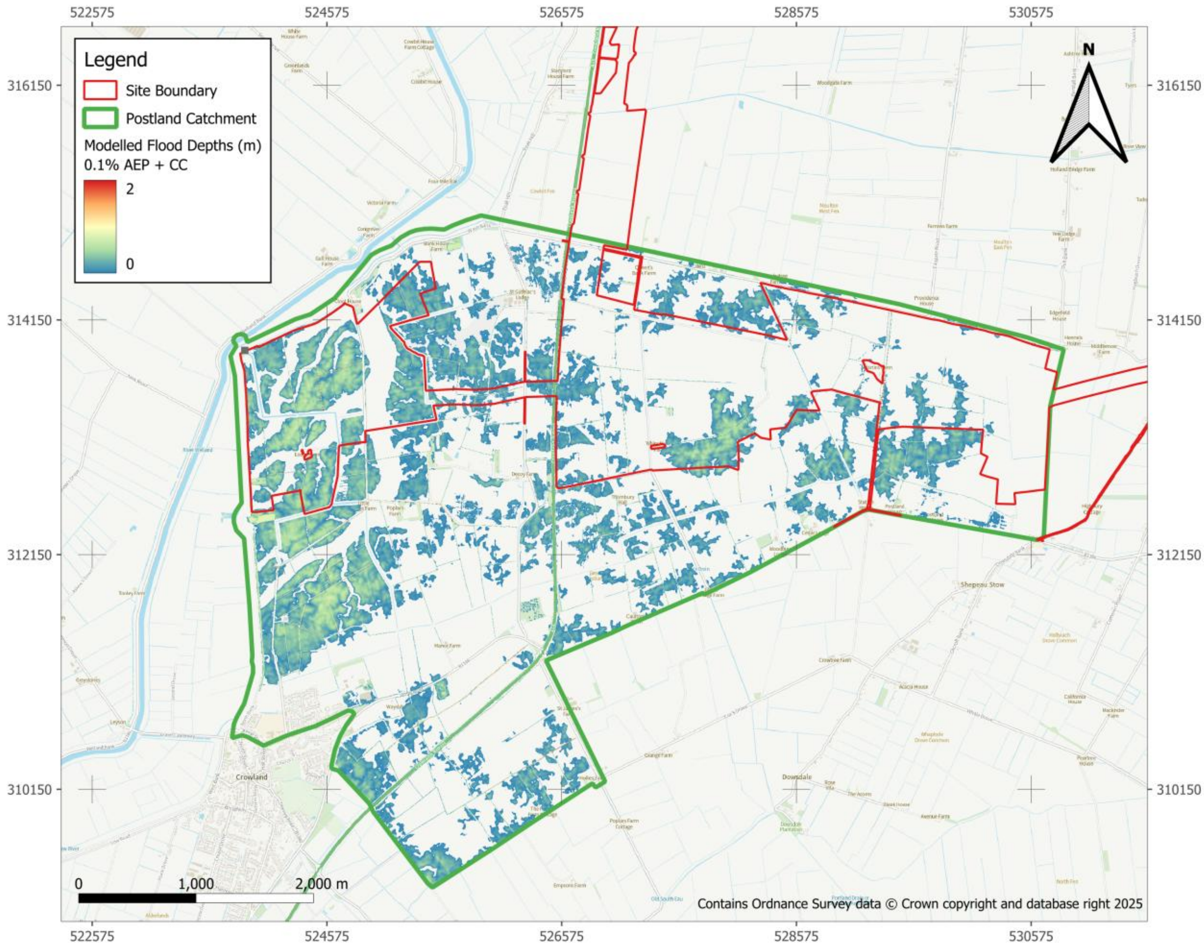


Figure 7 – Baseline maximum modelled flood depths for design event (0.1% AEP + CC event)

3.4.2 Pump Failure Scenario

The modelled flood depths and extent for the pump failure scenario are shown in Figure 8. The model results indicate that flooding is extensive and significantly deeper in Parcel A (west of the site), with flood depths ranging between 1.0 m and 2.0 m. This indicates there is a significant flood risk in Parcel A under the pump failure scenario during the design event. The flooding appears to follow the IDB drainage features and low-lying zones.

For Parcel B (the central zone), flood depths are shallow to moderate, mostly being < 1.0 m. Some areas along the southern (extending to the centre of the parcel) and northern boundaries of this parcel show isolated deeper areas of pooling, corresponding to local topographic depressions.

Parcel C (Eastern Zone) is the least affected by flooding, with sparse and shallow flood areas of pooling (typically < 0.5m). Flooding in this area is discontinuous, and as such, is the most favourable for locating sensitive infrastructure due to the lower flood risk.

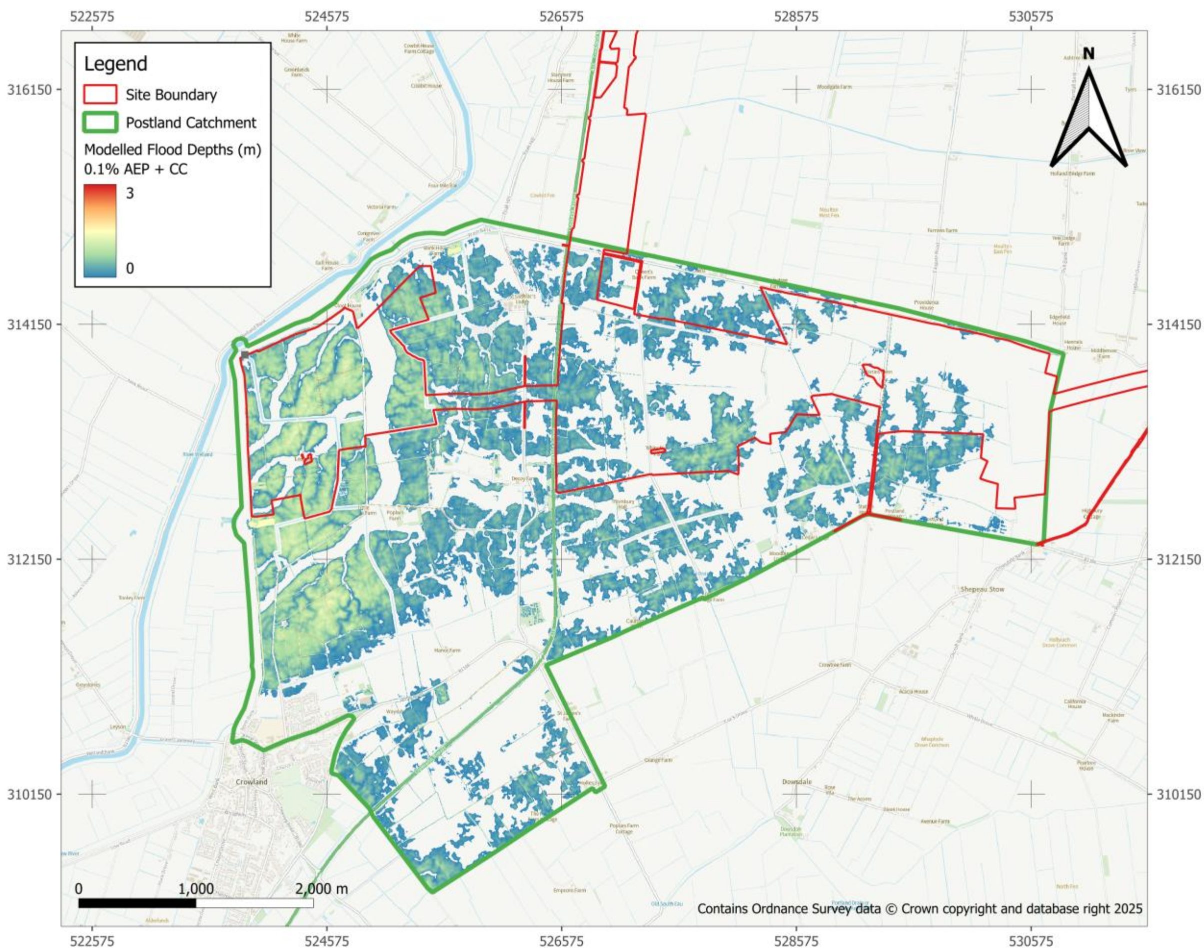


Figure 8 – Pump failure maximum modelled flood depths for design event (0.1% AEP + CC event)

3.4.3 Post-development Scenario (With Pump Failure)

The flood depths and extent from the post-development scenario are shown in Figure 9. The results indicate similar risks of flooding across the development area as in the pump failure scenario, which is confirmed by the depth change analysis outlined in section 0.

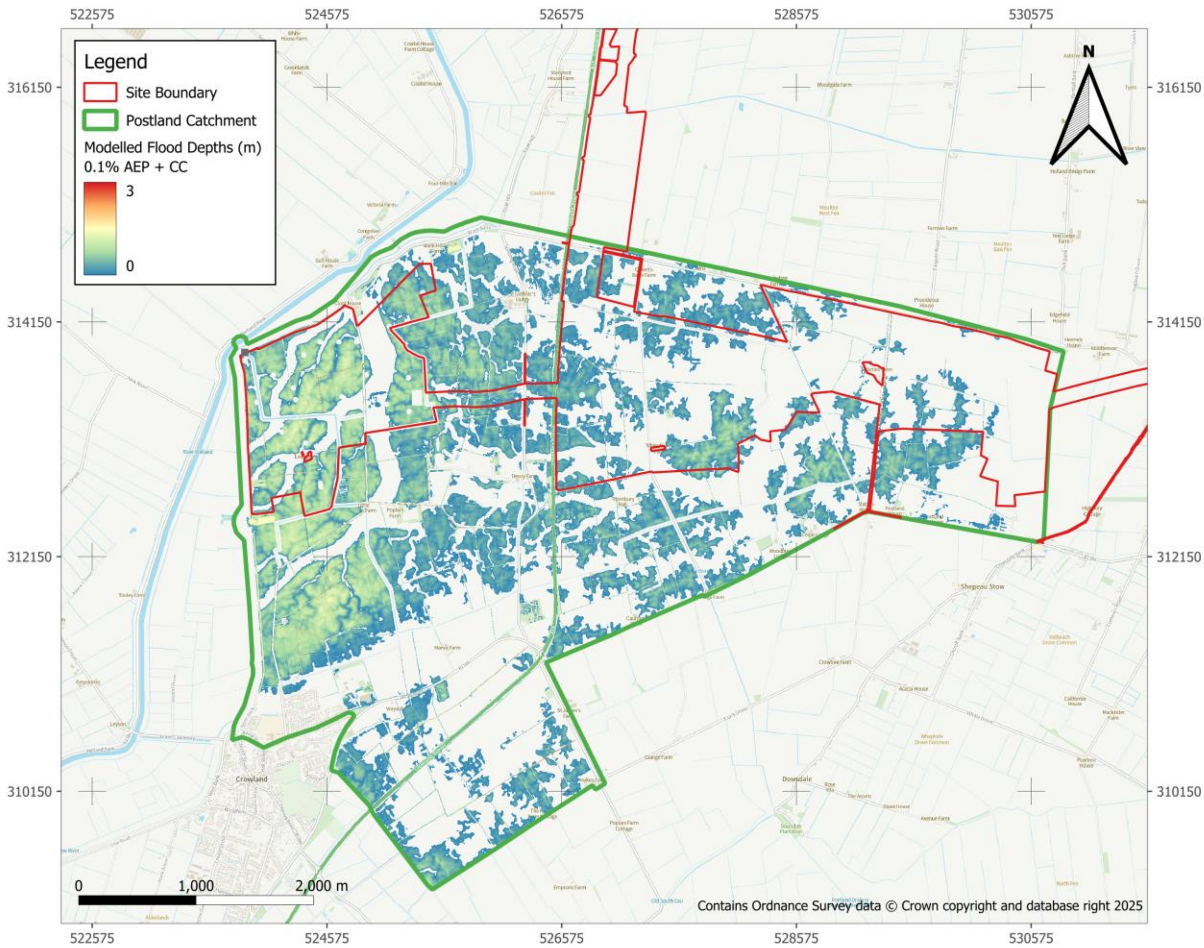


Figure 9 – Post-development model results

3.4.4 Depth Change Analysis

A depth change analysis was undertaken between the pump failure and post-development scenarios to assess whether the proposed development results in any increased third-party impacts. The results mostly indicate negligible changes in flood depths across the PPC due to the development, see Figure 10. However, there are some small increases in flood levels in isolated areas of up to 560 mm.

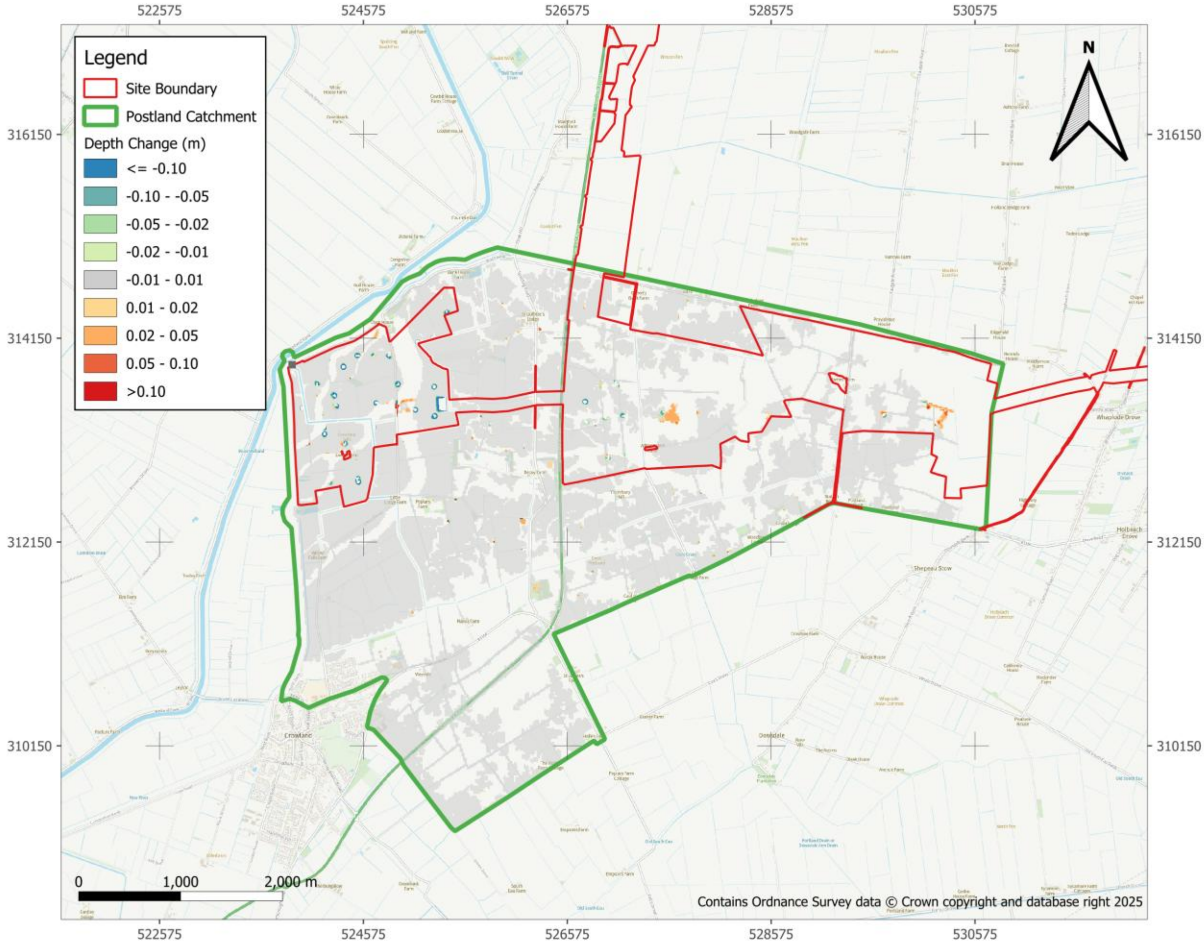


Figure 10 – Depth Change Analysis

3.5 Design Flood Level

The design flood level for each parcel should be determined using the maximum modelled flood level within each land parcel, based on the worst-case event (i.e., pump failure throughout the design flood event), with the addition of a 300 mm freeboard.

Due to the significant variability in flood levels across the site, maximum flood depths were extracted at sample points at 200 m intervals across the area of the parcels for the 0.1% AEP pump failure scenario. The sample flood depths are provided in Appendix 1alongside shapefiles of all the modelled flood extents (baseline and pump failure).

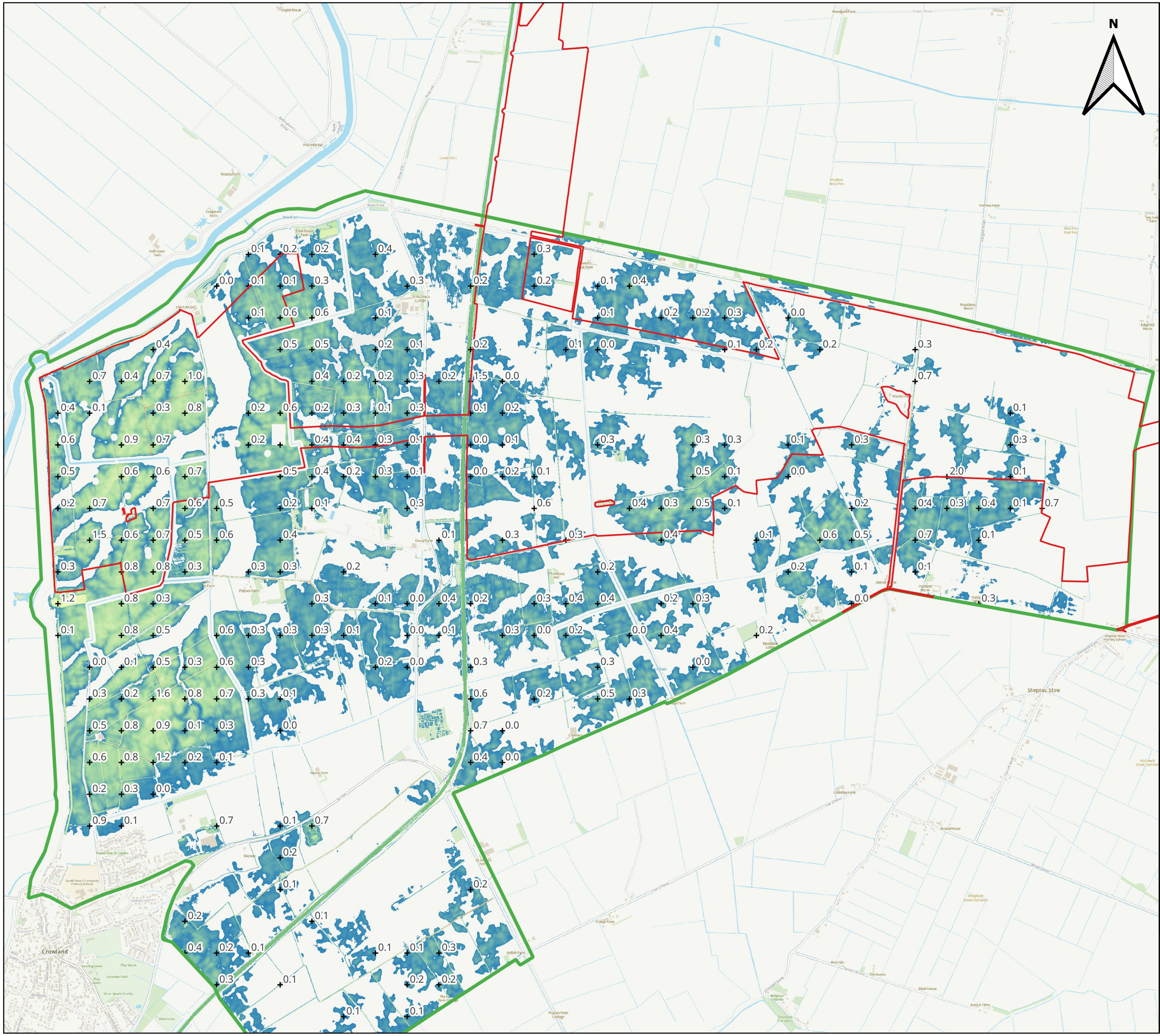
4 Conclusions

The Postland Pumped Catchment (PPC) model, which covers land parcels A, B and C, was updated to assess the 'worst-case' flood risk scenario where the IDB pumping station failed during the design 0.1% AEP plus climate change flood event.

For the model update, the hydrograph for the design event was derived by applying a 28% climate change allowance (as per EA's Guidance for the Welland Catchment) to the existing 0.1% AEP hydrograph.

Results from the pump failure scenario indicated that the development is vulnerable to flooding, particularly within areas critical for infrastructure placement (Parcels A, B and C). Consequently, it is recommended that all infrastructure be designed to be above the maximum modelled flood level, with sufficient freeboard.

Appendix 1 – Sampled Maximum Flood Depths



Project :

Meridian Solar Farm

Client :

AECOM

Legend :

- Site Boundary
- Postland Catchment
- + Sampled Max. Flood Depths (m)

0.1% AEP + CC Modelled Flood Depths (m)

3

0

Scale :

0 500 1,000 m

Title : 0.1% AEP Flood Event + CC Modelled Flood Depths (m) for Post-development Pump Failure Scenario

Drawing :	Rev :
WHS10217-T01-0001	3

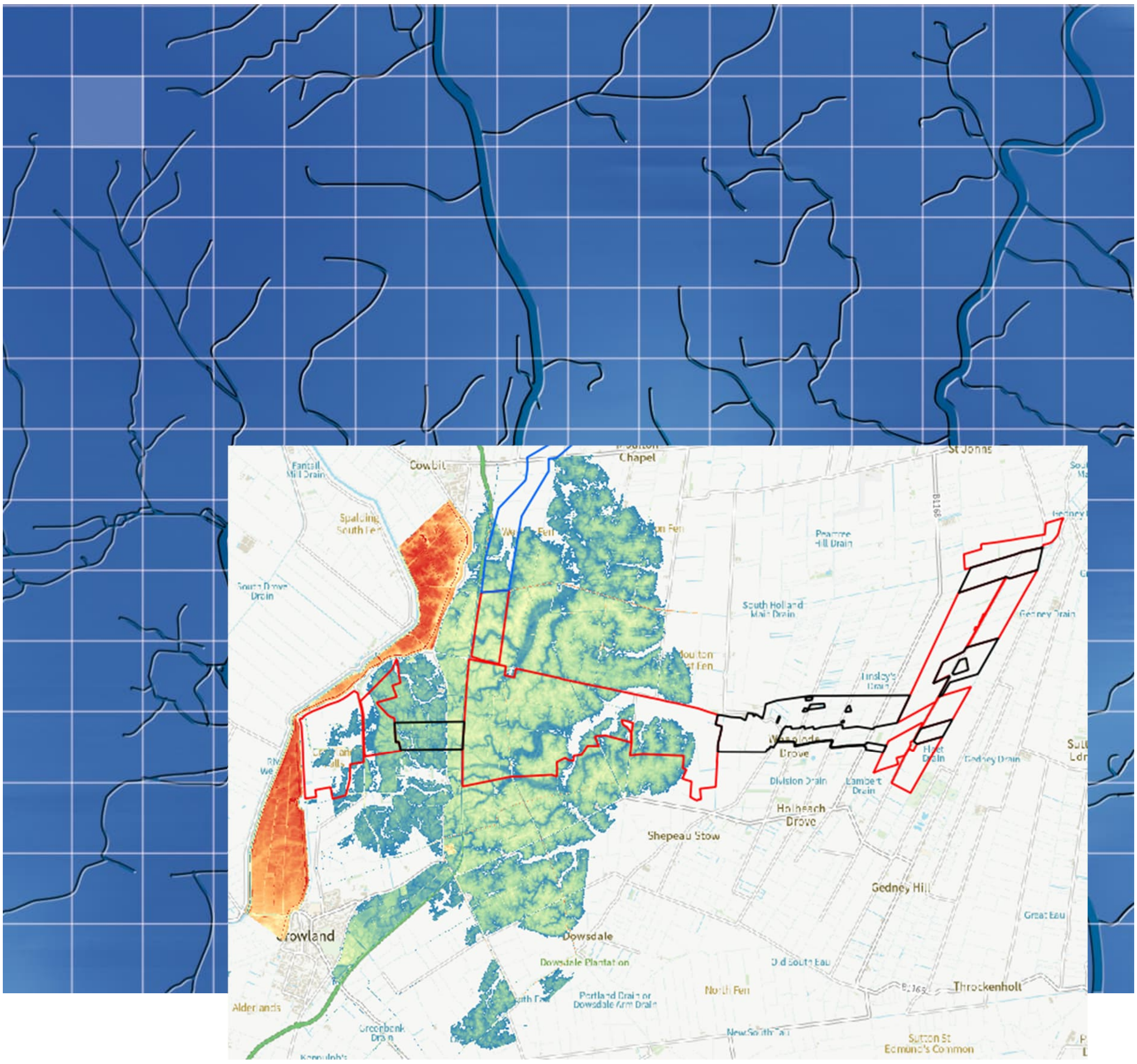
WHS

AECOM

September 2025

River Welland Hydraulic Modelling

Modelling Methodology Report



WHS

AECOM

River Welland Breach Hydraulic Modelling

Document issue details

WHS10217

Version	Issue date	Issue status	Prepared By	Approved By
1.0	15th August 2025	Final	JL (*Graduate Consultant*)	SP (*Senior Consultant*)

For and on behalf of Wallingford HydroSolutions Ltd.

The WHS Quality & Environmental Management system is certified as meeting the requirements of ISO 9001:2015 and ISO 14001:2015 providing environmental consultancy (including monitoring and surveying), the development of hydrological software and associated training.

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Contents

1 Introduction

1.1 Scope

Wallingford HydroSolutions (WHS) has been commissioned by AECOM to complete a bespoke hydraulic model of the River Welland to assess breach scenarios and their impact upon the proposed Meridian Solar Farm (E: 527196, N: 313799).

Breach modelling is required to confirm and quantify the residual risk to the proposed development, which currently benefits from a reduction in flooding due to existing flood defences. These existing flood defences consist of both an inner raised embankment (EA Asset ID: 510051), a Water Storage Area (EA Asset ID: 343253) and an outer raised Embankment (EA Asset IDs: 124918, 503744, 503746, 503748, 503750, 124179, 503832).

1.2 Methodology

An existing 1D only hydraulic model is available for the River Welland, which has been built using MIKE 11. As MIKE 11 is not commonly used in the UK, its outputs have been extracted and used as inputs into a TUFLOW 2D breach model. This is achieved by using the modelled flood levels from the MIKE 11 model and using them to set a Head-Time (HT) boundary in the new TULOW model along the centreline of the River Welland. Ground levels for the model have been informed by LiDAR published in 2022.

The model has been run for the 1.0% annual exceedance probability (AEP) plus climate change (CC) and the 0.1% AEP plus CC scenarios. The climate change factors for fluvial flows have been updated to be in line with the latest EA allowances based on the 2080s climate change epoch and the higher estimates have been used because the scheme is classed as nationally significant infrastructure projects.

1.3 Data sources

The data sources used to inform this hydraulic modelling study include:

- River Welland Hydraulic Model[1]
- LIT 56413 Breach of Defence Guidance[2]
- LiDAR data from the National LiDAR Program.[3]
- Flood risk assessments: climate change allowances[4]
- Ordinance Survey - OpenMap - Local

[1] Environment Agency (2016). *River Welland Hydraulic Modelling Report.*
[2] Environment Agency (2021). *Breach of Defence Guidance.* LIT 56413
[3] Environment Agency LIDAR Composite Digital Terrain Model (DTM) 1m, 2022 Available at: https://environment.data.gov.uk/dataset/13787b9a-26a4-4775-8523-806d13af58fc
[4] Gov.uk (2022) Flood risk assessments: climate change allowances. Available at: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

2 Site Description

The proposed development site covers PV Areas, Inter-Array Areas and the Grid Connection Corridor, located between the villages of Crowland and Cowbit in Lincolnshire (E: 527196, N: 313799). The PV Areas comprise approximately 1100ha, the Inter-Array Areas combined are approximately 303ha, and the Grid Connection Corridor is approximately 718ha. Current land use at the site is agricultural. The locations of the land parcels relative to the River Welland are shown in Figure 1 below.

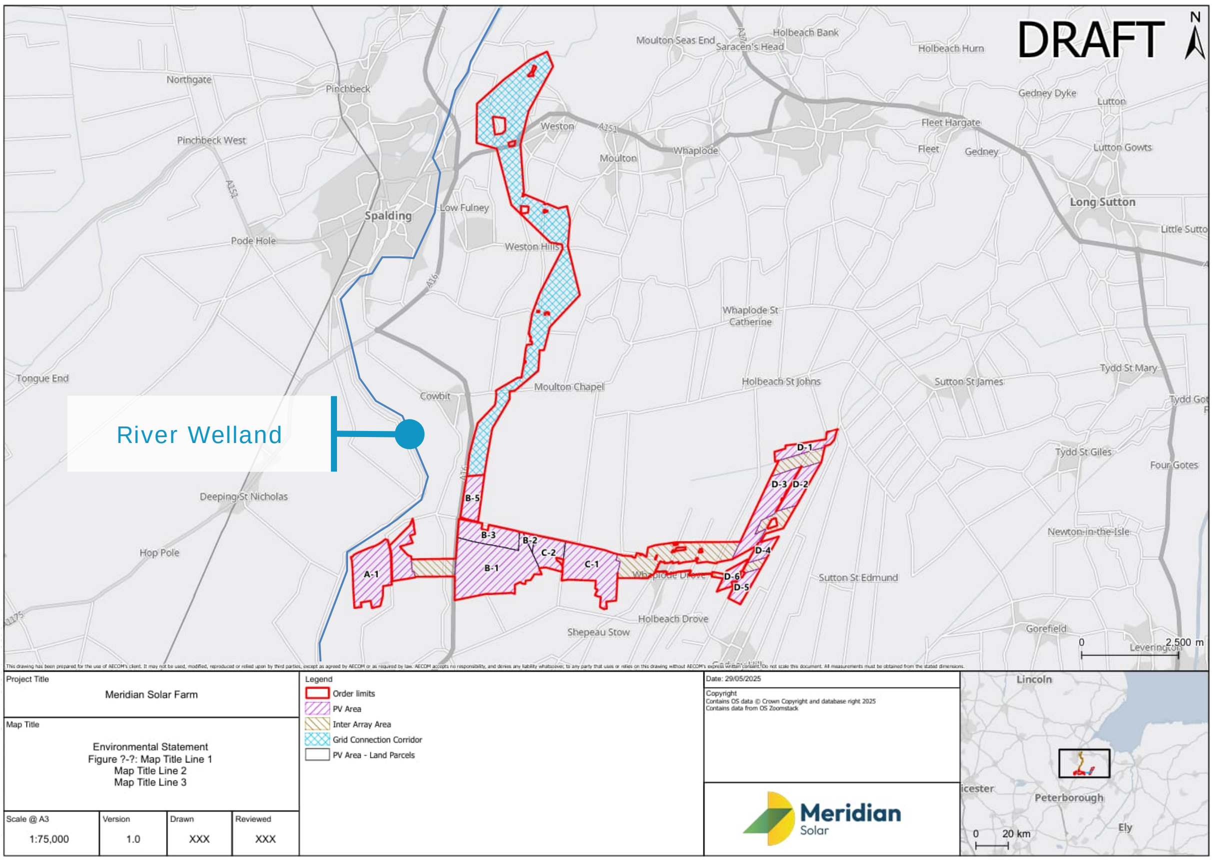

Figure 1 - Proposed Location of the Development

3 Model Build (2D domain)

3.1 Software Version

The 2D domain is modelled using TUFLOW's 2025 release, which runs on a Graphic Processing Unit (GPU) using the Heavily Parallelised Compute (HPC) solver.

3.2 Model Extent

The 2D domain extends from the centreline of the River Welland and covers the area to the east of the river, encompassing the area of the PV array that is at risk of flooding during a breach event, shown in Figure 2.

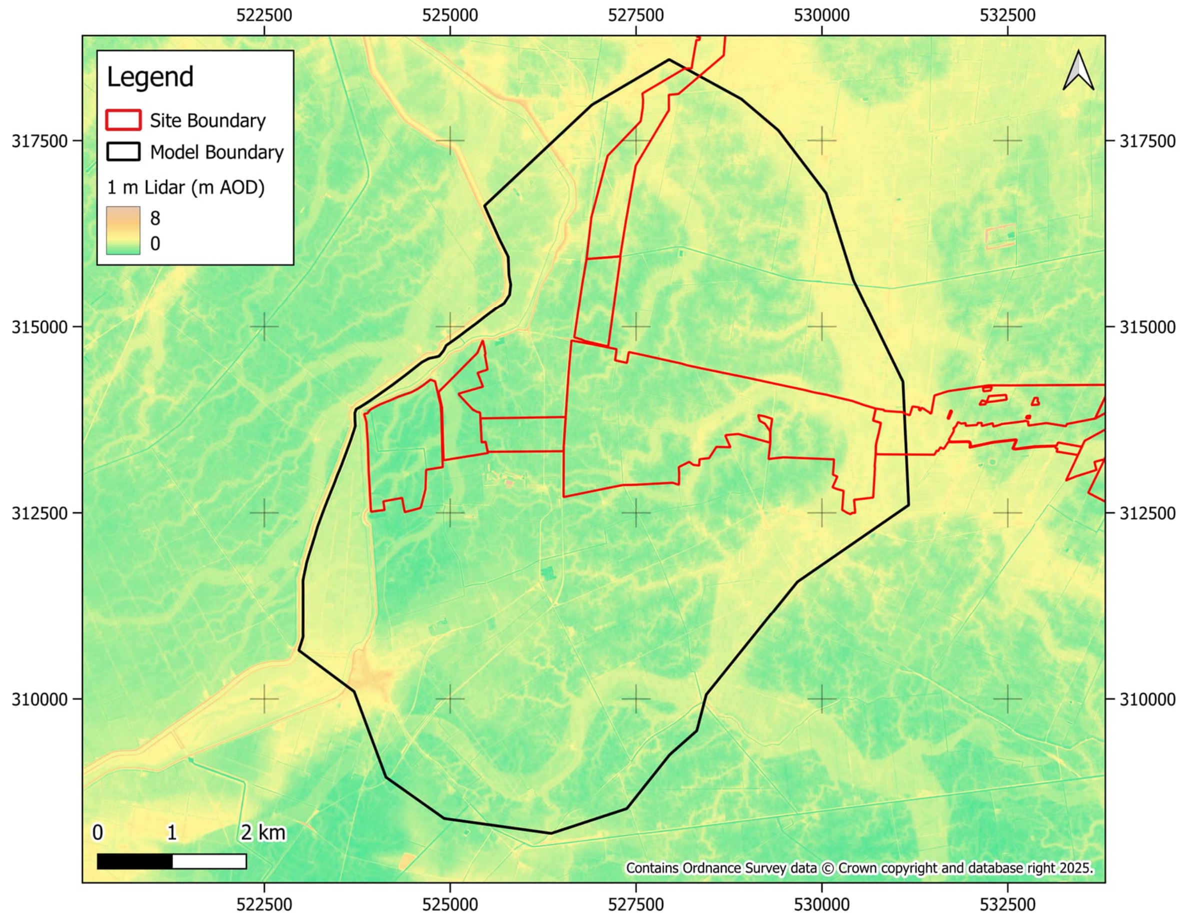

Figure 2 - 2D Model Extent

3.3 Grid Size

A 4m grid size has been adopted as this is sufficient to represent major flow pathways across the domain. The computational area is aligned north to south, as the direction of breach flows is generally in an easterly direction.

3.4 LiDAR data

The topography of the study area is represented by 1 m resolution LiDAR data flown in 2022. This is considered adequate to represent the key flow routes across the modelled area.

3.5 DTM Modification

No modifications to the DTM tiles were made. Any existing buildings within the model domain will be represented within the model as an increase in Manning's n value.

3.6 Boundary Conditions

The boundary conditions for the breach model are the modelled flood levels abstracted from the River Welland Hydraulic Model[1] provided by the EA.

These level hydrographs have been applied to the TuFLOW model as HT (Head vs Time) boundaries along the centreline of the River Welland and have been extracted from each 1D node within the Mike 11 model. The location of the nodes that have been used in the breach model are shown in Figure 3 and the extracted level at each node for the 1.0% and 0.1%, 1.0%CC [2115] and 0.1%CC [2115] AEP flood event are shown in Figure 4, Figure 5, Figure 6 and Figure 7 respectively.

No downstream boundaries are required to prevent glass walling as the model domain is sized so that no glass walling occurs.

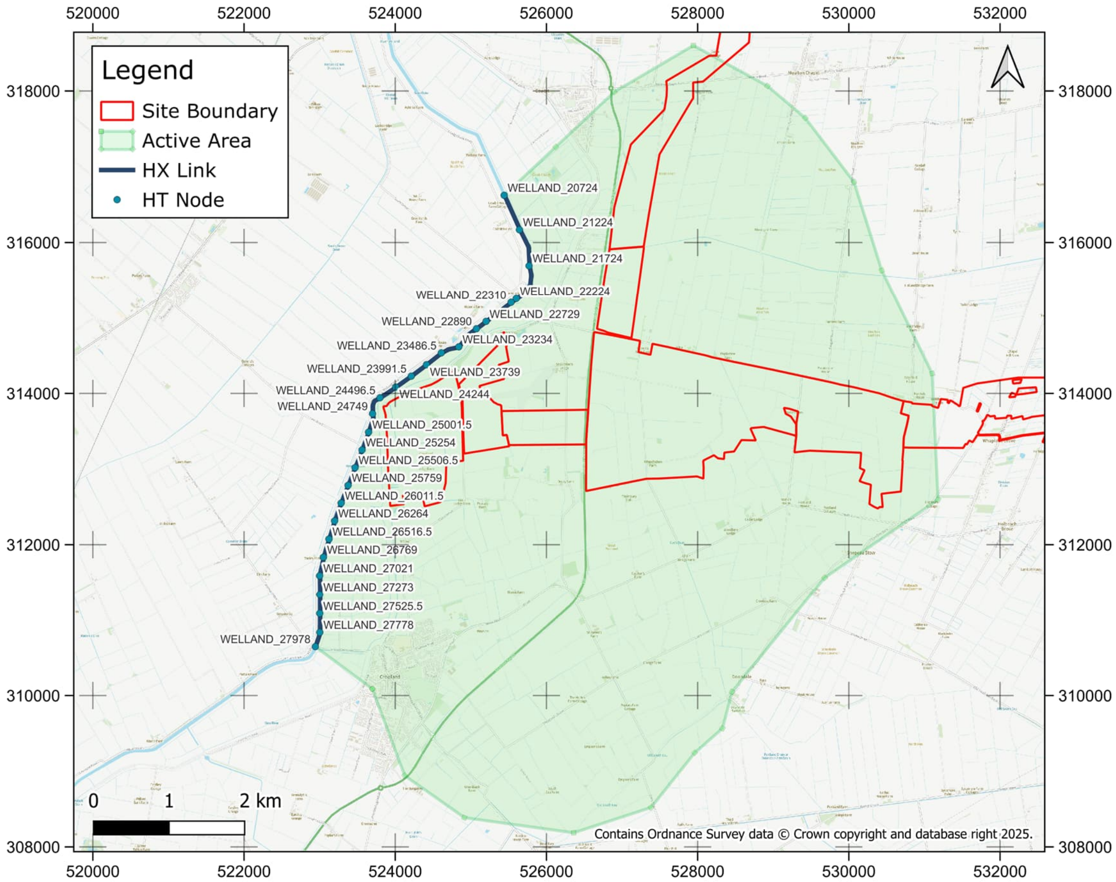

Figure 3 – Location of defined water levels (HT boundaries)

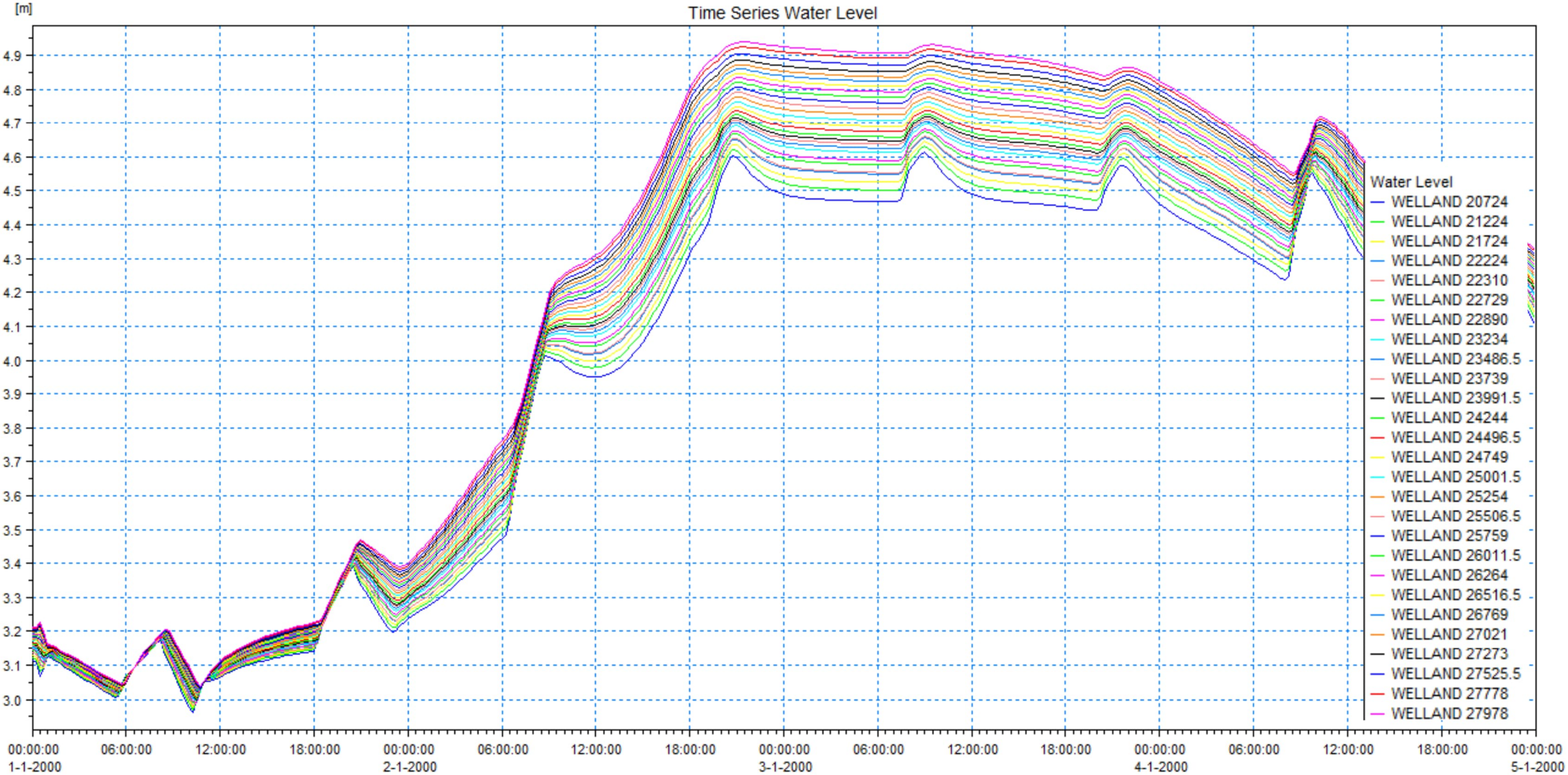

Figure 4 – 1.0% AEP Fluvial Flood Levels extracted from the MIKE Model

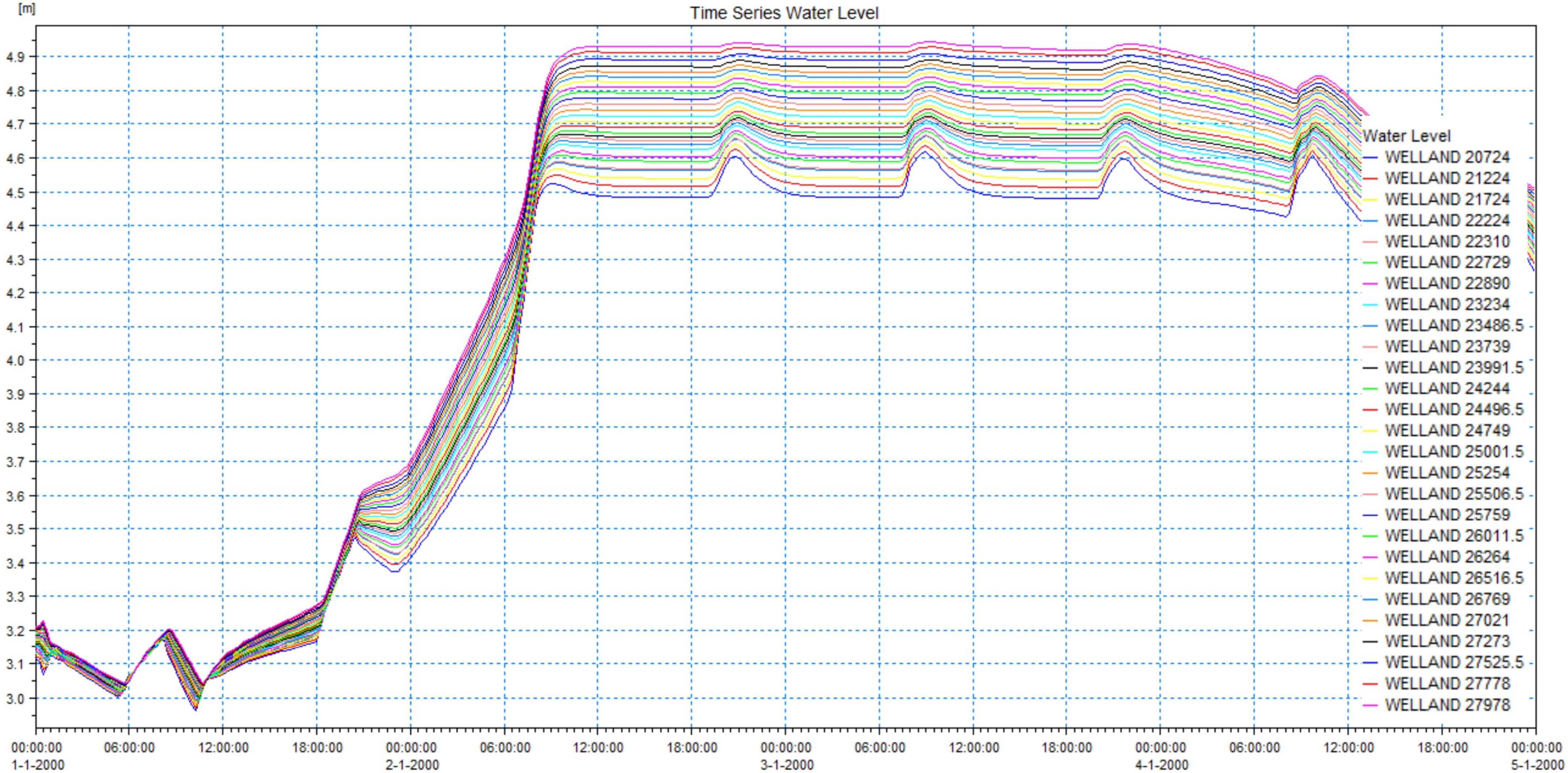

Figure 5 – 0.1% AEP Fluvial Flood Levels extracted from the MIKE Model

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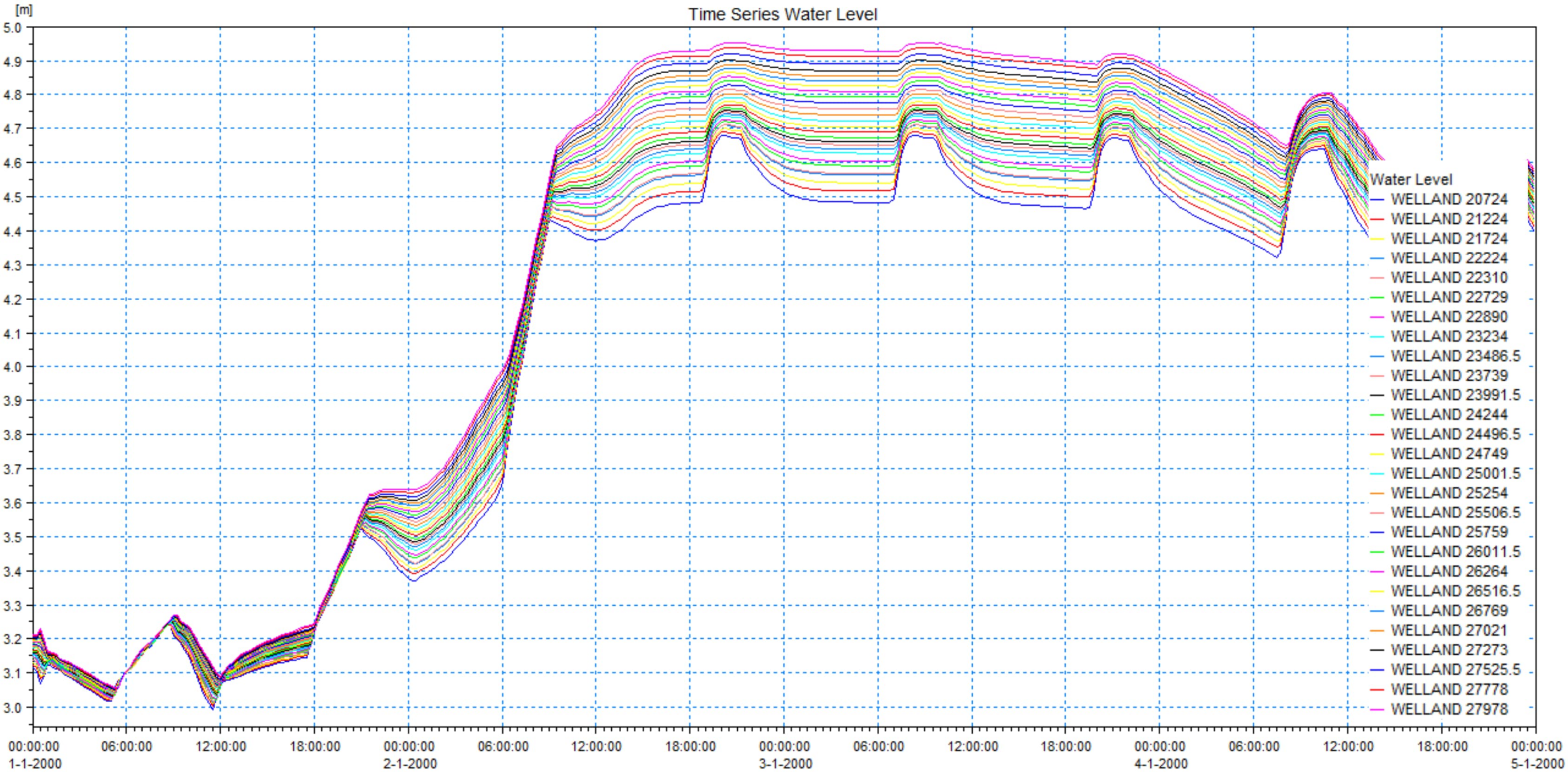

Figure 6 – 1.0%CC AEP Fluvial Flood Levels extracted from the MIKE Model

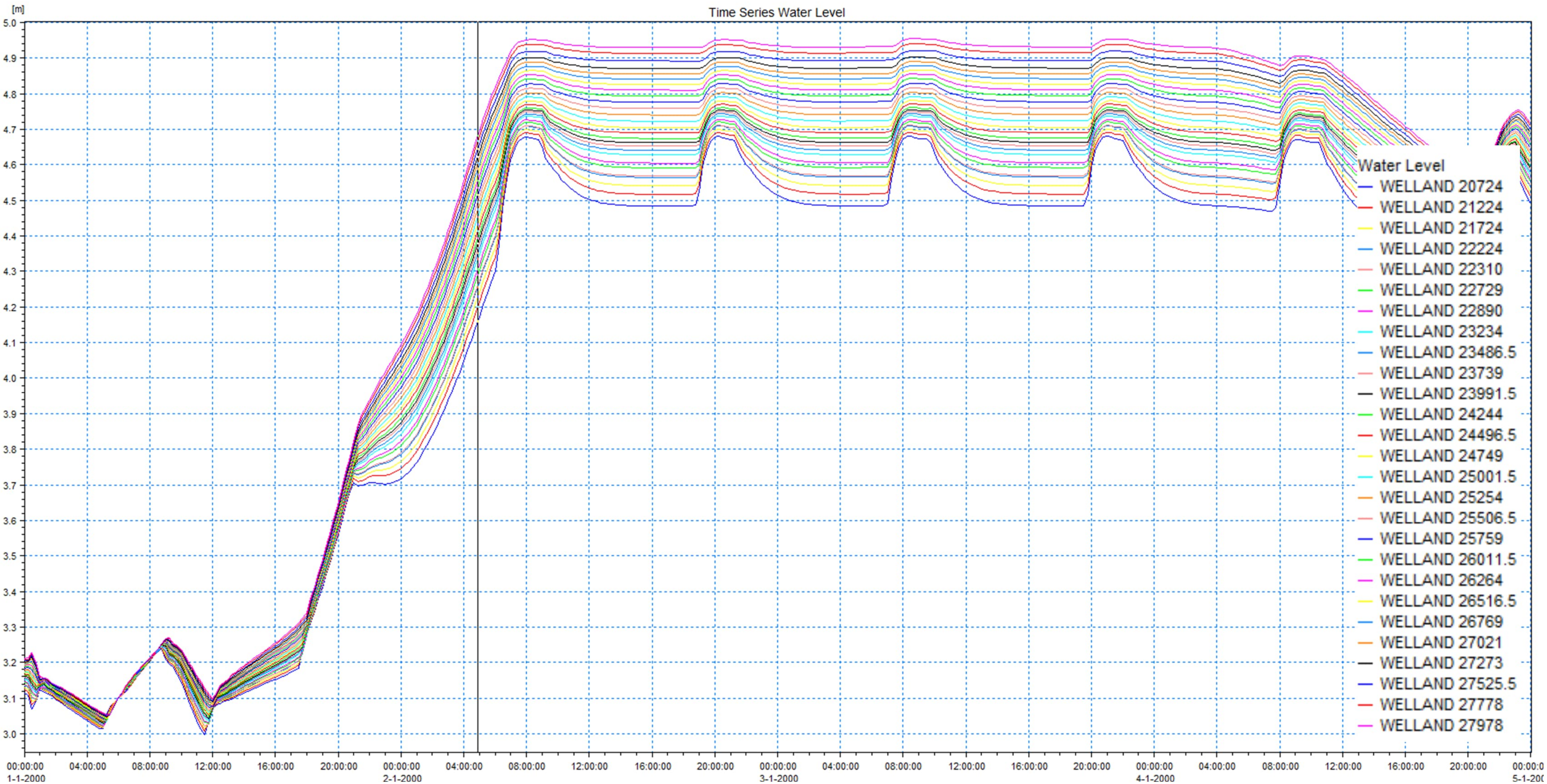

Figure 7 – 0.1%CC AEP Fluvial Flood Levels extracted from the MIKE Model

3.7 Surface Roughness

The floodplain roughness is defined using the Manning's n roughness coefficient, with the land use identified using OS OpenMap – Local, to define roads, buildings, forestry and grassland within the model domain.

2D material files contain an associated code which is referenced within a TUFLOW materials file, where they are assigned an appropriate roughness coefficient as detailed in Table 1.

Table 1 - 2D Domain Surface Roughness Values

Mat_ID	Description	n
1	Fields/Natural Land	0.050
2	Water Bodies	0.020
3	Buildings	0.300
4	Roads	0.020

4 Climate Change

The existing 1D model of the River Welland accounted for climate change in the fluvial flows as well as the downstream tidal boundary at the Fosdyke Bridge tidal gauge (E: 531885, N: 332235). These were applied to both the 1.0% and 0.1% AEP design events. These allowances were applied based on climate change guidance at the time which has since changed. Therefore, the allowances applied have been reviewed to determine whether any modifications to the river level boundary conditions is necessary to bring them in line with current published values for flood risk assessment[5].

4.1 Fluvial

In the existing River Welland hydraulic model a 20% uplift was applied to fluvial peak flows to account for climate change. Referring to current standards, the development is in the Welland Management Catchment. The development is classed as 'essential infrastructure' and has a 40 year design lifetime from its expected start of between 2029 (earliest possible date) and 2033 (likely latest date).

Therefore, the '2080s' epoch (2070 to 2125) for the higher central allowance should be selected, which is +28%. This is higher than the 20% uplift used in the existing River Welland model.

A review of the existing model results has been undertaken to identify if they can be used to define the 0.1% + 28% climate change river levels. This has been done by reviewing the flows in the river at Node 'WELLAND 32831', which is a location upstream of the site where all flows are in bank, against the level at a node adjacent to the proposed development, 'WELLAND 24749'. See Table 2. This comparison has been done with the model results using the climate change boundary condition at Fosdyke Bridge (25.9hr duration fluvial event).

Table 2 – Estimating the 0.1%+36%CC River Levels

Event	Q @ Node 32831 (m^3/s)	h @ Node 24749 (m AOD)	Comment
0.1%	147.72	4.780	Level based on 1.0%+20%CC which has a similar flow at 147.68m^3/s. Hence it can be used as a stand in for the 0.1% event i.e 1.0%+20%CC ≈ 0.1%.
0.1%+20%CC	177.26	4.781	

As can be seen in Table 2, the levels at the site are insensitive to fluvial flows. This is thought to be because the peak levels adjacent to the proposed site are controlled by a combination of peak tidal levels and the crest level of the adjacent defence, which due to its length, would require a large change in flow to result in a change in level. The influence of the tidal levels can also be seen in Figure 4 to Figure 7, where peak levels coincide with the tidal peaks.

As such, the boundary levels within the model will be taken from the 0.1%+20%CC model, as they are a suitable proxy for the 0.1%+28%CC event levels.

[5] Flood risk assessments: climate change allowances, Environment Agency, May 2022. Available from: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

4.2 Tidal

The River Welland model focuses on fluvial flood risk rather than tidal flood risk, hence the use of a spring high water tide was used at the downstream boundary in the existing model, with no tidal surge.

The derivation of the tidal boundary in the existing model used observed data at Fosdyke Bridge from June 2014, and it was estimated that the mean high water spring level is 4.1m AOD.

Climate change up to 2115 (100 years from the baseline year) was applied to the existing model by adding 1.10m to the water levels.

To validate the sea level rise allowance in the existing model, it was compared with current allowances. The proposed development is located in the Anglian area, and Table 3 and Table 4 show the projected sea level rise for the higher central and upper-end allowances throughout its 40-year design life and likely latest date start date of 2033, resulting in a decommissioning date of 2073. Note that climate change calculations start from 2014, which is the base year for the tidal curve used in the existing model.

Table 3 - Sea level rise for Anglian area 2014 to 2125 (Higher Central)

Allowance	Sea level rise (Higher Central)			
	2014 to 2035 (mm)	2036 to 2065 (mm)	2066 to 2073 (mm)	Cumulative rise to 2173 (mm)
Raise/yr (mm) [1]	5.8	8.7	11.6	
Number of Yrs [2]	22	30	8	
Sea Level Raise over Epoch [3] = [1] * [2]	127.6	261.0	92.8	481.4

Table 4 - Sea level rise for Anglian area 2014 to 2125 (Upper End)

Allowance	Sea level rise (Upper End)			
	2014 to 2035 (mm)	2036 to 2065 (mm)	2066 to 2073 (mm)	Cumulative rise to 2173 (mm)
Raise/yr (mm) [1]	7.0	11.3	15.8	
Number of Yrs [2]	22	30	8	
Sea Level Raise over Epoch [3] = [1] * [2]	154.0	339.0	126.4	619.4

This shows that the 1.10m applied to the existing River Welland model is greater than the cumulative rise for the higher central and upper end allowances in the Anglian area through to 2073, which is 0.48m and 0.62m respectively. As such, the use of 1.10m uplift for climate change in the existing tidal boundary will be retained and is considered to be a conservative approach.

Regardless, at the location of the proposed site, the peak flood levels in the River Welland are relatively insensitive to the tidal boundary. This is shown in Table 5, where it can be seen that a 1.10m change in tidal level at the tidal boundary, results in very little change in the river levels at the location of the proposed scheme. As such, if a lower uplift for climate change was used to reflect the 2073 decommissioning date, it is unlikely to have any material impact on peak river levels in the River Welland at the location of the proposed site.

Table 5 – Summary of Peak Levels in the Mike 11 model at key locations during the 1.0% and 1.0%CC event

Chainage (m)	Description	1.0% Level (m AOD)	1.0% CC [2115] Level (m AOD)	Impact of Climate Change (m)
0	Tidal Boundary	4.24	5.35	+1.11
20724	D/S extent of model	4.61	4.68	+0.07
20978	U/S extent of model	4.94	4.95	+0.01

5 Breach Model Set Up

The Environment Agency Breach of Defences Guidance[6] has been used to determine the breach parameters that should be applied in this modelling exercise. As specified in the scope, four breach locations along the Crowland-Cowbit Washes embankment are to be modelled to determine residual flood risk water levels for the proposed infrastructure. The breaches will be run individually using variable Z-shapes with trigger levels, and the resulting model outputs merged to create a single breach flood risk map.

5.1 Breach Locations and Dimensions

The location of the four breaches are shown in Figure 8 relative to the baseline 1.0% +CC AEP defended flood extent. The river itself has raised embankments on either side but there is an additional flood defence further to the east, creating a flood storage area between it and the river.

Therefore, it was considered appropriate to apply the breaches to the outer defence only, with the breach opening being set to the toe level of the embankment. For earth banks in an estuary river, the breach width will be 50m in line with the EA guidance and the four locations were chosen so that they are evenly spaced along the length of the proposed development.

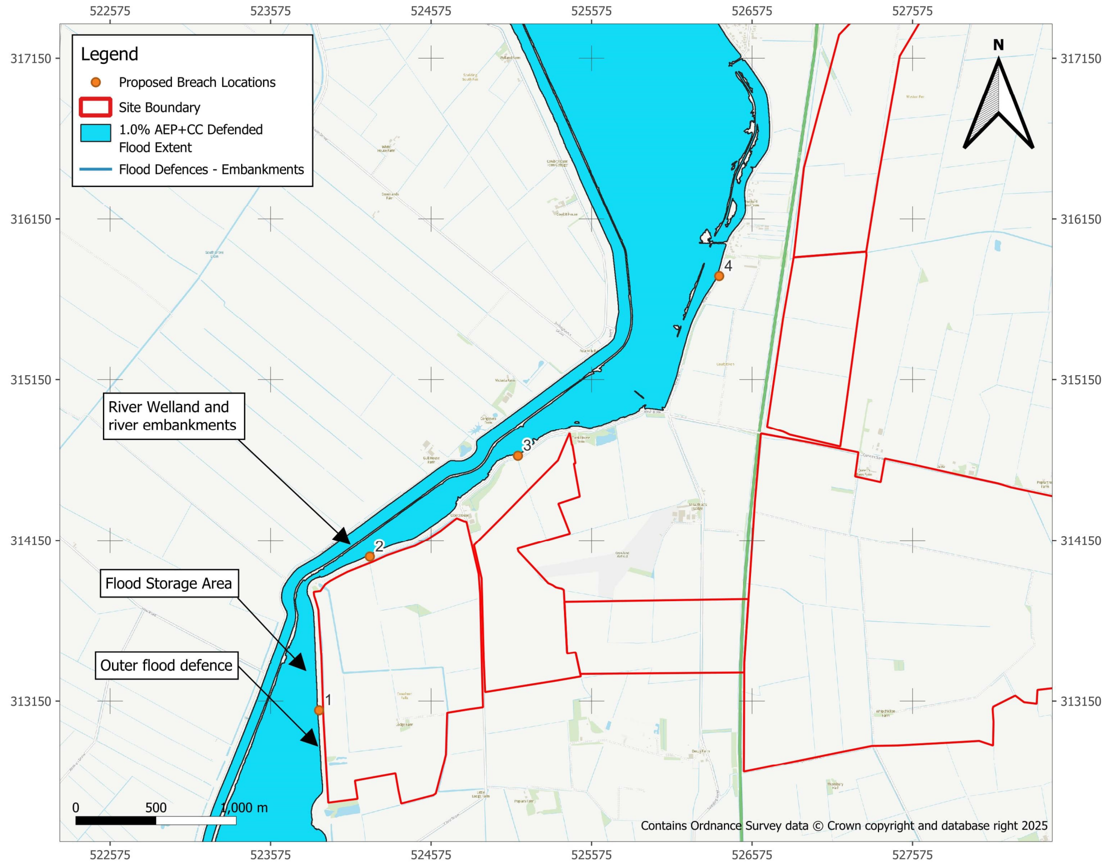

Figure 8 - Proposed Breach Locations

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[6] LIT56413 Breach of Defences Guidance, Environment Agency, June 2021.

5.2 Breach Duration

The EA guidance recommends that the start time should occur when there is some hydraulic loading on the defence. Initially, a breach start when the water level reaches three quarters of the defence height was considered appropriate. However, a review of the defended maximum flood levels shows that the water level may not reach this height along the length of the outer defence. Therefore, it was decided to start the breach when the water level reaches half the height of the defence. Once the breach is triggered, it is assumed that the breach will be fully open within 0.1hrs.

For an earth bank in an estuary river, the EA guidance suggests a time to close the breach of 30 hours in a rural environment. Therefore, this value has been taken forward.

5.3 Summary of Breach Modelling

A summary of the breach modelling parameters is provided in Table 6. The toe and crest elevations of the embankment at each breach location are derived from LiDAR data. The use of LiDAR levels is considered appropriate because they serve to define the trigger points for the breach opening. As this trigger definition incorporates a degree of subjectivity, accurate crest levels are not critical. This is because the resultant breach flooding is a consequence of flows passing through the breach void, rather than water overtopping the embankment crest.

Table 6 – Summary of Breach Modelling

ID	Easting	Northing	Width (m)	Defence Toe Level (mAOD)	Defence Crest Level (mAOD)	Trigger Level (mAOD)	Collapse Duration (hrs)	Open Duration (hrs)
1	523877	313093	50	1.8	5.8	3.8	0.1	30
2	524194	314049	50	2.3	5.8	4.1	0.1	30
3	525117	314675	50	2.1	6.1	4.1	0.1	30
4	526371	315797	50	1.8	5.8	3.8	0.1	30

6 Post-Development Scenario

6.1 Post-development Model Overview

Figure 9 shows the sections of the proposed development located within Parcels A, B and C. These were incorporated into the model to create the post-development scenario to determine the impacts on flood risk as a result of the proposed development during a pump failure event. The post-development infrastructure incorporated into the model includes the following:

- Solar Tables
- Solar Stations
- Substations

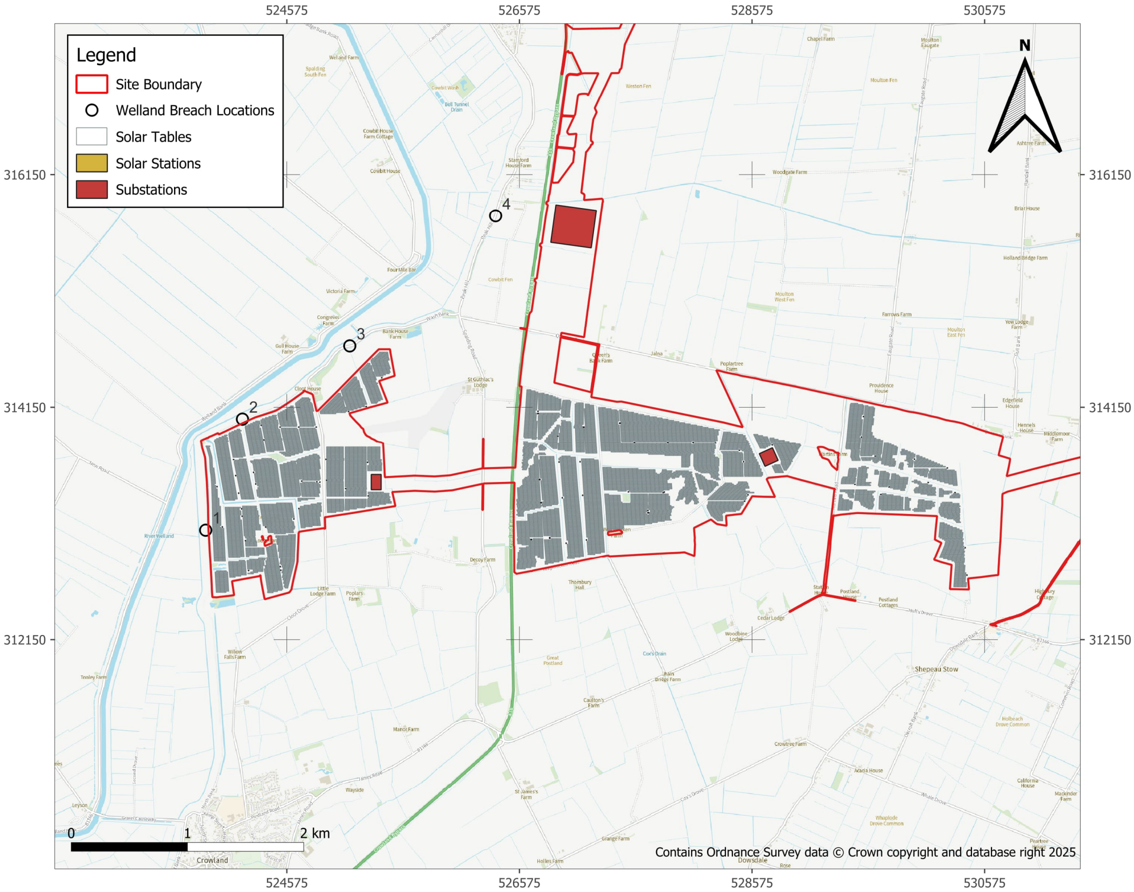


Figure 9 – Proposed infrastructure with land parcels A, B and C

6.1.1 Modifications for the Post-Development Scenario

<u>Solar Tables</u>

The solar tables will be raised on supports above ground level, with the lower edge of the panels positioned at least 0.8 m above the ground. To assess the impact that the proposed Solar Tables will have on flood risk they have been included in the model as form loss coefficients, based on the proposed dimensions and number of support legs. Therefore, the supports for the panels have been represented using a layered flow constriction layer (2d_lfcsh) to assess their impact on flow routes within the PPC. Though consideration was given to modifying the underlying DTM to explicitly represent each support, this approach was deemed inappropriate due to the small size of the supports relative to the model grid resolution.

To represent the supports as a flow constriction layer, two key attributes are required – the Form Loss Coefficient (FLC) and Blockage Fraction (J) associated with the supports. The FLC used for the post-development scenario was derived using the method for applying losses from Hydraulics of Bridge and Waterways[7], which uses the width of the supports normal to the flow. The procedure set out in Hydraulics of Bridge and Waterways applied to derive the FLC is outlined below:

1. The average width of the supports was taken as 135 mm, as the width of the supports throughout the development will range between 80-150 mm.
2. The flow width was calculated for each support by assessing the flow area blocked on a span-by-span approach. For this approach, the flow width from mid-support to mid-support was calculated using an approximate distance of 3 m between the supports.
3. The average width of the supports and the flow width were used to calculate the blockage fraction (J value) for each support.
4. Using the Hydraulics of Bridge and Waterways chart (Figure 10) and assuming a single rounded shape support, the FLC (ΔKp) was determined.

Table 7 – Parameters used to derive FLC

Parameter	Description	Value
b	Flow width assessed on a span-by-span approach	3.000 m
W_p	Support with normal to flow	0.135 m
J	Blockage fraction (b/W_p)	0.045 (4.5%)
Δkp	FLC (from graph based on J value)	0.051

[7] Hydraulics of Bridge and Waterways. Hydraulics Design Series No. 1. Bradley, J. 1978.

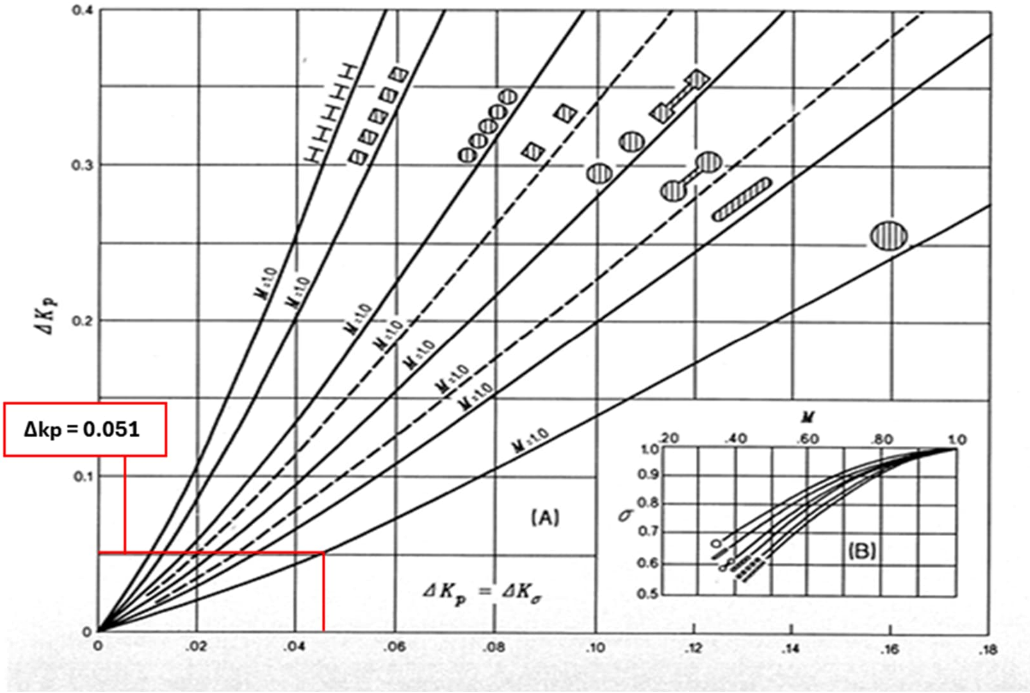


Figure 10 – Method to derive FLC, Hydraulics of Bridge and Waterways[7]

Several limitations were encountered when deriving the FLC, which related to the proximity of the rows for the supports for each solar table, as TuFLOW does not allow multiple FLC lines within the same model cell. Therefore, a single FLC line was used for each solar table within the 2d_lfcsh layer, and the Δkp value was taken as 0.102, i.e., the combined Δkp value for both rows of supports.

The layered flow constriction layer (2d_lfcsh) layer was used to allow losses and blockages to be varied with water depth. This allowed:

- Layer 1 to represent the supports beneath the panels, where the supports are 0.8 m above the ground (L1_Obvert), with the 4.5% Blockage Fraction and 0.102 combined Δkp applied.
- Layer 2 to represent the inclined solar at a height of 3.5 m (L2_Depth), i.e., 4.3 m above ground level, including the 0.8 m height of the supports. This layer will be 100% blocked and has an increased Δkp of 1.56 due to the additional energy losses associated with the flow surcharging the panels. The Δkp value of 1.56 was adopted from the approach used for modelling bridge decks and represents form loss when the flood level is above the soffit level of the bridge, i.e when it is surcharged.

Solar Stations

The solar stations have been represented in the post-development scenario by a direct modification of the underlying DTM using a z-shape layer (2d_zsh). The z-shape layer was used to define the area for the bunds around the solar station, and the terrain elevations in the 2D domain were modified based on this defined area.

A Z value of 2.3 m and the 'ADD' shape option were used for the z-shape layer, which instructed TuFLOW to increase the ground levels within the defined area by 2.3 m. This created discrete structures, representing the worst-case bund level around the solar stations. The z-shape features were produced by buffering the solar stations by 20 m, providing the flood protection zone for the worst-case flood depths.

Substations

Similar to the solar stations, bunds for the substations were represented using a z-shape layer. The area for the layer was informed using the footprint of the substations provided in the masterplan for the development[8]. The levels of the bunds for the substations are outlined below:

- 400 kV substation and BESS Compound: 1.7 m above ground
- 132 kV substation on Parcel A: 2.2 m above ground
- 132 kV substation on Parcel B: 1 m above ground

[8] Meridian Illustrative Site Layout, Constraints and Layout Overview. Downing. Dwg. No. DOW-2025-01-DCO.

7 Modelling Results

7.1 Baseline Scenario

Figure 11 to Figure 14 show that in a defended scenario, floodwater fills up the storage area between the inner and outer defences but does not overtop the outermost embankment and enter the development. They also highlight the similarities between the four design events, caused by the minimal differences between the water levels extracted from the original MIKE model as highlighted in section 4. The maximum flood depth during the 1.0% AEP and 1.0% AEP +CC events is 4.3 m. Similarly, during the 0.1% AEP and the 0.1% AEP +CC events, the maximum flood depth is 4.4 m. These maximum flood depths occur within the small streams and drains present across the floodplain.

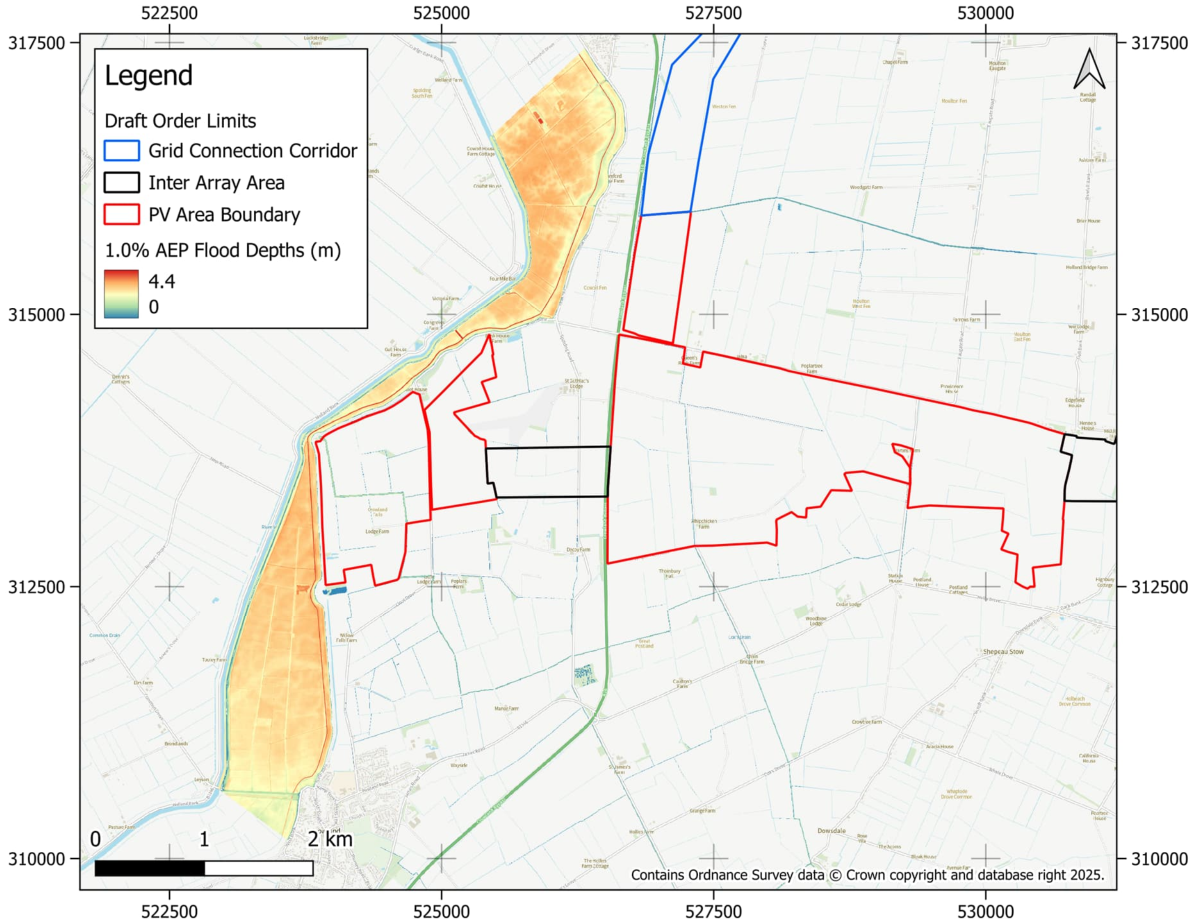


Figure 11 - 1.0% AEP Modelled Flood Depths (m)

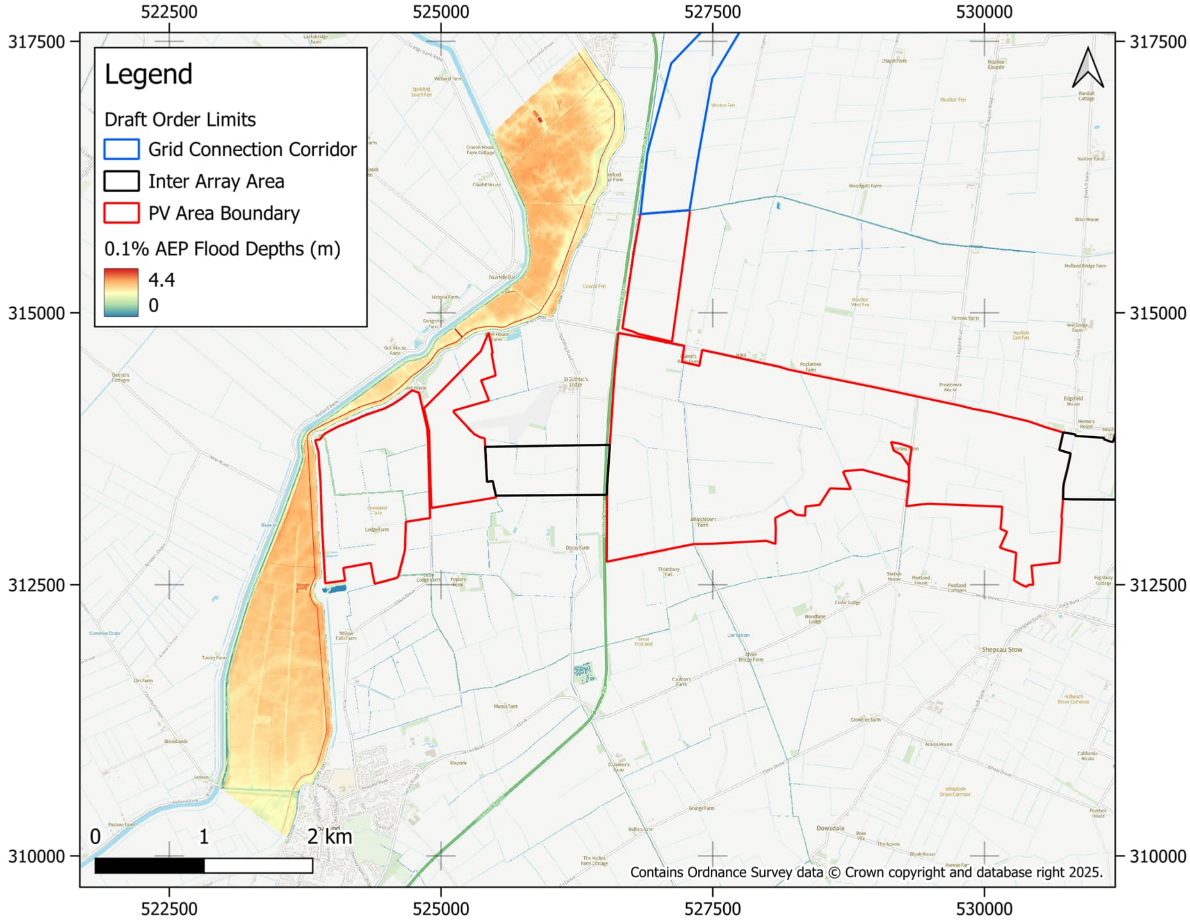


Figure 12 - 0.1% AEP Modelled Flood Depths (m)

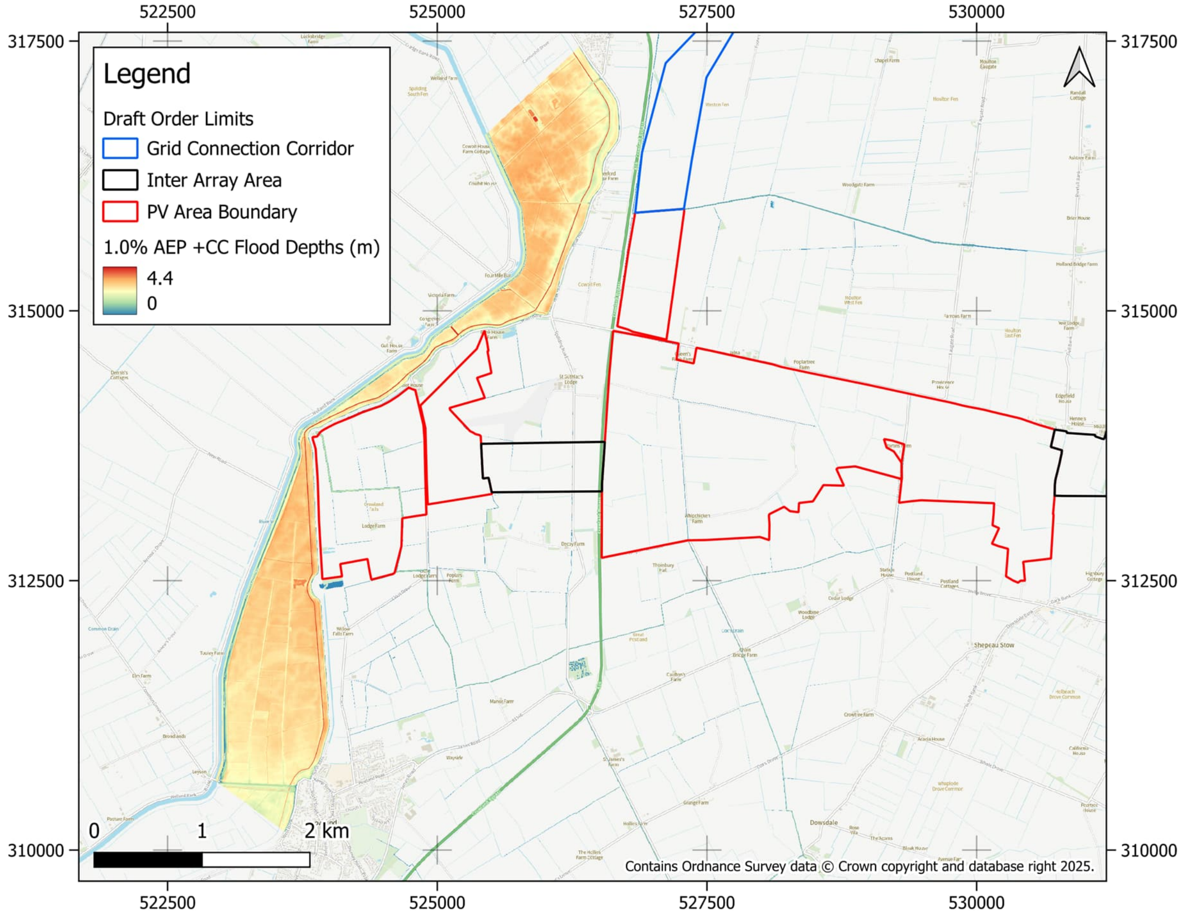


Figure 13 - 1.0% AEP +CC Modelled Flood Depths (m)

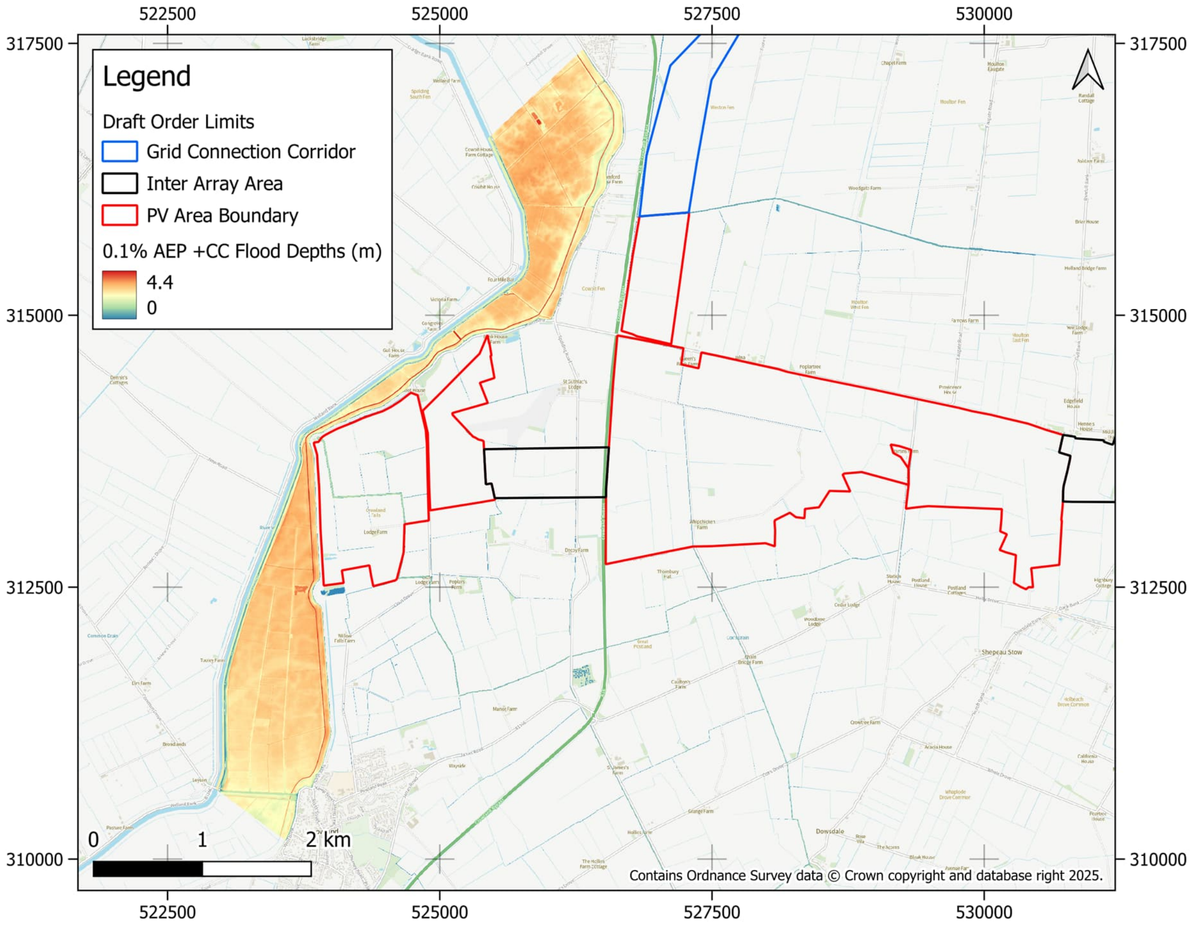


Figure 14 - 0.1% AEP +CC Modelled Flood Depths (m)

7.2 Breach Scenario

Figure 15 to Figure 18 show the modelled flood depths for the 0.1% AEP + CC design event during the four breach scenarios as described in section 5.

Figure 15 to Figure 17 show breaches at locations 1 to 3 result in very similar flood extents and depths. Areas of the proposed development located within the breach flood extents for locations 1 to 3 include PV areas A-1, B-1, B-2, B-3, B-5, and C-2. Flood depths within the PV areas are greatest in the west of the site closest to the River Welland. Table 8 shows the maximum flood depths within PV areas during breach scenarios 1 to 3 is 3.67 m. This maximum flood depth is associated with small streams or drains within the site boundary. Outside of these streams, maximum flood depths are greater than 2 m and associated with PV area A-1.

Figure 18 shows breach at location 4 results in more widespread inundation to the north, south, and east, but a reduction to the flood extents to the west in comparison to breach locations 1 to 3. Maximum flood depths are also lower during this breach event, with a maximum depth of 2.94 m within the PV areas, see Table 8.

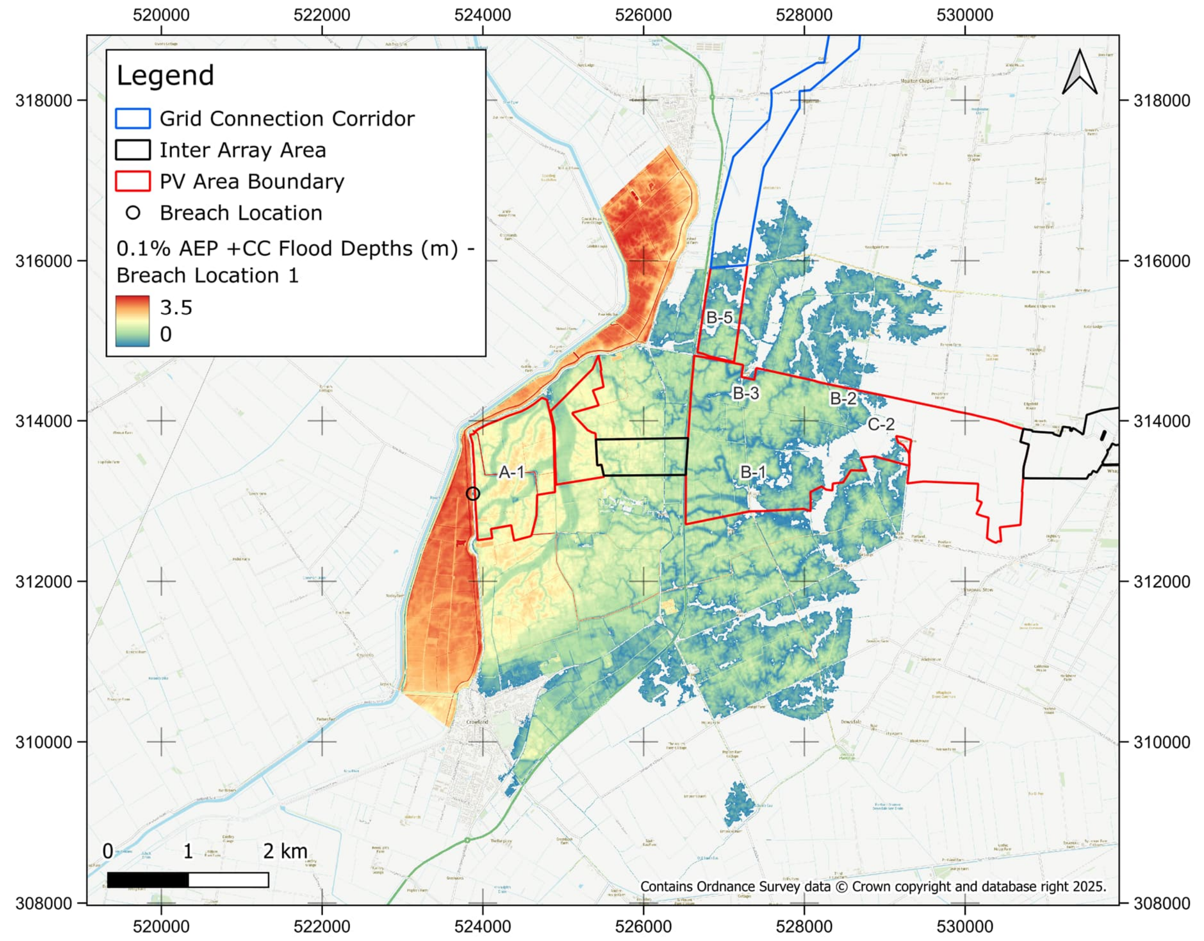


Figure 15 - 0.1% AEP +CC Flood Depths (m) - Breach Location 1

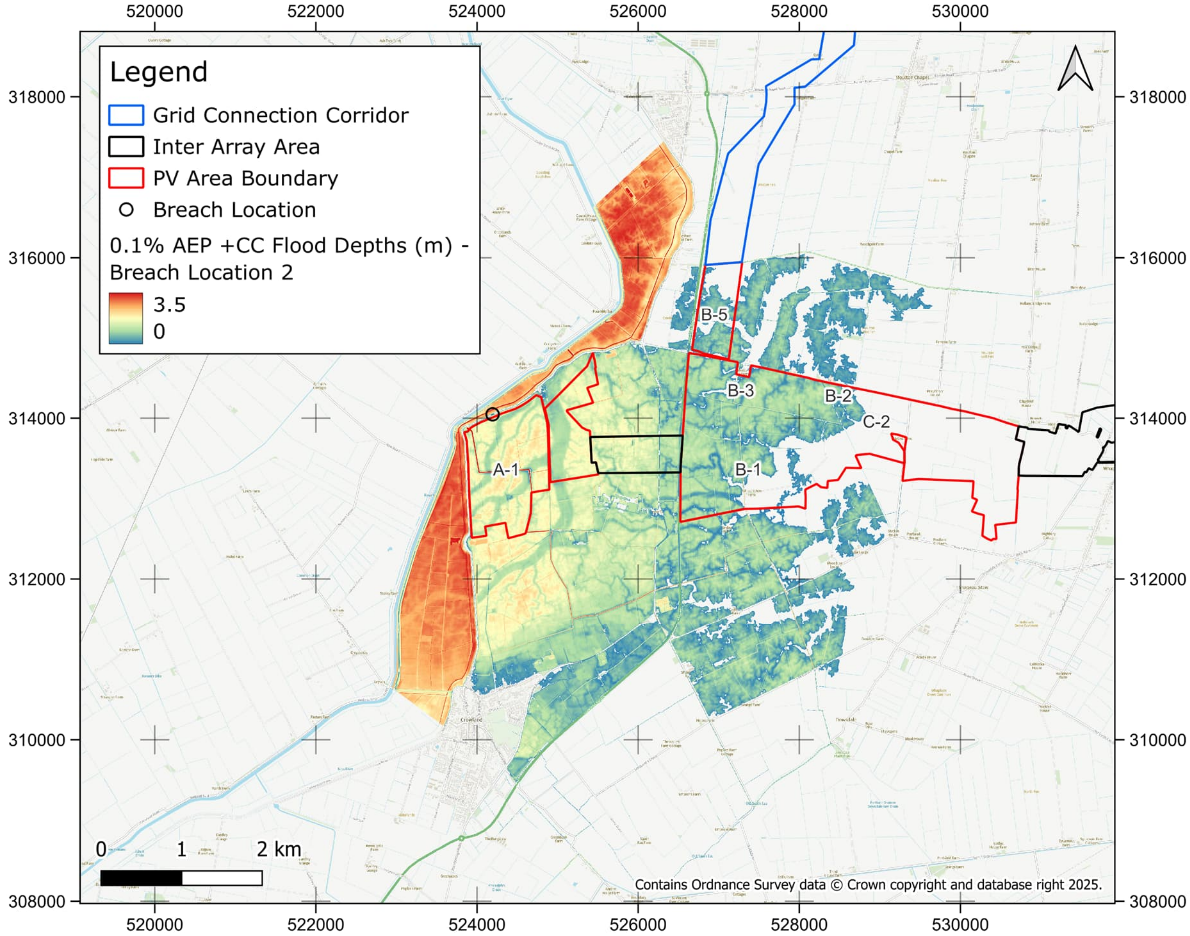


Figure 16 - 0.1% AEP +CC Flood Depths (m) - Breach Location 2

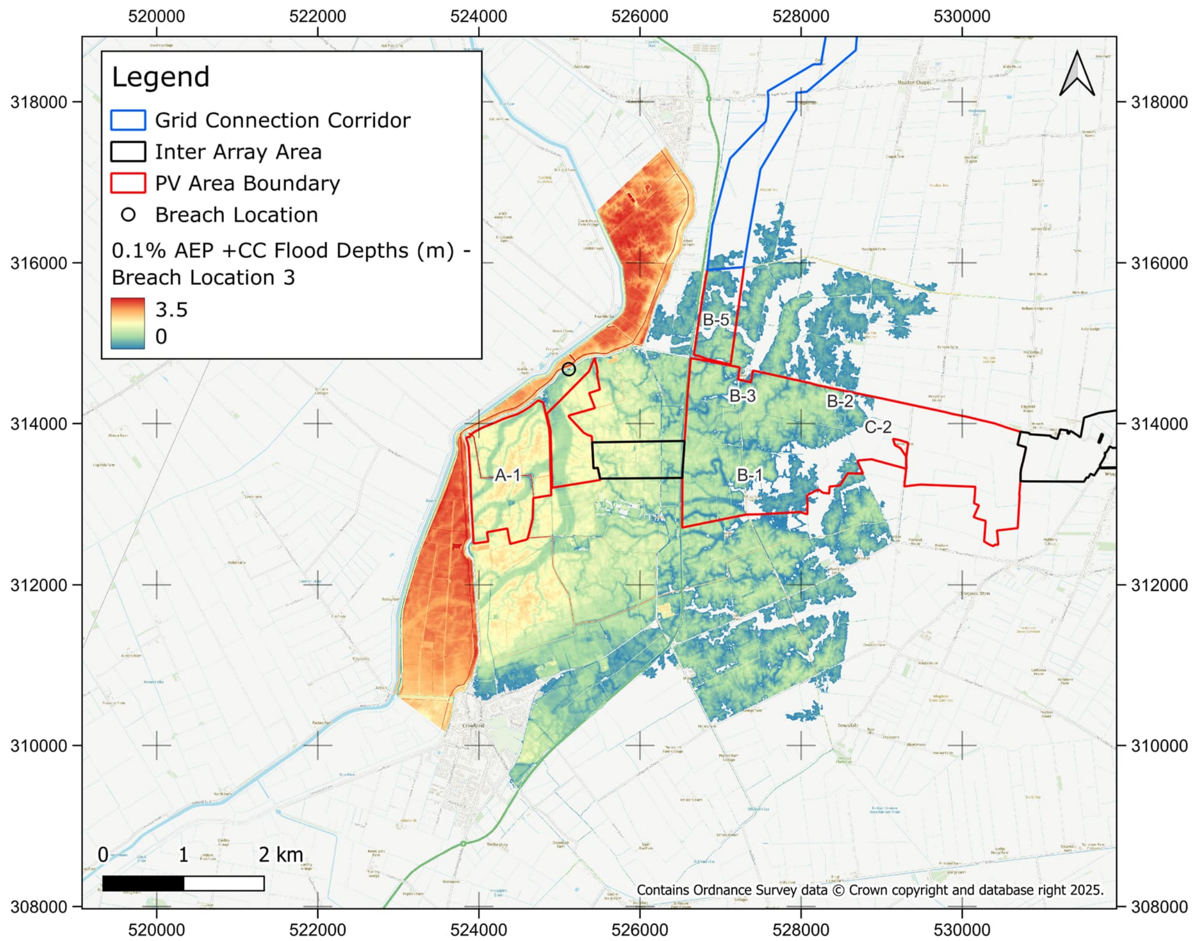


Figure 17 - 0.1% AEP +CC Flood Depths (m) - Breach Location 3

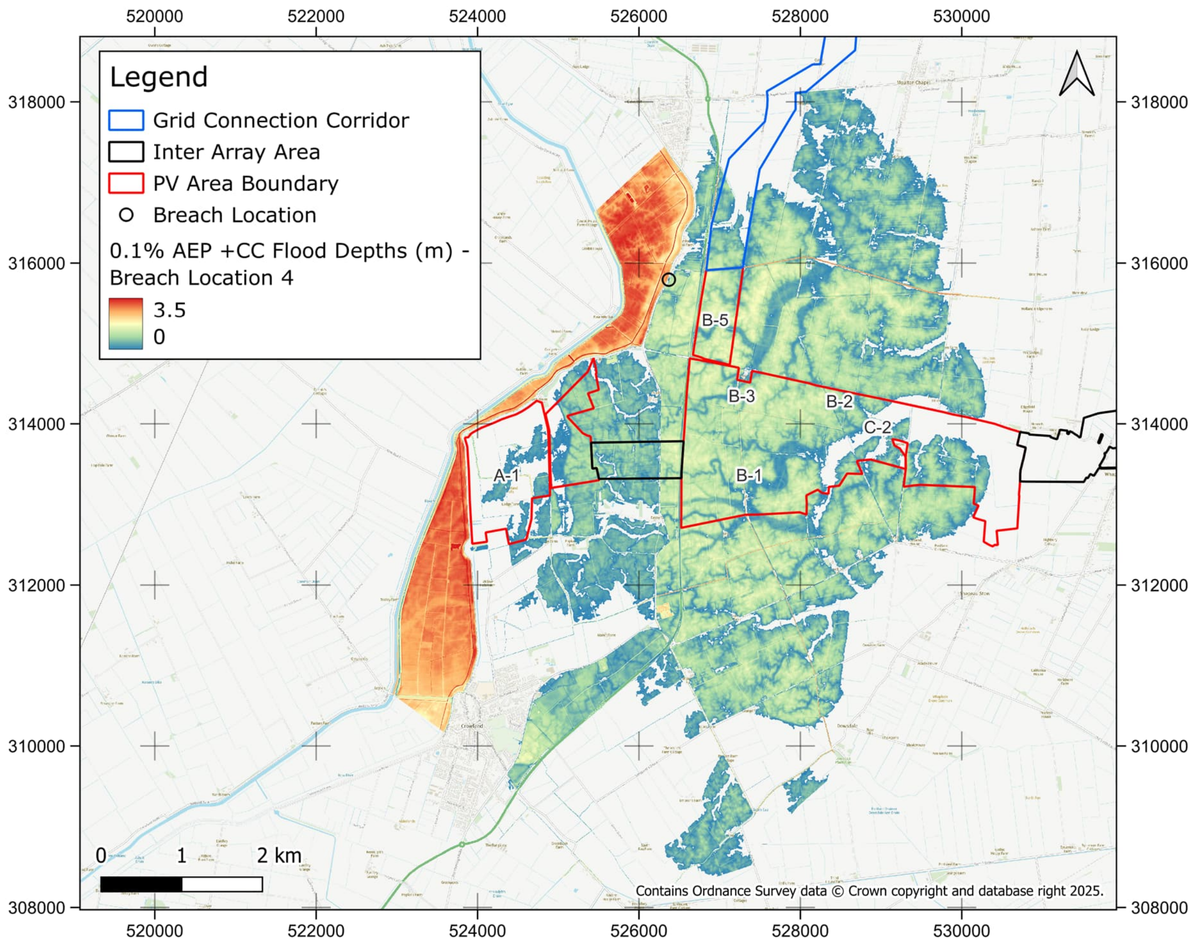


Figure 18 - 0.1% AEP +CC Flood Depths (m) - Breach Location 4

Table 8 - Maximum Breach Flood Depths

Breach Location	Maximum Flood Depth (m)		
	PV Area Boundary	Inter Array Area	Grid Connection Corridor
1	3.69	3.18	1.16
2	3.67	3.16	0.42
3	3.66	3.17	1.09
4	2.94	2.31	1.90

7.3 Post-Development Scenario

The post-development scenario was run for the selected breach scenarios 1 and 4. The flood depths and extent from these runs are shown in Figure 22 and Figure 24 . The results indicate similar risks of flooding across the development as in the baseline breach scenarios at locations 1 and 4, which is confirmed by the depth change analysis outlined in section 0.

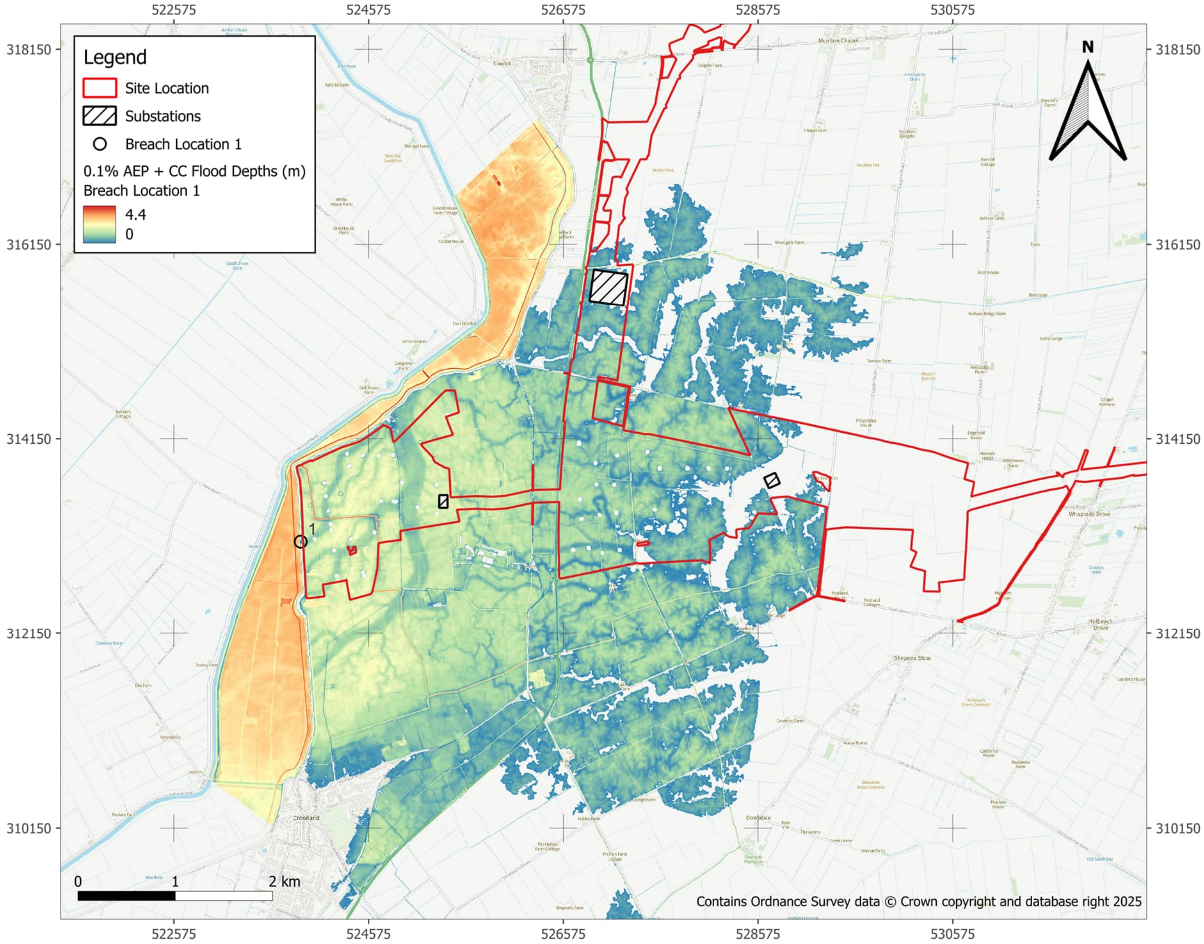


Figure 19 – Post-Development 0.1% AEP +CC Flood Depths (m) - Breach Location 1

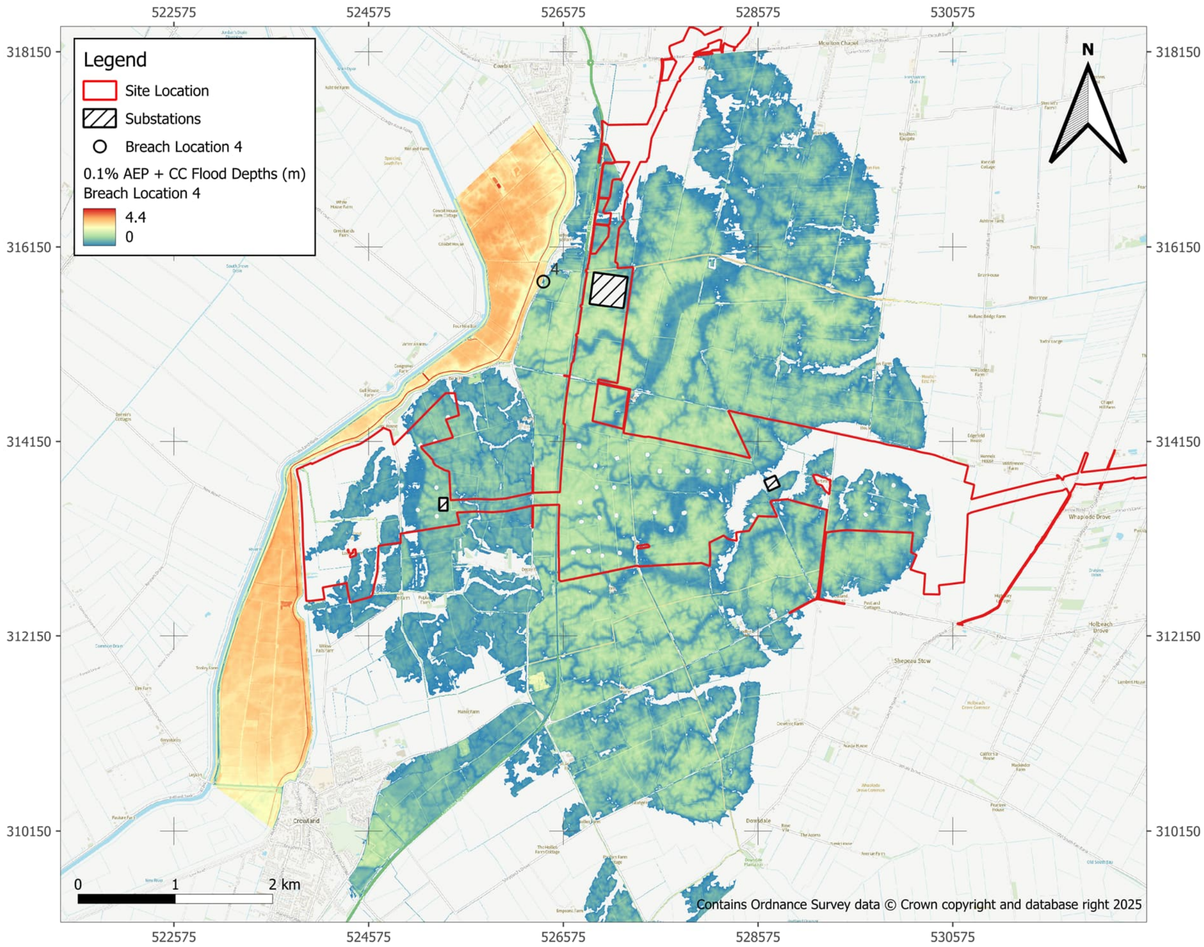


Figure 20 – Post-Development 0.1% AEP +CC Flood Depths (m) - Breach Location 4

7.3.1 Extent and Depth Change Analysis

An extent and depth change analysis was undertaken between the baseline breach scenarios at locations 1 and 4, and the post-development scenario to assess whether the proposed development results in any increased third-party impacts. The results mostly indicate negligible changes in flood depths and extent across the development and floodplain, see Figure 22 to Figure 23. Similarly, the majority of the floodplains have no impact, however, there are some small increases in flood levels in isolated areas of up to 140 mm for breach location 1 and 350 mm for breach location 4.

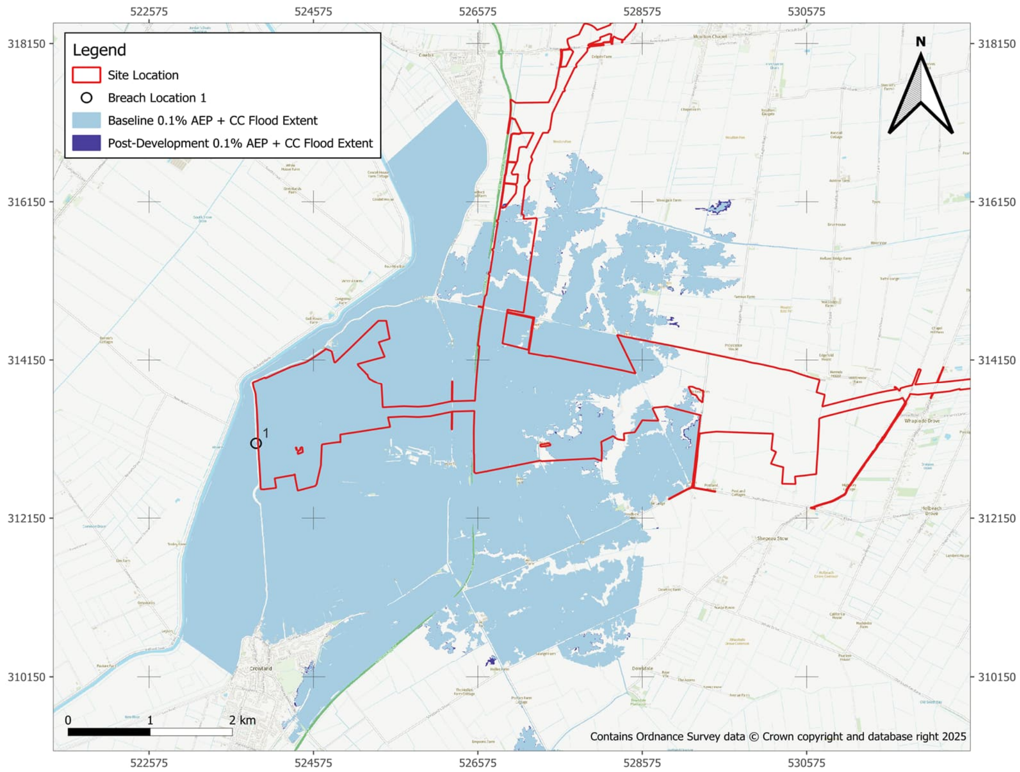


Figure 21 – 0.1% AEP +CC - Breach Location 1 Extent Change

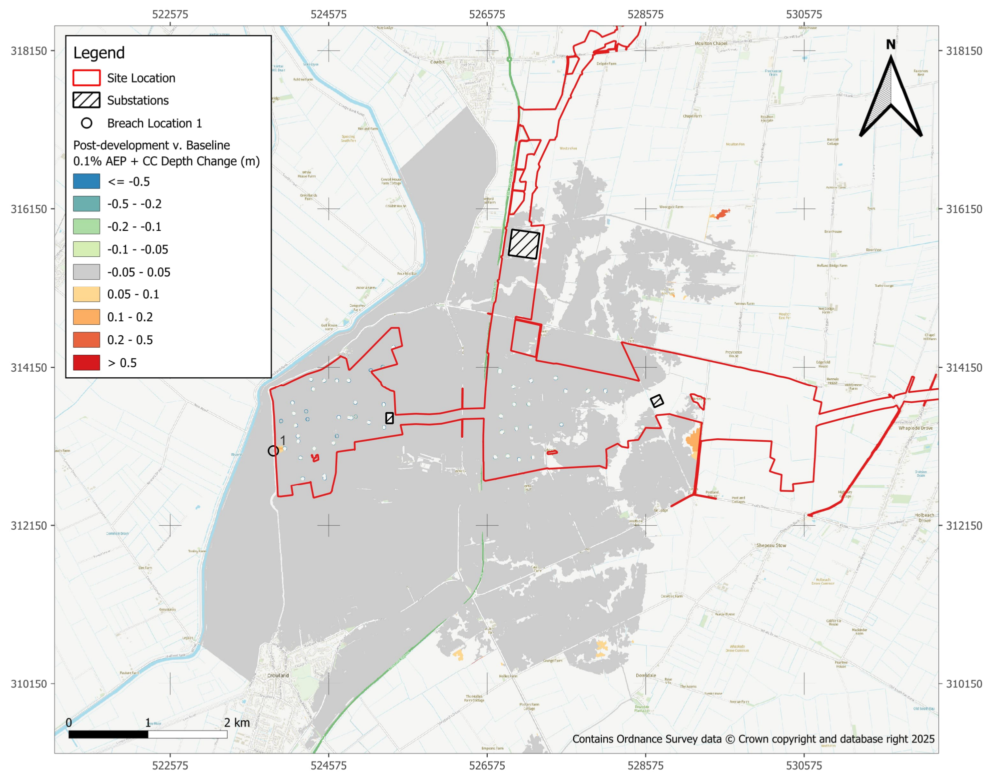


Figure 22 - 0.1% AEP +CC - Breach Location 1 Depth Change (m)

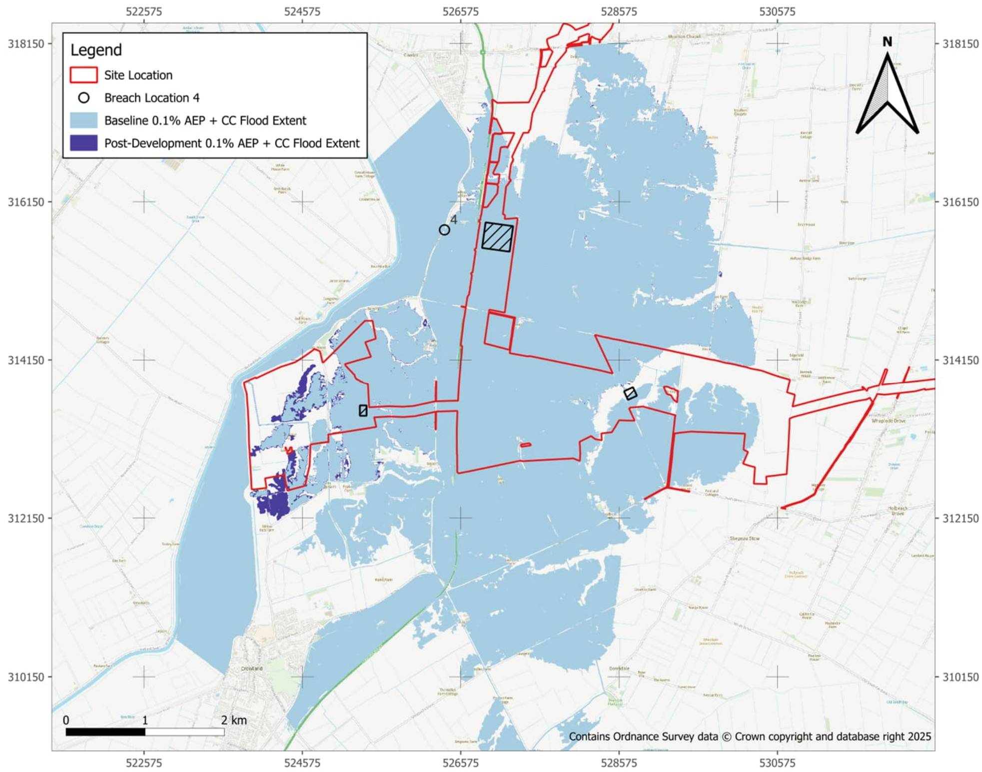


Figure 23 – 0.1% AEP +CC - Breach Location 4 Extent Change

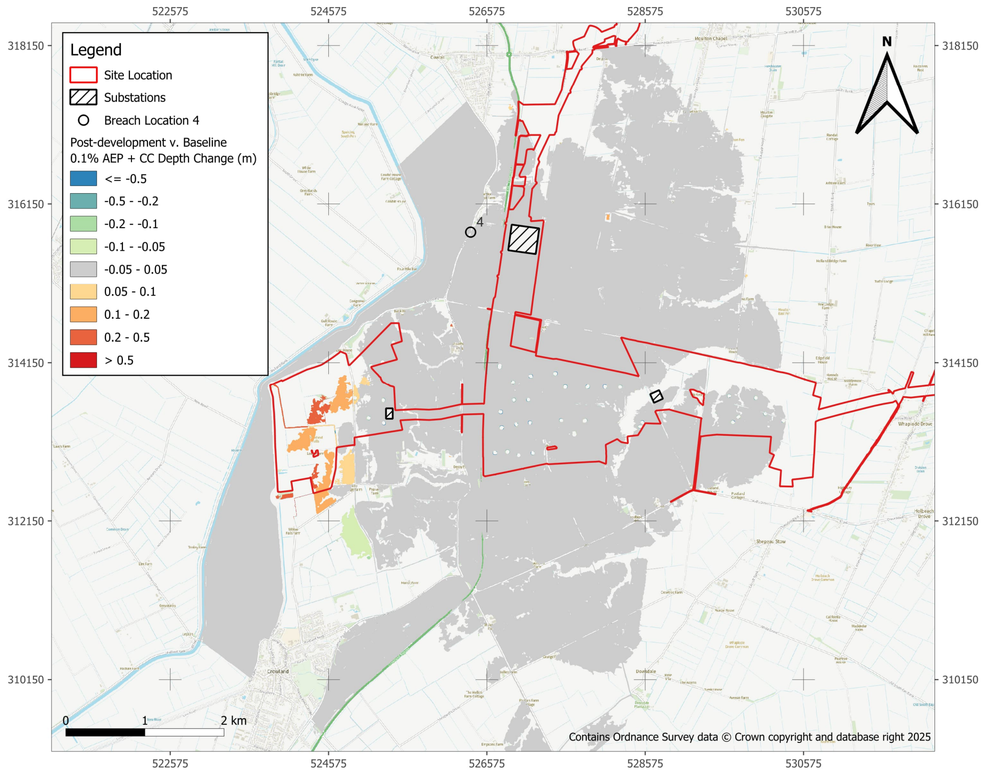


Figure 24 - 0.1% AEP +CC - Breach Location 4 Depth Change (m)

8 Sensitivity Analysis

Sensitivity analyses have been carried out on the breach model, specifically breach number 4 for the 0.1% +CC AEP fluvial event on the following parameters:

- 2D Grid: The resolution of the model will be increased from a grid size of 4m to 2m. At the same time, the grid orientation will also be rotated by 45°.
- Mannings n.

The results of the sensitivity analysis have been assessed at key points within the model domain and the sensitivity sample point locations are shown in Figure 25.

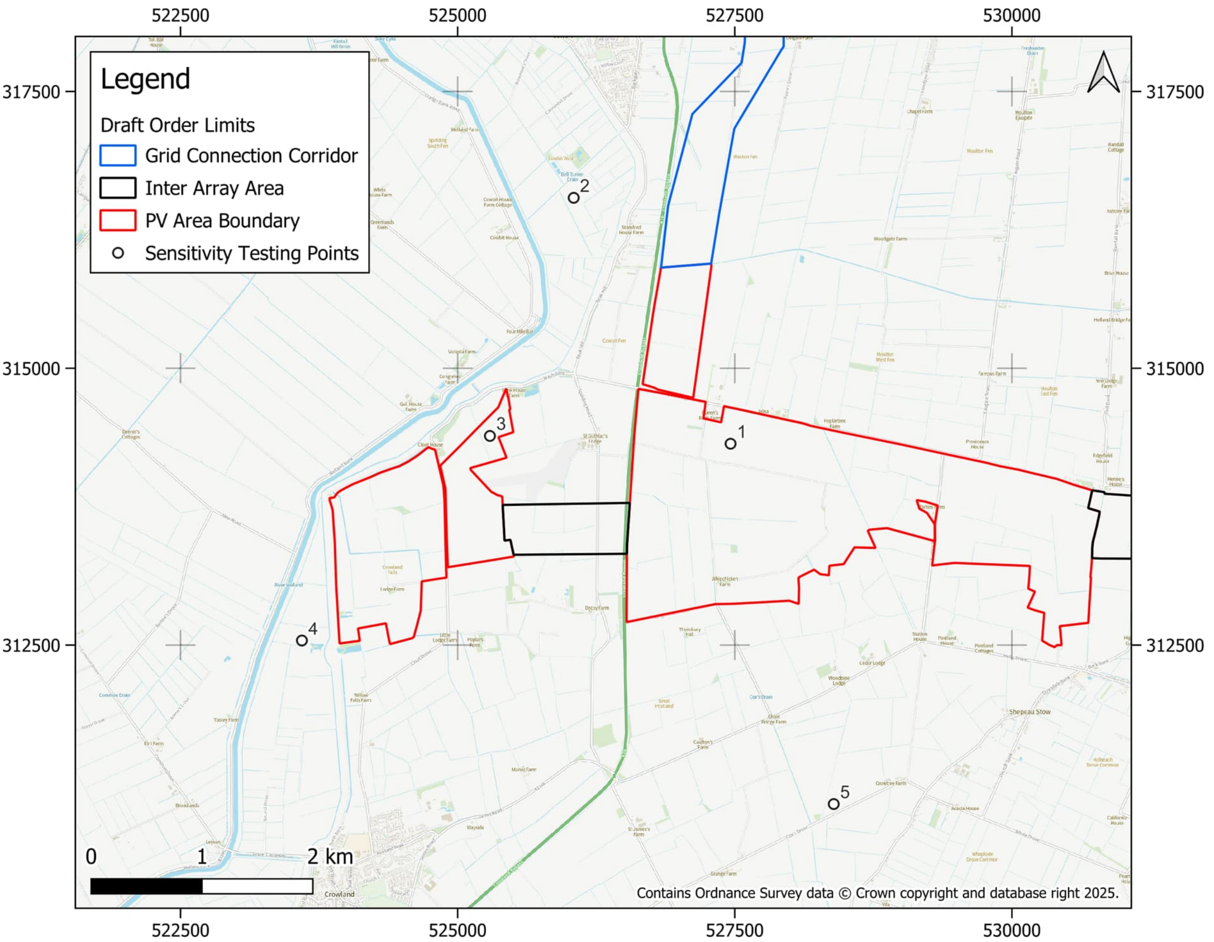


Figure 25 – Location of sensitivity sample points.

8.1 2D Grid Size and Orientation

The results of this sensitivity analysis are displayed in Table 9 below.

Table 9- Grid size and orientation sensitivity analysis.

Sensitivity Point	Breach Level (m AOD)	SEN(Grid) Level (m AOD)	Difference (m)
1	2.25	2.21	-0.04
2	4.75	4.76	0.01
3	1.45	1.09	-0.36
4	4.83	4.80	-0.03
5	1.93	1.89	-0.04

The outputs from this test generally show that decreasing the grid size and rotating the grid orientation resulted in a small decrease in flood depths at points 1, 4, and 5. However, at point 3, a greater decrease of 0.36 m was observed. No change occurred at point 2 which is within the flood storage area. Therefore, it has been concluded that the model is not very sensitive to a change in the grid size and that a larger cell size would be appear to result in more conservative depths.

8.2 Manning's Number

The results of this sensitivity analysis are displayed in Table 10 below.

Table 10- Manning's n sensitivity analysis.

Sensitivity Point	Breach Depth (m)	SEN(N+) Depth (m)	Difference (m)	SEN(N-) Depth (m)	Difference (m)
1	1.62	1.62	0.00	1.62	0.00
2	3.10	3.10	0.00	3.10	0.00
3	0.79	0.79	0.00	0.79	0.00
4	3.09	3.09	0.00	3.09	0.00
5	0.86	0.86	0.00	0.86	0.00

Table 10 shows that the model is not sensitive to changes in manning's n, with no changes to model depth seen with either a 20% increase or decrease in roughness values.

9 Model Stability and Limitations

9.1 2D Model Stability and Limitations

With the HPC solver, one of the main indicators of model stability is the time step selected by TuFLOW. This has been reviewed and plotted in Figure 26.

Ideally, the time step should not fall below one tenth of the timestep that would have been used with the classic solver. For this model, a timestep of between 2.0 and 0.8 would have been selected for the grid resolution of 4m, hence the HPC timestep should not fall below 0.2 and 0.08 seconds. The plot of the evolution of the 2d timestep (dtStar) indicates that the timestep does not drop below 0.5 s for the duration of the model run.

In addition to the 2d timestep, there are three control numbers which must be satisfied:

- A Nu value of 1.0 or greater may indicate that the velocity is unusually high, or the cell size is too small for the modelled velocity.
- A Nc value of 1.0 or higher can be caused by large depths in relation to the cell size.
- A Nd value of 0.3 or higher, might suggest that there is poor boundary setup, or insufficient SX cells linked to a 1D structure, or the cell size is too small.

Figure 26 indicates that all of the control numbers are satisfied during the simulation, with Nc the limiting factor for the majority of the model run.

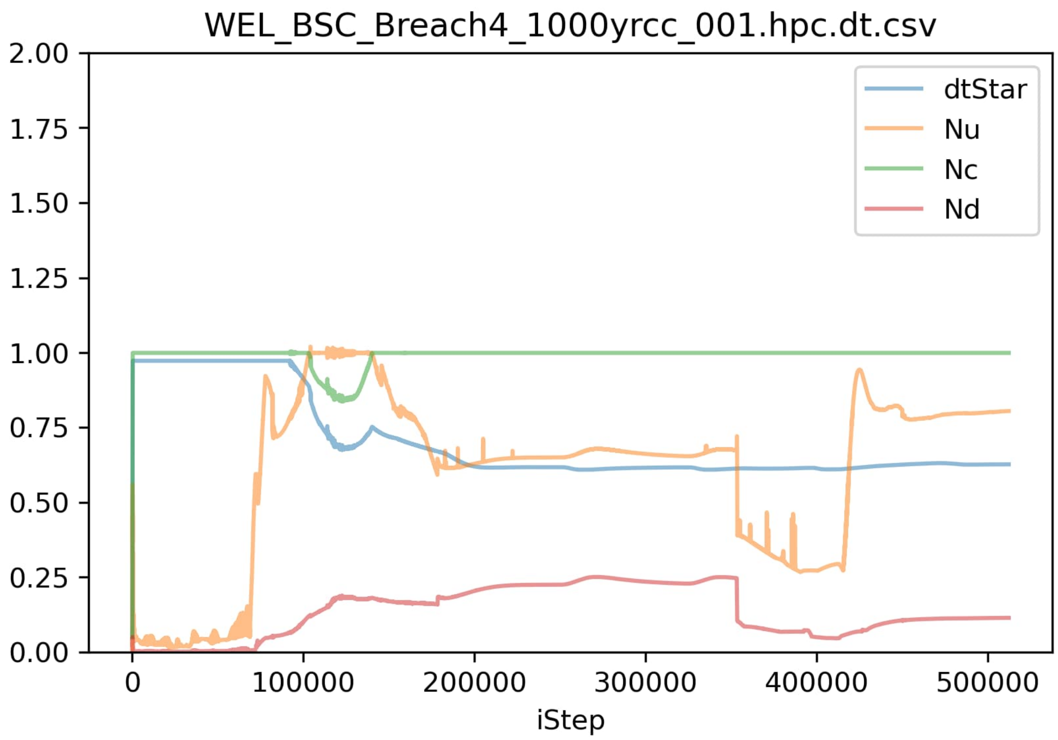


Figure 26: Model stability indicators

9.2 Checks and Warning Messages

A number of Check and Warning messages are present in the TUFLOW log file upon completion of the model runs, which are summarised in Table 11. These have been reviewed and are all considered to be acceptable.

Table 11 – TuFLOW check and warning messages.

ID	Count	Comment
CHECK 1284	27	Connecting a 1D H boundary to 2D HX link.
WARNING 2583	1	Material ID 3 has a manning's n value (0.300) greater than Wu n limit (0.100) - n value will be limited in Wu formulation.

10 Conclusions

This report has detailed the methodology used to develop the River Welland breach hydraulic model. The conclusions from the study are as follows:

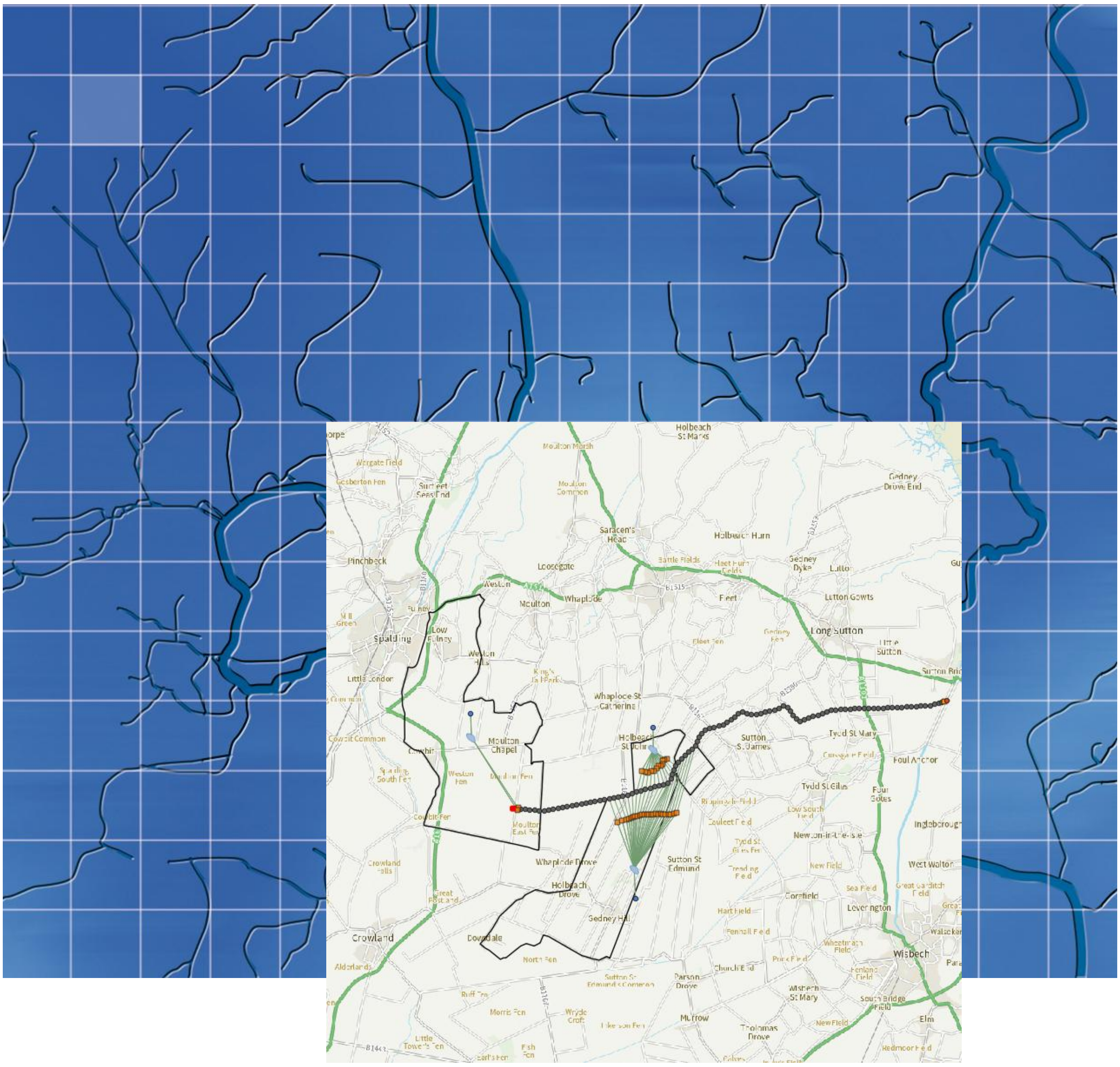
- The hydraulic model has been run for the 1.0%, 1.0% + CC, 0.1% and 0.1% + CC AEP return periods.
- Boundary data from River Welland has been extracted from the existing 1D model provided by the EA.The tidal and fluvial climate change allowances used have been assessed alongside the baseline flow and level outputs to determine that the results remain valid when compared to current climate change allowances.
- The results of the model indicate that a breach in flood defences along the River Welland results in widespread flooding across the southwest of the site located predominantly in PV areas A-1, B-1, B-2, B-3, B-5, and C-2.
- PV Area A-1 is associated with the greatest depth of flooding during breach scenarios 1 to 3, whereas PV Areas B-1, B-3, and B-5 are associated with the greatest depth of flooding during breach scenario 4.

The outputs from this model will be used to inform the placement and height of infrastructure in the Flood Risk Assessment.

AECOM

October 2025

South Holland Main Drain Hydraulic Modelling Report



WHS

AECOM

South Holland Main Drain Hydraulic Modelling Report

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1.0	12/09/25	Draft	AJ (*Consultant*)	BP (*Principal Consultant*)
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For and on behalf of Wallingford HydroSolutions Ltd.

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Contents

1 Introduction

1.1 Scope of Study

Wallingford HydroSolutions Ltd (WHS) has been commissioned by AECOM to undertake detailed hydraulic modelling to support the scheme design for the proposed Meridian Solar Farm (E: 527196, N: 313799). Following agreement with the Environment Agency (EA), the modelling aims to define the flood levels for land parcels A, B, C and D to inform the elevation of flood-sensitive infrastructure.

As part of this work, three hydraulic models will be reviewed or built to assess realistic 'worst-case' flood risk scenarios. One of these is a new one-dimensional (1D) hydraulic model of the South Holland Main Drain (SHMD), including three of the South Holland Internal Drainage Board (SHIDB) pumped catchments:

- Fleet Fen (Catchment E) and Gotts (Catchment S) covering land parcels D1-D6.
- Wisemans (Catchment B) covering land parcel B5.

This report presents the methodology and results from the pumping and pump failure scenarios, which account for the tidal influence from the River Nene. For the pump failure scenario, it assumed the SHIDB pumping stations are offline throughout the design flood events.

1.2 Methodology

As the data held by the SHIDB is outdated and insufficient for defining the design flood levels within land parcels D1-D6 and B5, hydraulic modelling is required to assess peak water levels associated with the baseline scenario (pump on) and a failure of the pumping stations scenario during an extreme rainfall event. Therefore, a 1D model of the SHMD has been developed to assess the flood risk from the design event. This modelling approach and extent were confirmed with the EA[1].

The hydraulic model was developed using Flood Modeller Pro, and the flows within the model were estimated from a hydrological assessment of the three SHMD pumped catchments. The hydrological assessment also considered the tidal influence of the River Nene at the outfall of the SHMD based on data provided by the EA[2]. The 1D model network was constructed based on survey data provided by the SHIDB[3].

The model has been run for a range of events, including the 0.1% annual exceedance probability (AEP) event plus climate change allowance design event. This event has been selected as the design event to determine suitable levels for flood-sensitive infrastructure within the eastern block of land parcels and Parcel B5, in line with the South East Lincolnshire Strategic Flood Risk Assessment (SFRA)[4].

[1] Hydraulic modelling method statement. AECOM. December 2024.
[2] WEM Lot 1 Modelling, Mapping and Data. Tidal Nene Modelling Improvements. JacksonHyder. April 2016.
[3] South Holland Main Drain cross sections and longitudinal sections. Water Management Alliance. Provided July 2025.
[4] South East Lincolnshire Strategic Flood Risk Assessment (SFRA). South East Lincolnshire Joint Strategic Planning Committee. March 2017

1.3 Data sources

The data used to inform the hydraulic modelling process are as follows:

- Model data from the Tidal Nene Modelling Improvements[2]
- River channel sections of the SHMD provided by the Water Management Alliance on behalf of the SHIDB[3].
- Peak flow assessment for SHMD Modelling (attached as Appendix 1).
- LiDAR Data[5] flown in 2022.
- South East Lincolnshire Strategic Flood Risk Assessment (SFRA)
- South Holland IDB Map Index[6]
- South Holland IDB supplied data (see Appendix B)

1.4 Key Limitations and Assumptions

The model was built based on the following assumptions:

- The LiDAR and river section data are both suitable for use to inform the hydraulic model.
- A blockage assessment is not required.
- Sensitivity analysis is appropriate to test model robustness.
- Limited information is provided on the structures. The survey includes some key dimensions on structures within the SHMD, but there is insufficient detail to include them in the model. Therefore, they have been excluded from the model on the basis that this would be a conservative approach, allowing the tidal surge to reach the site unimpeded.
- There is limited information available for the Sutton Bridge sluice, with the only information being that the sill is at -10 ft ODN. Therefore, the sluice has been modelled as closed, as tidal surge would be forecasted, with the overtopping based on the crest levels of adjacent ground, which appears to be at the same level.
- The section data provided was for silted bed levels, whereas hard bed levels are normally used to inform the channel bed. The silted bed level is a conservative assumption representing the loss in channel capacity.
- The peak flow estimates are for the Fleet Fen, Gotts and Wisemans pumped catchments. However, levels in the SHMD will, to some extent, be influenced by the pumped discharges from adjacent catchments. As the adjacent catchments are not being modelled, levels in the SHMD will be mainly based on the tidal surge levels in the Nene overtopping the closed sluice.

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[5] National LiDAR Programme. Environment Agency. Updated: August 2025.
[6] SHIDB_webmap2020 - South Holland IDB Map Index. Water Management Alliance. February 2023.

2 Site Description

2.1 Site Location

Figure 1 shows the location and land use for the proposed Meridian Solar Farm development in Lincolnshire, England, primarily within the South Holland District. The proposed solar farm development is spread across four main land parcels labelled A, B, C and D. The site is located east of the River Welland, with the solar farm infrastructure situated between the towns of Spalding (to the northwest) and Holbeach (to the northeast). The SHMD flows westerly through the northern section of land parcels B and D.

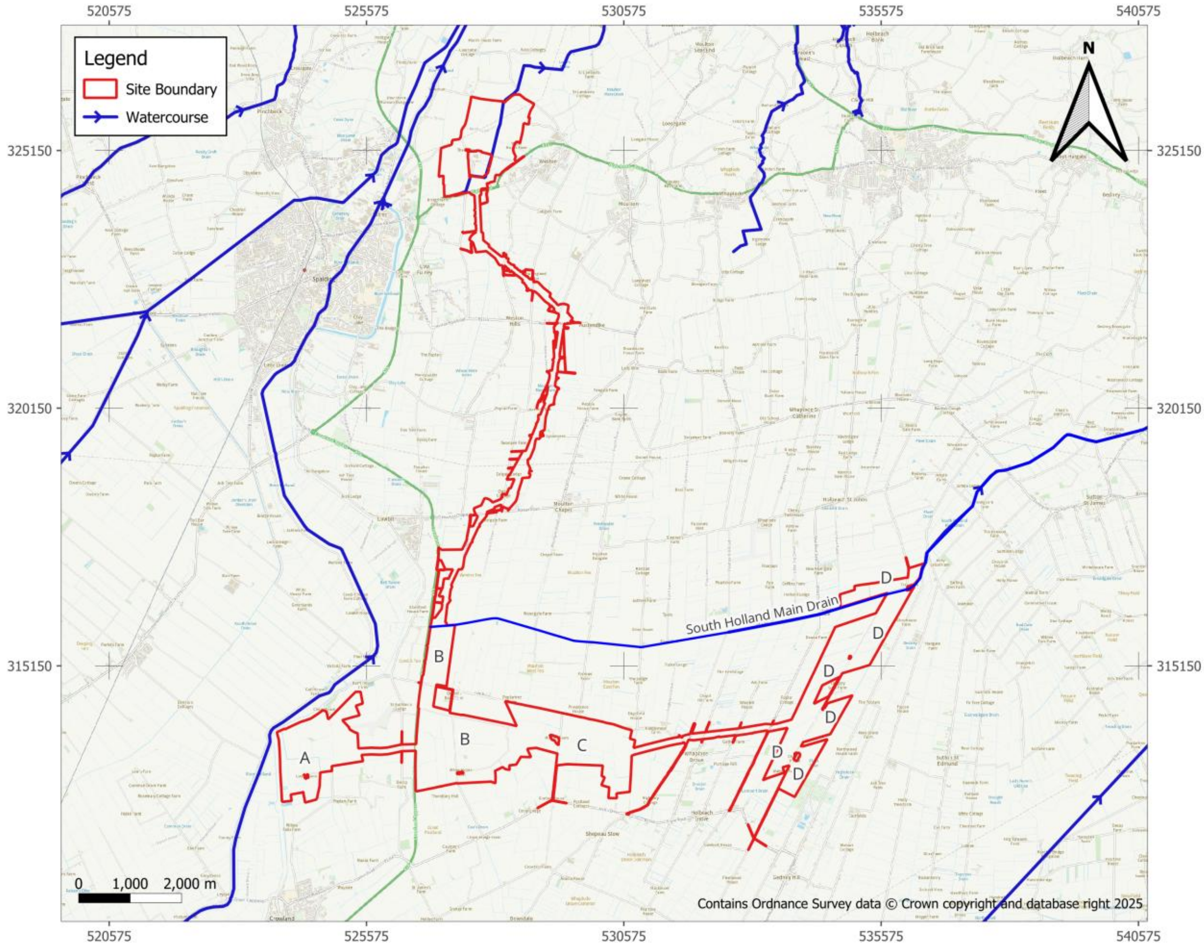


Figure 1 – Site Location

Figure 2 shows the Fleet Fen and Gotts catchments, which are the SHIDB catchments containing land parcels D1-D6 which are proposed to contain PV panel areas. It also shows the Wisemans pumped catchment containing land parcel B5, where the 400 kV substation and BESS compound are to be located.

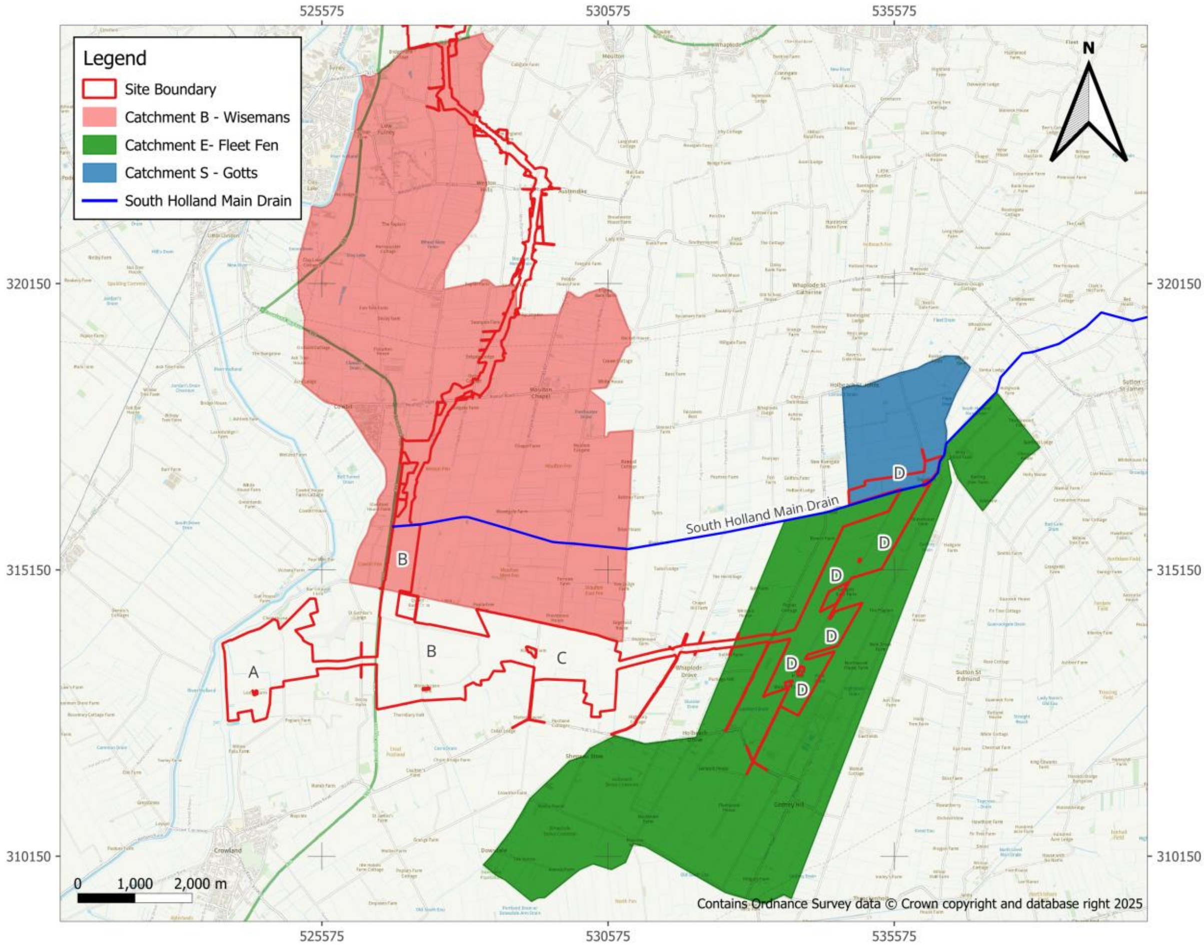


Figure 2 – Pump catchments relative to land parcels.

2.2 Historic Flooding

The EA's Historic Flood Map[7] indicates that there have been no recorded flood events within the catchments and at Parcels B5 and D1-D6.

2.3 Existing Flood Defences

The EA's Asset Management Database[8], which includes flood defence locations, indicates the Site is within an area where flood risk is reduced due to coastal defences along the coastline and the River Nene.

[7] Historic Flood Map. Environment Agency. January 2024.
[8] Asset Management. Environment Agency. Available at: https://environment.data.gov.uk/asset-management/index.html

3 Hydrology Assessment

The downstream boundary of the model is the SHMD's confluence with the River Nene. To capture the tidal influence from the Nene, a tidal boundary will be used based on modelled levels in the main Nene channel.

For the fluvial flood risk, a catchment has been defined for the SHMD at the confluence's location. The Fleet Fen and Gotts catchments are located in the southern section of the defined catchment, and the Wisemans catchment is located along its western boundary, as shown in Figure 3. The catchments have been modelled hydrologically using ReFH 2.3 software to estimate the peak flows and hydrographs for input into the hydraulic model. The peak flow estimates for the pumped catchments have been derived by adopting the same catchment descriptors as the identified SHMD catchment, except for area, which has been adjusted to match the areas for each pumped catchment. This approach is preferred over deriving catchment descriptors specific to each pumped catchment, as flows into them will be primarily influenced by the surrounding wider catchment. Therefore, adopting the wider SHMD catchment descriptors ensures consistency with the regional hydrological behaviour and compensates for the limited data available at the local scale.

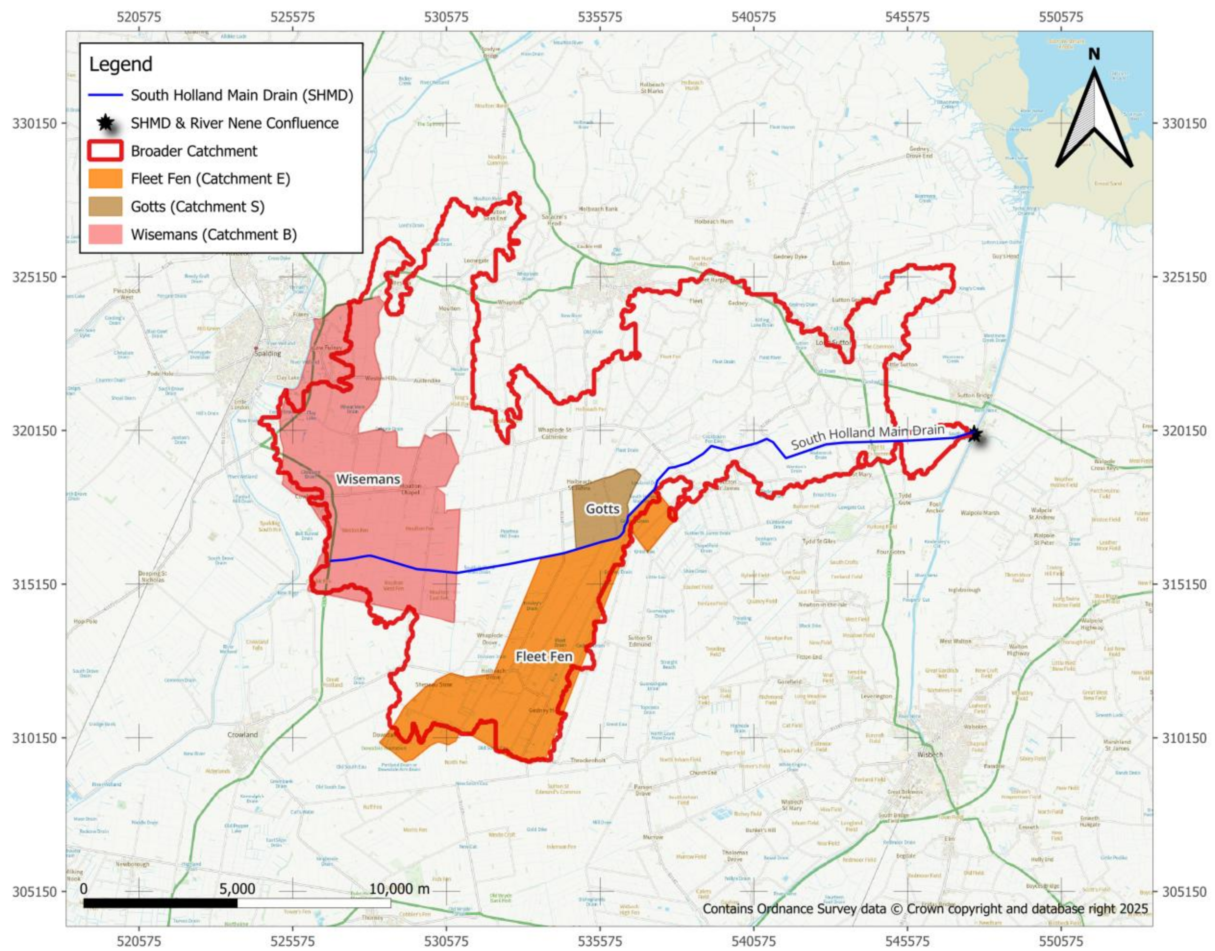


Figure 3 – Defined catchment and IDB Pumped Catchment

The full methodology used to derive the peak flows for the model is detailed in the hydrology report attached as Appendix 1. The final peak flows for the SHIDB pumped catchments are presented in Table 1.

Table 1 – Peak flow estimates for Gotts, Fleet Fen and Wisemans catchments

Return Period (years)	Peak Flow estimate (m^3/s)		
	Fleet Fen	Gotts	Wisemans
2	1.179	0.244	1.581
25	2.775	0.592	3.717
50	3.371	0.713	4.515
100	4.148	0.859	5.556
500	6.772	1.358	9.075
1000	8.147	1.642	10.921

4 Hydraulic Model Build

4.1 1D Domain

The 1D domain was modelled using Flood Modeller Pro (FMP) and was informed by river section data provided by the SHIDB[3].

4.1.1 Model Extent

Figure 4 shows the 1D model extent for the hydraulic model. The SHMD has been modelled as an open channel approximately 19.59 km from its upstream node (NGR: 529977, 315606) to the downstream node (NGR: 547617, 320055). The model's downstream boundary has been taken as the confluence of the SHMD and the River Nene (NGR: 547769, 320102) to account for the tidal influence of the Nene at the outfall of the drain.

The 1D model extent includes the Fleet Fen and Gotts SHIDB catchments, represented as reservoir/storage units. The reservoir units were defined using the SHIDB catchment boundaries and the latest LiDAR data[5], which allowed the software to calculate the volume and surface area of water that would accumulate at various elevations (i.e., floodplain storage capacity at each level) within each catchment boundary.

The Wisemans catchment and the extent of the SHMD from the sluice at the Wisemans Pumping Station up to the A16 Crowland Bypass were modelled as a single reservoir unit due to the hydraulic connectivity of the catchment.

The final representation of the 1D model was a 1D channel with spills connecting to the floodplain storage areas. For Fleet Fen and Gotts catchments, the spills have been set to the top of bank levels for the river sections directly adjacent to the pumped catchments. For the Wisemans catchment, a single spill was used with its level set to the estimated top of sluice level for the Wisemans Pumping Station sluice.

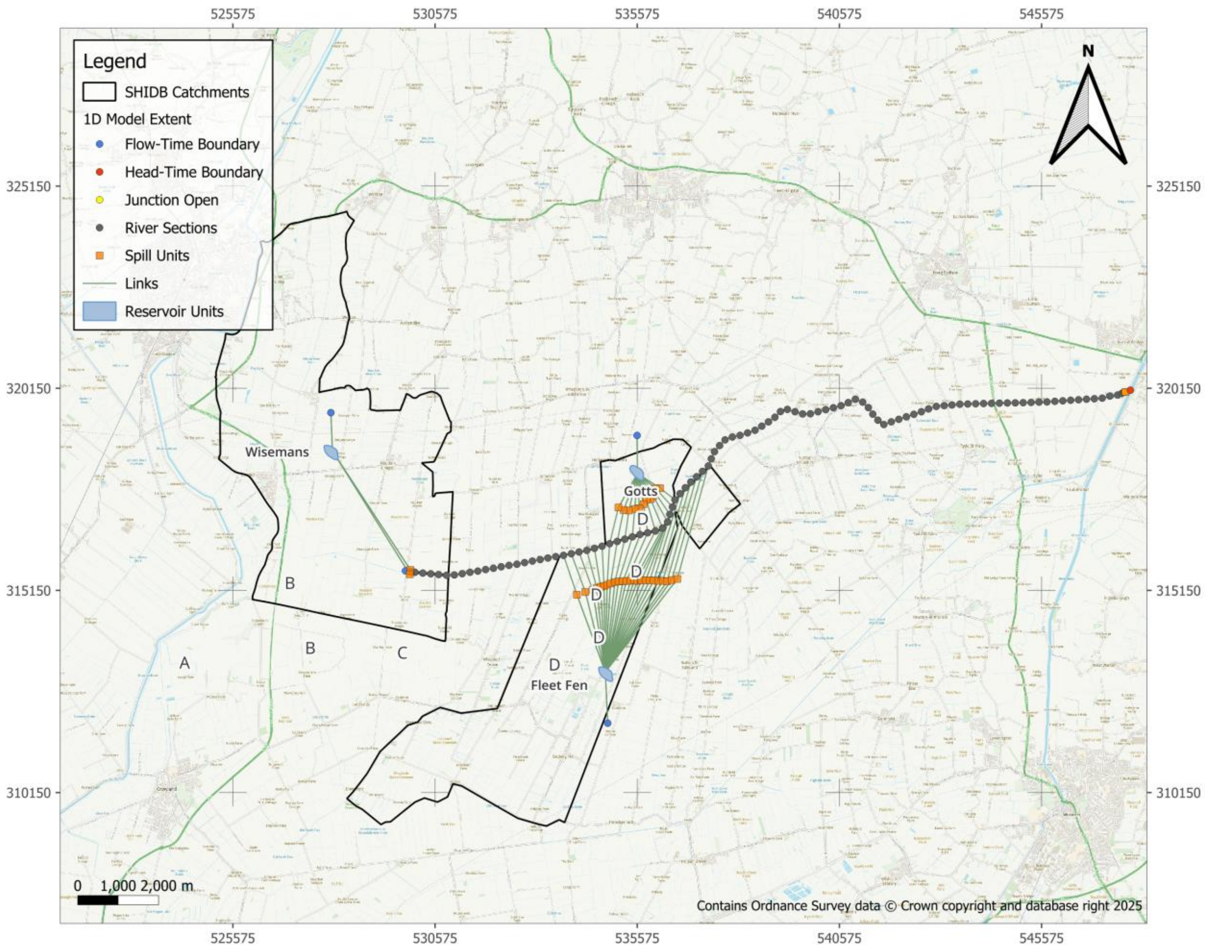


Figure 4 – 1D Model Extent

4.1.2 River Channel Geometry

The channel geometry of the model has been informed by data provided by the SHIDB in July 2025, however the data itself was captured in 2012. The data was provided as drawing files (.dxf, and .dwg) and in tabular format in Excel. The tabular data included the cross-sections with grid reference and elevation data. The Excel data was converted to a comma-delimited file (.csv) and imported into FMP as cross sections. The section data provided was for silted bed levels, whereas bed levels are normally informed by hard bed levels. However, using the silted bed levels is a conservative approach as channel capacity is lost.

4.1.3 Upstream Boundary Conditions

The inflows into the 1D model were applied directly to the reservoir units using Flow-Time (QT) boundaries.

4.1.4 Downstream Boundary Conditions

A Head-Time (HT) boundary was used as the downstream tidal boundary to model the influence of the River Nene at the outfall of the SHMD. The levels for the HT boundary were extracted from the Tidal Nene Improvement Modelling provided by the EA. The extracted tidal peak flows from the Nene, which started at the outfall of the SHMD at 135 hrs, were shifted to coincide with the peak river inflows to the catchments. The shifted HT boundary used for the design event is shown in Figure 5.

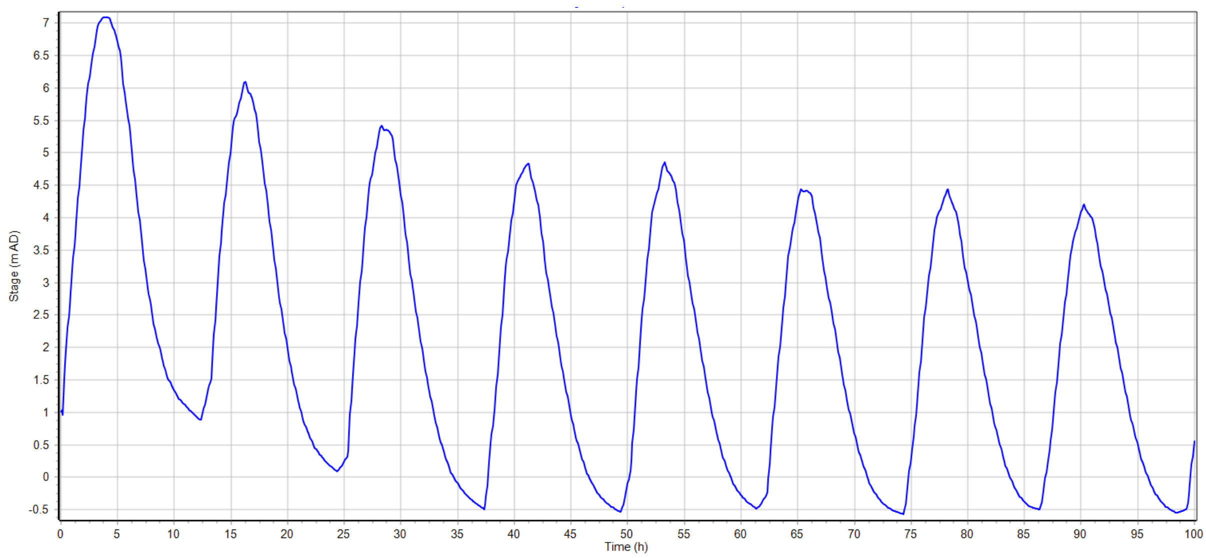


Figure 5 – HT Boundary at outfall of the SHMD for the 0.1% AEP + climate change design event

4.1.5 Initial Conditions

Initial conditions were established throughout the model by calculating initial water levels in the channel before a storm event. The water levels were calculated by first modelling a steady baseflow, which is the flow that occurs in the first timestep of the 0.1% AEP event hydrograph.

The HT downstream boundary was also used to establish bank-full conditions as part of the initial conditions. For this, the downstream boundary was set to a level of 1.0 m AOD, which was the average bank-full level throughout the SHMD channel, representing conservative initial conditions.

4.1.6 River Channel Roughness

The Manning's values used to represent the surface roughness are summarised in Table 2. The values were selected based on engineering judgement and were informed by aerial imagery and published guideline values[9]. Based on the aerial and street view imagery (see Figure 6):

- The 'in bank' section of the channel was classified as a clean, straight, full stage, no riffs or deep pools main channel.
- The out-of-bank sections were classified as pasture, no brush, and short grass. These sections, which formed part of the floodplains, were established by selecting appropriate channel and deactivation markers to specify the top of bank levels and deactivate the out-of-bank part of the cross sections.

Table 2 - Summary of Manning's Values

Domain	Description	Manning's n
1D	Drainage channel – in bank	0.030
1D	Drainage channel – out of bank	0.035

Figure 6 – Street View Image of SHMD, taken from Google Street View[10].

[9] Chow, V T (1959). *Open-channel Hydraulics.* McGraw-Hill.
[10] Street View of South Holland Main Drain. Google Maps, Google. Accessed: August 2025. Available at: https://www.google.com/maps

4.1.7 River Channel Structures

As outlined in the section 1.4, the information available for the structures was limited, and the details were insufficient to include them in the model. For this specific assessment, excluding them from the model is a conservative approach, as this allows the tidal surge to reach the Site unimpeded.

Similarly, insufficient information on the dimensions for the Sutton Bridge sluice was provided, as only the sill level at -10 feet ODN was provided. Therefore, the sluice has been modelled in the closed position, as a tidal surge would be forecasted.

Sufficient information was also unavailable for the sluice at the Wiseman's Pumping Station, which is located on the east side of Eaugate Road. Therefore, crest levels for the sluice were informed using LiDAR data of the adjacent banks.

Figure 7 – Street View Image of Wiseman's Pumping Station Sluice, taken from Google Street View[10].

For the more extreme flood events, overtopping of the sluice at Sutton Bridge is modelled based on the LiDAR levels of the adjacent ground, which appear to be at the same level as the sluice crest, see Figure 6 in section 4.1.6 above.

5 Design Runs

5.1 Summary of Design Run

The model has been run for each of the joint fluvial/tidal events below to produce baseline and pump failure results, to determine peak water levels within land parcels B5 and D.

- 1.0% annual exceedance probability (AEP) fluvial event applied to the pumped catchments / 0.5% AEP tidal event from the River Nene.
- 1.0% AEP + a 13% climate change allowance applied to the pumped catchments / 0.5% AEP event + climate change horizon from the River Nene*.
- 0.1% AEP + 13% climate change allowance applied to the pumped catchments / tidal 0.1% AEP event + climate change horizon from the River Nene*, i.e., the worst-case design event.

The climate change allowance has been applied following the EA's guidance[11] for using peak river flow allowances for flood risk assessments. A higher-end climate change allowance of 13% for the 2080s epoch was applied for the Nene Management Catchment.

*The best available data was used for the climate change horizon from the River Nene, (i.e., the 2016 results from the Tidal Nene Modelling Improvements). However, it is not clearly reported which climate change epoch was used for the 2016 Tidal Nene modelling.

5.1.1 Baseline Scenarios

The baseline scenarios represent the baseline conditions within the pumped catchments during the design runs when the pumps are operational. The pumps were modelled using abstraction units, with a defined discharge withdrawn from the reservoir units (i.e., the pumped catchments) and conveyed at a constant rate into the SHMD. The discharge rates for the catchments are shown in Table 3 below, and the pumping stations are in Appendix 2.

Table 3 – South Holland Drainage Board Pumping Stations for Fleet Fen and Gotts Catchment[12]

Station	Grid Reference	Lift (m) Max/Duty	No. of Pumps	Pumping rate (m^3/s)
Fleet Fen	536250, 316700	3.1/1.5	2	1.70
Gotts	536430, 317270	2.1/1.8	1	0.65
Wisemans	529970, 315620	3.2/2.6	4	1.34

For the baseline scenarios, the pumping operations were controlled by the following rules within each of the pumped catchments:

1. If inflows within the Fleet Fen catchment are < 3.4 m^3/s (i.e., the combined pumping rates), then inflows are pumped out. If inflows > 3.4 m^3/s, then pumps will operate at the combined pump rate.

[11] Peak river flow climate change allowances by management catchment. Environment Agency. February 2022.
[12] South Holland Drainage Board Pumping Stations. Water Management Alliance. Available at: https://www.wlma.org.uk/uploads/SHIDB_Pumping_Stations.pdf

2. If inflows within the Gotts catchment are < 0.6 m^3/s, then inflows are pumped out. If inflows > 0.6 m^3/s, then pumps will operate at the pumping rate.
3. If inflows within the Wisemans catchment are < 5.36 m^3/s (i.e., the combined pumping rates), then inflows are pumped out. If inflows > 5.36 m^3/s, then pumps will operate at the combined pump rate.

5.1.2 Pump Failure Scenario

As outlined in section 1.1, the pump failure simulation assesses a worst-case scenario, where the pumps in the pumped catchments are non-operational for the entire design run. For the pump failure scenarios, the pumps were excluded from the model.

5.1.3 Post-development Scenario

Figure 8 shows the sections of the proposed development located within Parcel D in relation to the SHIDB catchments. These were incorporated into the model to create the post-development scenario to determine the impact that the proposed development has on flood risk. The post-development infrastructure incorporated into the model includes the following:

- Solar Stations
- Substations

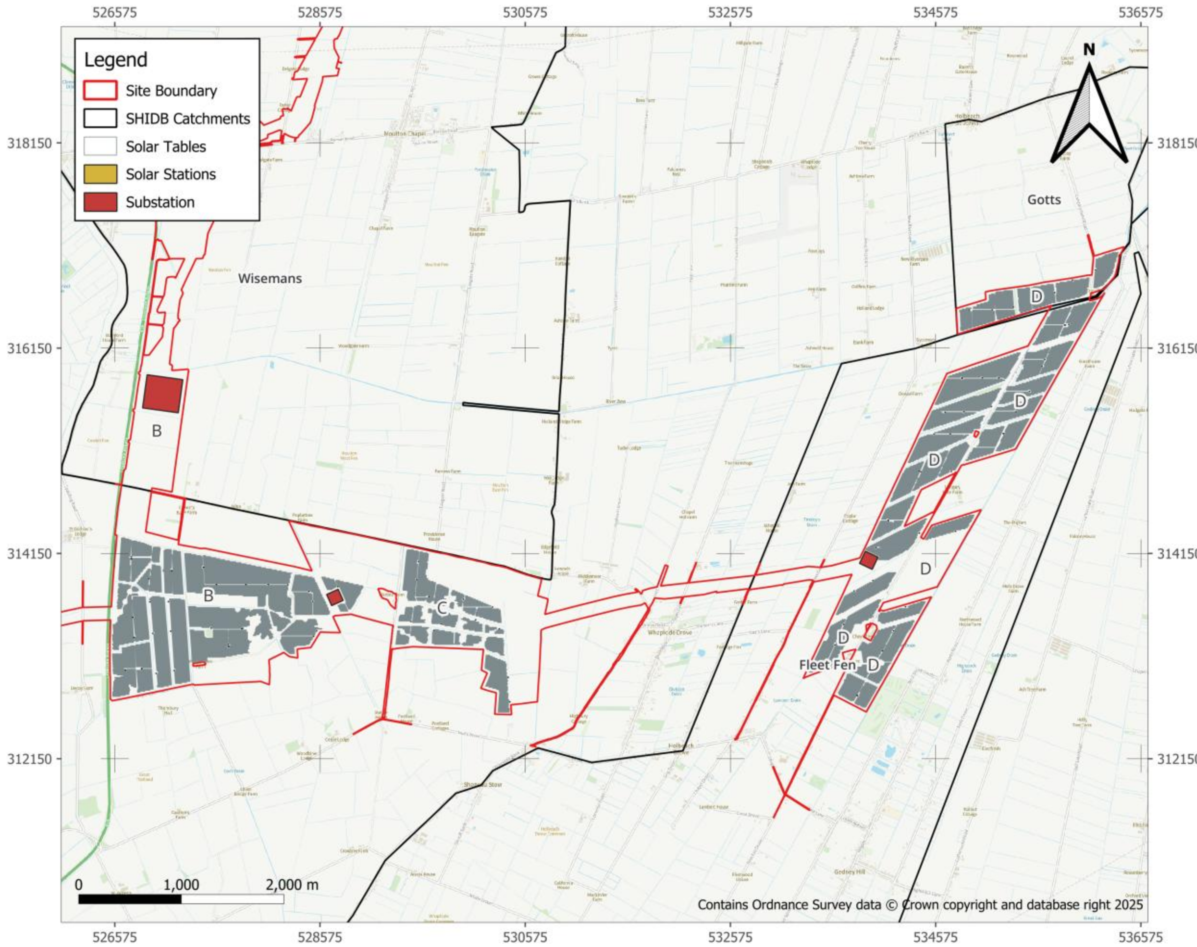


Figure 8 – Proposed infrastructure within land parcels B5 and D1-6

To create the post-development scenario, the solar stations were buffered by 20 m, representing the worst-case flood protection zone. The land-take requirement for the buffered zones and the substation was then used to adjust the storage area of the reservoir units. This represented the SHIDB catchments with the applied storage loss due to the solar stations, including the worst-case flood protection zone, and the substation. The storage loss resulting from the proposed infrastructure was calculated at the elevation-band level for each catchment, using the final infrastructure levels, i.e., the maximum ground level at the structure plus the proposed raising for flood protection.

The baseline and post-development plan areas, along with the total area loss for the reservoir units, are presented in Table 4 and Figure 9 for Fleet Fen, in Table 5 and Figure 10 for Gotts, and in Table 6 and Figure 11 for Wisemans.

Table 4 – Baseline and Post-development elevation and storage area for the Fleet Fen catchment

Elevation (m AOD)	Plan Area (m^2)		
	Baseline	Total Area Loss	Post-development
-1.043	1	0	1
-0.705	80,534	0	80,534
-0.366	133,913	0	133,913
-0.028	205,493	0	205,493
0.31	303,238	0	303,238
0.649	673,092	0	673,092
0.987	4,084,000	1,591	4,082,409
1.326	11,130,000	9,018	11,120,982
1.664	16,870,000	25,541	16,844,459
2.002	21,010,000	59,728	20,950,272
2.341	23,960,000	51,311	23,908,689
2.679	25,410,000	51,311	25,358,689
3.017	25,650,000	51,311	25,598,689
3.356	25,700,000	48,128	25,651,872
3.694	25,720,000	37,519	25,682,481
4.032	25,720,000	25,770	25,694,230
4.371	25,720,000	9,536	25,710,464
4.709	25,720,000	0	25,720,000
5.047	25,720,000	0	25,720,000
5.386	25,720,000	0	25,720,000
5.724	25,730,000	0	25,730,000
6.063	25,730,000	0	25,730,000
6.401	25,730,000	0	25,730,000
6.739	25,730,000	0	25,730,000
7.078	25,730,000	0	25,730,000
7.416	25,730,000	0	25,730,000
10.416	25,730,000	0	25,730,000

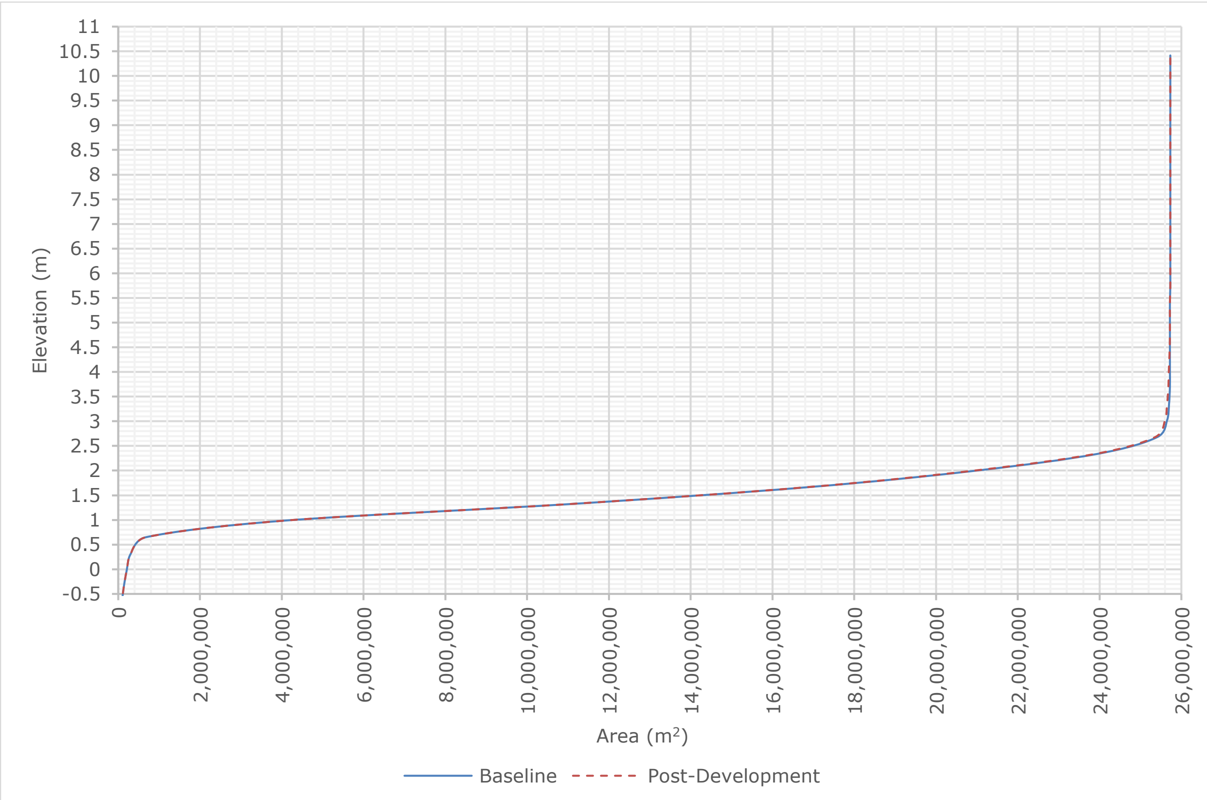


Figure 9 – Baseline vs Post-development Elevation-Area Curve for Fleet Fen

Table 5 – Baseline and Post-development elevation and storage area for the Gotts catchment

Elevation (m AOD)	Plan Area (m^2)		
	Baseline	Total Area Loss	Post-development
-0.24	148	0	148
-0.034	7,148	0	7,148
0.172	13,960	0	13,960
0.378	21,847	0	21,847
0.585	34,576	0	34,576
0.791	60,109	0	60,109
0.997	295,278	0	295,278
1.203	1,085,000	1,591	1,083,409
1.409	2,102,000	7,956	2,094,044
1.615	2,819,000	9,547	2,809,453
1.822	3,211,000	11,139	3,199,861
2.028	3,457,000	11,139	3,445,861
2.234	3,679,000	11,139	3,667,861
2.44	3,750,000	11,139	3,738,861
2.646	3,766,000	11,139	3,754,861
2.852	3,770,000	11,139	3,758,861
3.059	3,772,000	11,139	3,760,861
3.265	3,775,000	11,139	3,763,861
3.471	3,775,000	9,547	3,765,453
3.677	3,776,000	3,182	3,772,818
3.883	3,776,000	3,182	3,772,818
4.089	3,776,000	0	3,776,000
4.296	3,776,000	0	3,776,000
4.502	3,776,000	0	3,776,000
4.708	3,777,000	0	3,777,000
4.914	3,777,000	0	3,777,000
7.914	3,777,000	0	3,777,000

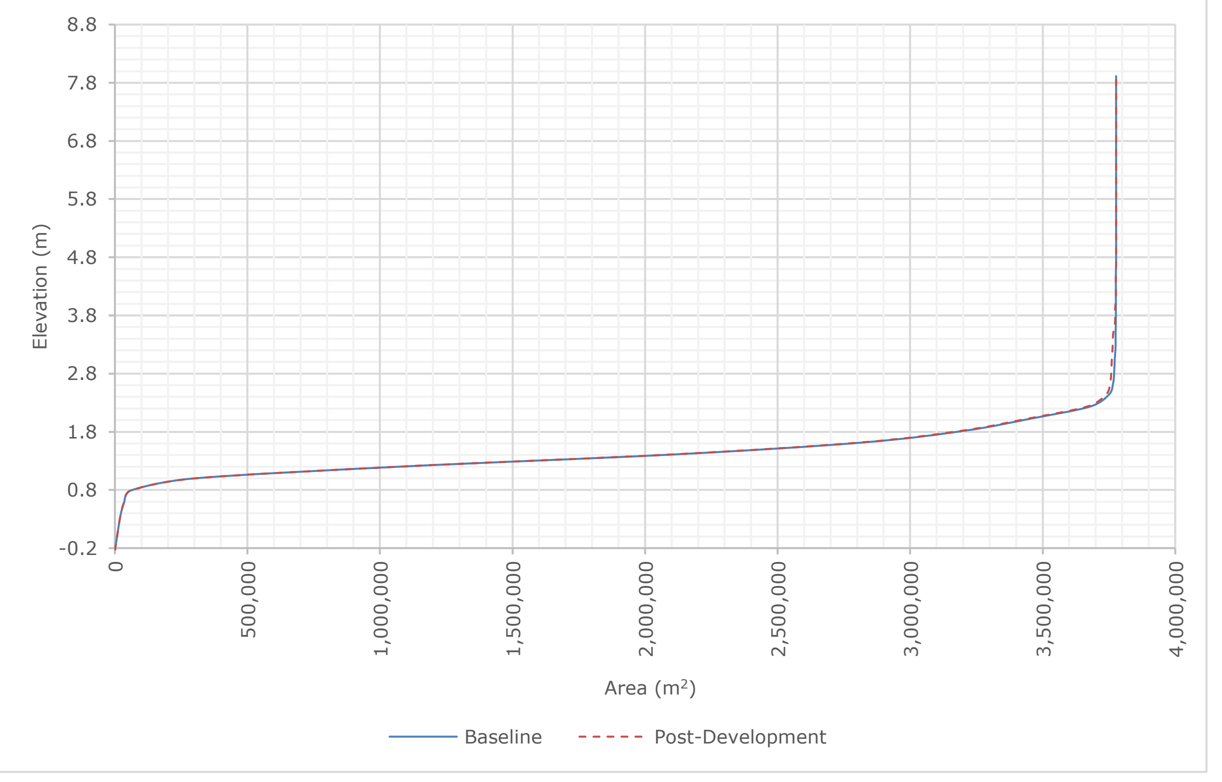


Figure 10 – Baseline vs Post-development Elevation-Area Curve for Gotts

Table 6 – Baseline and Post-development elevation and storage area for the Wisemans catchment

Elevation (m AOD)	Plan Area (m^2)		
	Baseline	Total Area Loss	Post-development
-0.69	57	0	57
-0.341	45091	0	45,091
0.007	76611	0	76,611
0.356	118010	0	118,010
0.705	210358	0	210,358
1.054	1,176,000	0	1,176,000
1.402	3,495,000	0	3,495,000
1.751	7,420,000	0	7,420,000
2.1	12,510,000	114,472	12,395,528
2.448	20,310,000	114,472	20,195,528
2.797	31,050,000	114,472	30,935,528
3.146	35,710,000	114,472	35,595,528
3.495	36,750,000	114,472	36,635,528
3.843	37,010,000	0	37,010,000
4.192	37,080,000	0	37,080,000
4.541	37,130,000	0	37,130,000
4.89	37,150,000	0	37,150,000
5.238	37,170,000	0	37,170,000
5.587	37,190,000	0	37,190,000
5.936	37,210,000	0	37,210,000
6.284	37,210,000	0	37,210,000
6.633	37,210,000	0	37,210,000
6.982	37,210,000	0	37,210,000
7.331	37,210,000	0	37,210,000
7.679	37,210,000	0	37,210,000
8.028	37,210,000	0	37,210,000
11.028	37,210,000	0	37,210,000

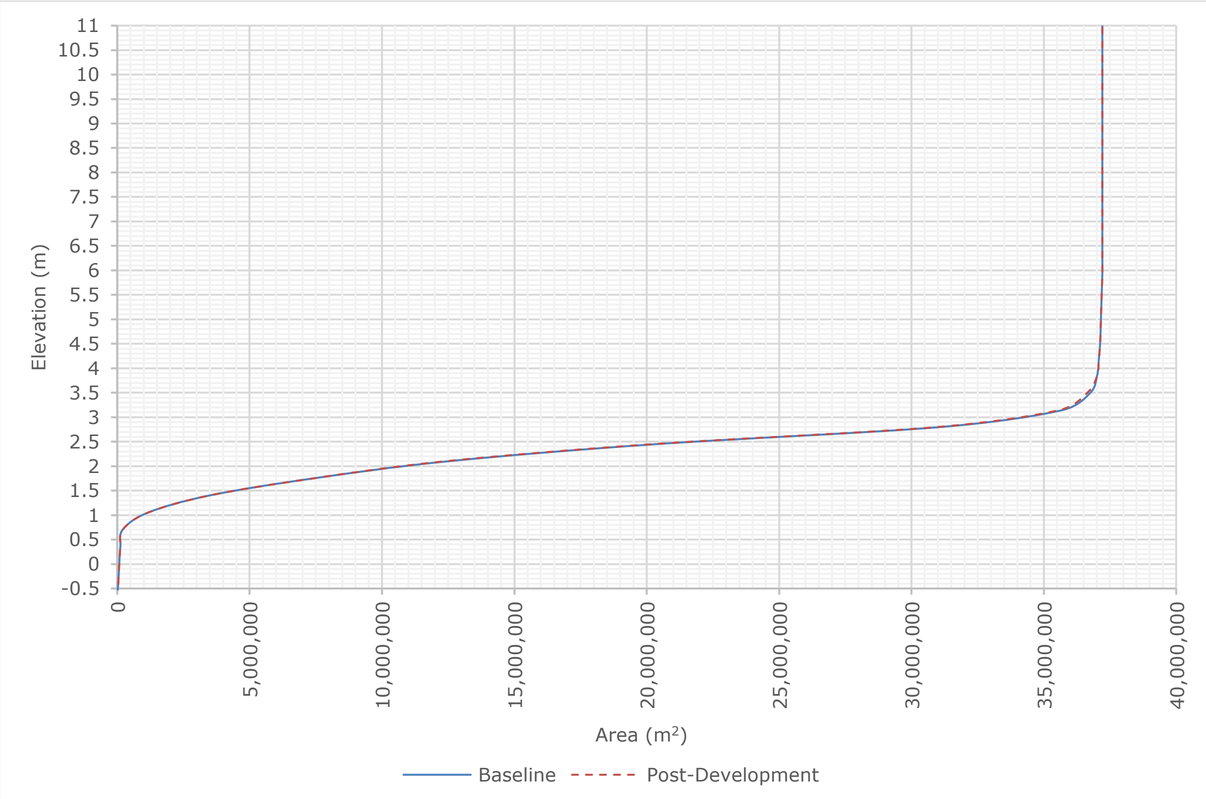


Figure 11 - Baseline vs Post-development Elevation-Area Curve for Wisemans

6 Model Results

6.1 Summary of Modelled Flood Levels

A summary of the modelled flood levels during the design runs is provided in Table 7 to Table 9, which shows the flood level within the 1D reservoir unit for each pumped sub-catchment under the different scenarios modelled.

In addition to the model flood levels, the flood maps for the 0.1% AEP plus climate change fluvial boundary in combination with a 0.1% AEP plus climate change tidal boundary are presented in Section 6.2 to 6.4.

Table 7 –Summary of Modelled Flood Levels during the 1.0% AEP Fluvial with 0.5% Tidal Event

Station	Baseline Flood Level (m AOD)	Pump Failure Flood Level (m AOD)	Post Development Flood Level (Pump Failure) (m AOD)
Fleet Fen	-0.634	0.869	0.870
Gotts	1.644	1.106	1.106
Wisemans	0.027	1.383	1.383

Table 8 –Summary of Modelled Flood Levels during the 1.0% AEP + 13% CC Fluvial with 0.5% + CC Tidal Event

Station	Baseline Flood Level (m AOD)	Pump Failure Flood Level (m AOD)	Post Development Flood Level (Pump Failure) (m AOD)
Fleet Fen	-0.234	0.901	0.901
Gotts	1.802	1.126	1.126
Wisemans	0.252	1.423	1.423

Table 9 –Summary of Modelled Flood Levels during the 0.1% AEP + 13% CC Fluvial with 0.1% + CC Tidal Event

Station	Baseline Flood Level (m AOD)	Pump Failure Flood Level (m AOD)	Post Development Flood Level (Pump Failure) (m AOD)
Fleet Fen	1.137	1.09	1.09
Gotts	1.851	1.254	1.254
Wisemans	1.139	1.662	1.662

6.2 Baseline Model Results

Figure *12* shows the maximum modelled flood depths for the 0.1% AEP plus climate change design event for the baseline scenario when the pumps are operational. The modelled extent indicates a significant risk of flooding during the design event, even with the pumps operational within the Fleet Fen and Gotts catchments. This is due to the conservative initial bank-full condition; as floodwater is pumped from the catchments, it enters the SHMD, which is already at capacity and subsequently overtops back into the catchments.

For the Gotts catchment, the increased flood risk during the pumping scenario is significantly higher than when the pumps are off due to the pumping operations within the Fleet catchment, where two pumps discharge into the SHMD adjacent to the Gotts catchment. The water levels pumped from Fleet Fen into the bank-full SHMD rapidly overtop into the Gotts catchment due to its lower bank crest levels. This flooding mechanism with the Gotts catchment was confirmed through sensitivity analysis in which the Fleet Fen pumps were turned off, and the model was rerun with only the Gotts pump in operation.

For the Wisemans catchment, the results show that flood risks are reduced when the pumps are operational within the catchment due to the high bank levels at the sluice at the Wisemans pumping station.

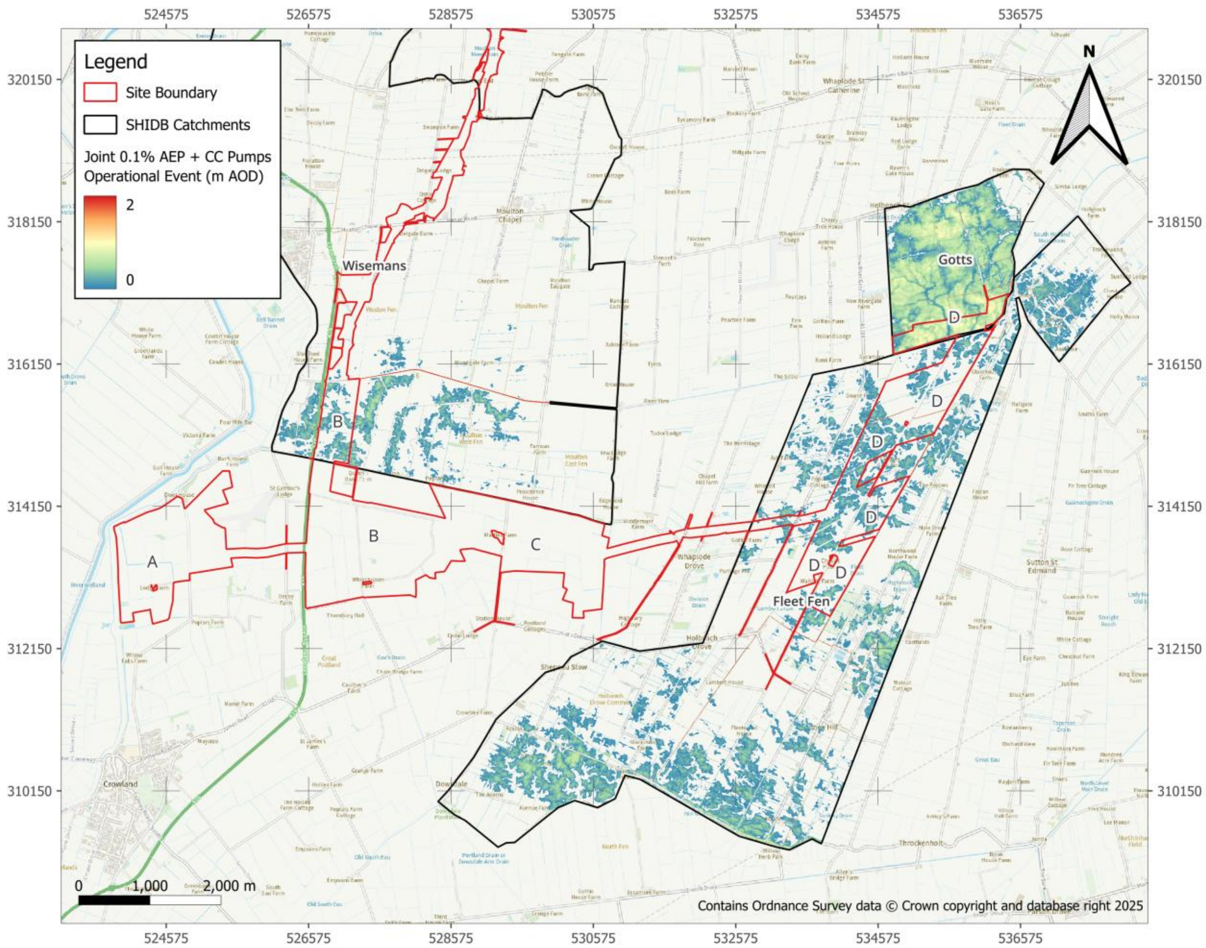


Figure 12 – Baseline joint 0.1% AEP + CC model results

6.3 Pump Failure

The modelled flood depths and extent for the pump failure scenario during the 0.1% AEP plus climate change are shown in Figure 13. The model results indicate that the risk of flooding to the development within Parcel D is mostly concentrated around the central sections of the site within the Fleet Fen catchment. In this area, flood depths > 1.5 m within the drainage channels. Areas of Parcel D along the northern site boundary which fall within the Gotts catchment are also shown to be at significant risk of flooding, as this is a lower-lying area. However, the maximum modelled flood depth within the PV area reaches up to approximately 0.5 m.

Generally, the flooding within the site boundary appears to follow the drainage channels and low-lying farmland, indicating the site is highly dependent on pumping infrastructure for flood management.

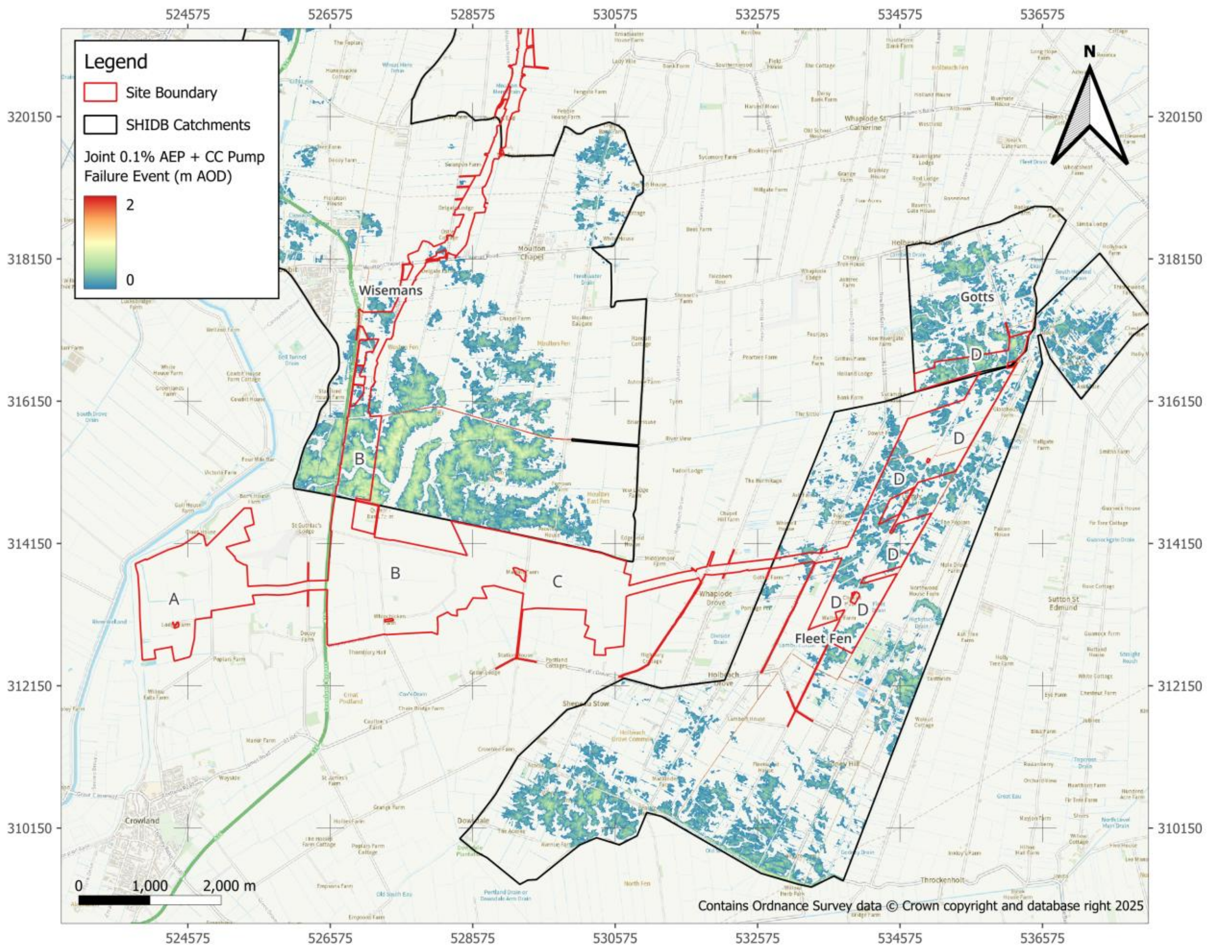


Figure 13 – Pump failure maximum modelled flood depths for design event (joint 0.1% AEP + CC events)

6.4 Post-Development Scenario

The post-development model scenario results (with pump failure) are shown in Figure 14. The results were generated from the reduced flood storage area for the reservoir units, as outlined above in section 5.1.3.

The post-development results show a negligible change in flood risk when accounting for the calculated storage loss, as the loss of floodplain storage due to the solar stations and substation is negligible when compared to the overall storage area of the catchments. This is shown by the elevation-area curves (Figure 9 and Figure 10) in section 5.1.3 above.

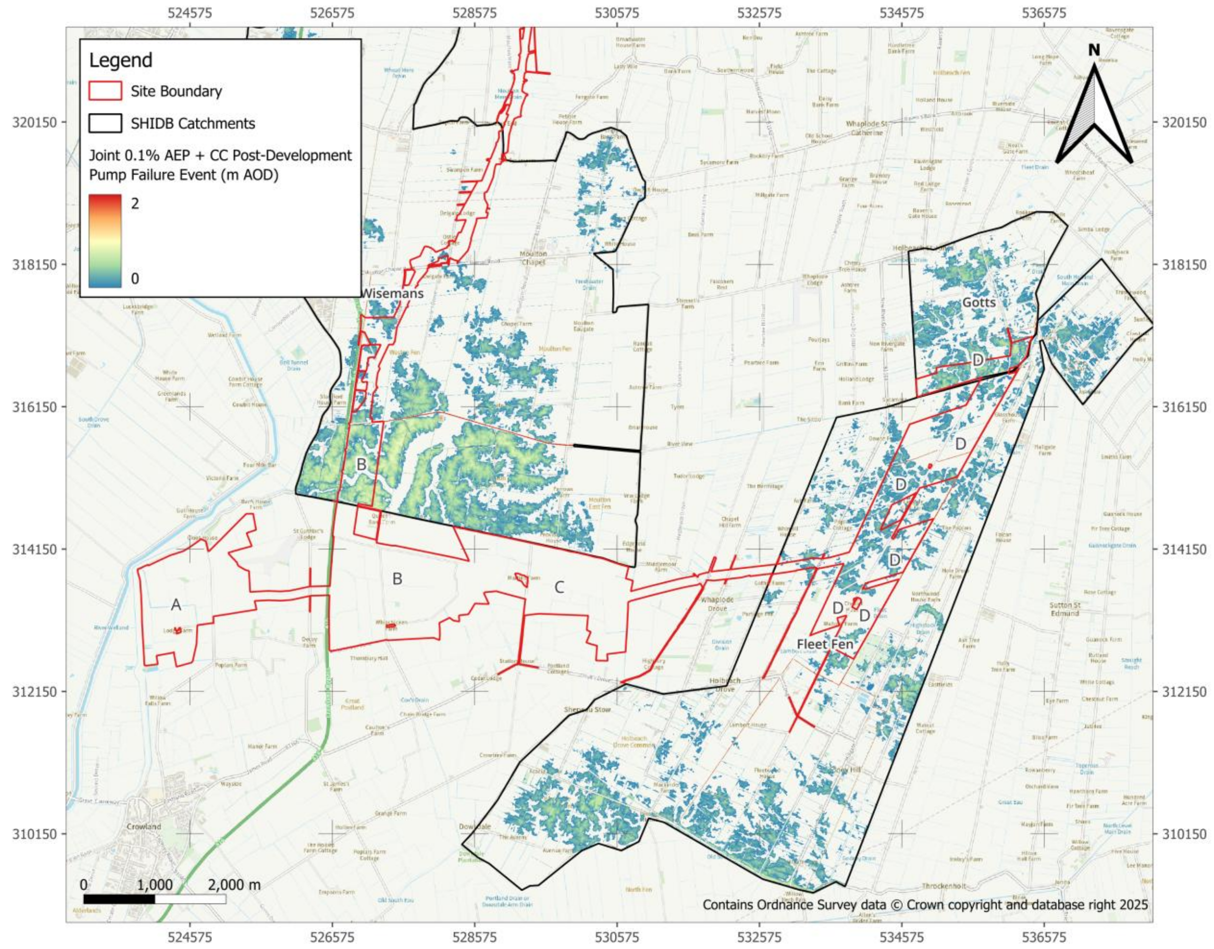


Figure 14 – Post-development model results

7 Sensitivity Analysis

Sensitivity analysis has been undertaken on the 0.1% AEP + 13% climate change allowance fluvial and 0.1% AEP + climate change tidal event, assessing the following parameters:

- Manning’s n value: Roughness coefficients for the channel and floodplain (out-of-bank section of the channel) have been increased and decreased by ± 20%, respectively. The sensitivity of the model to changes in the Manning’s n value was assessed using the joint 0.1% AEP + climate change design event.
- Downstream boundary water level: HT tidal boundary for the joint 0.1% AEP plus climate change event increased and decreased by ± 0.5 m, respectively.

7.1 Manning’s n Value

The results of the sensitivity analysis for Manning’s n value are shown in Table 10 below.

Table 10 – Manning’s n value sensitivity analysis

Catchment	Sensitivity Events	Levels
Fleet Fen	**0.1% AEP + CC**	**1.142**
	SEN(N+)	1.142
	Difference: (m)	-
	SEN(N-)	1.141
	Difference	-0.001
Gotts	**0.1% AEP + CC**	**1.365**
	SEN(N+)	1.363
	Difference (m)	-0.002
	SEN(N-)	1.367
	Difference (m)	0.002

The results of the sensitivity analysis on the channel roughness indicate that the model is not sensitive to the choice of Mannings n.

7.2 Downstream Boundary Water Level

The results of the sensitivity analysis for the downstream water level are shown in Table 11 below.

Table 11 – Downstream boundary (DSbdy) sensitivity analysis

Catchment	Sensitivity Events	Levels
Fleet Fen	**0.1% AEP + CC**	**1.142**
	DSBdy(-)	1.137
	Difference: (m)	-0.005
	DSBdy(+)	1.185
	Difference	0.043
Gotts	**0.1% AEP + CC**	**1.366**
	DSBdy(-)	1.254
	Difference: (m)	-0.112
	DSBdy(+)	1.554
	Difference (m)	0.188

The sensitivity analysis on the downstream HT tidal boundary indicated a varied response between the two catchments. The results show that:

- Gotts catchment demonstrated some sensitivity to the boundary condition. The changes in flood level followed the boundary changes, indicating the flood levels in Gotts are directly controlled by the downstream tidal level during this event.
- Fleet Fen showed a significantly lower sensitivity, indicating that flood levels within the catchment are not as sensitive to the tidal boundary, which is thought to be due to the higher riverbanks along the catchment boundary compared to the Gotts.

8 Model Stability and Limitations

One of the main indicators of model stability with the FMP solver is the model's convergence based on the user-defined convergence tolerances for timestepping. The convergence tolerances for timestepping runs in Flood Modeller are defined by the variables htol (stage tolerance) and qtol (flow tolerance), which were specified in the 1D Simulation file (ief) via the 1D simulation interface. A summary of the convergence tolerances is provided Table 12.

Table 12 – Summary and description of convergence tolerances

Convergence Tolerances	Description	User-Defined Values
htol (stage tolerance)	A combination of absolute and relative error thresholds for the water surface elevation (head or stage).	0.01
	htol is compared with the difference in water levels between iterations (dh).	
qtol (flow tolerance)	A combination of absolute and relative error thresholds for the flow rates (discharge).	0.01
	qtol is compared with the flow difference between the current and previous iteration at a given node.	

Figure 15 shows the diagnostic plot output from the model run for the joint 0.1% AEP + climate change design events for the pump failure analysis.

The Iteration/Timestep plot indicates that the model is stable, as the number of iterations per timestep is very low and consistent except for an initial jump at the beginning due to the baseflows from the initial conditions. The very low and consistent iterations per timestep are indicative that the model is solving the equations efficiently at each timestep without needing many recalculations.

The Model Convergence plot confirms that the model converged successfully. After the initial spike at the beginning, the flow and level lines significantly fell and remained below the tolerance line for the user-defined values in Table 12. This means the model's solution for both flow and water levels stabilised and met the predefined convergence criteria.

The Total Flows plot presents the total inflow and outflow over time. Based on the hydrographs, the model is balanced, indicating a complete and accurate mass balance within the model.

Based on the diagnostic plot output, the model ran to completion with robust performance and numerical stability.

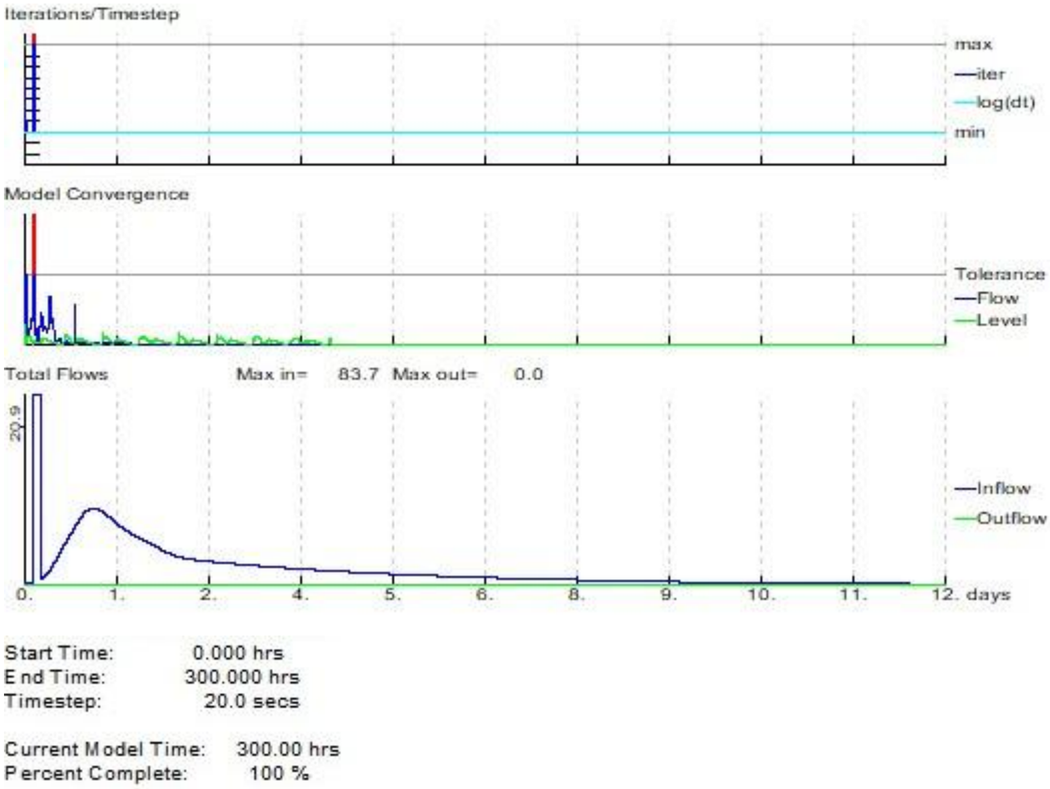


Figure 15 – Diagnostic plot for the design joint 0.1% AEP + climate change design event from the pump failure analysis

9 Conclusions

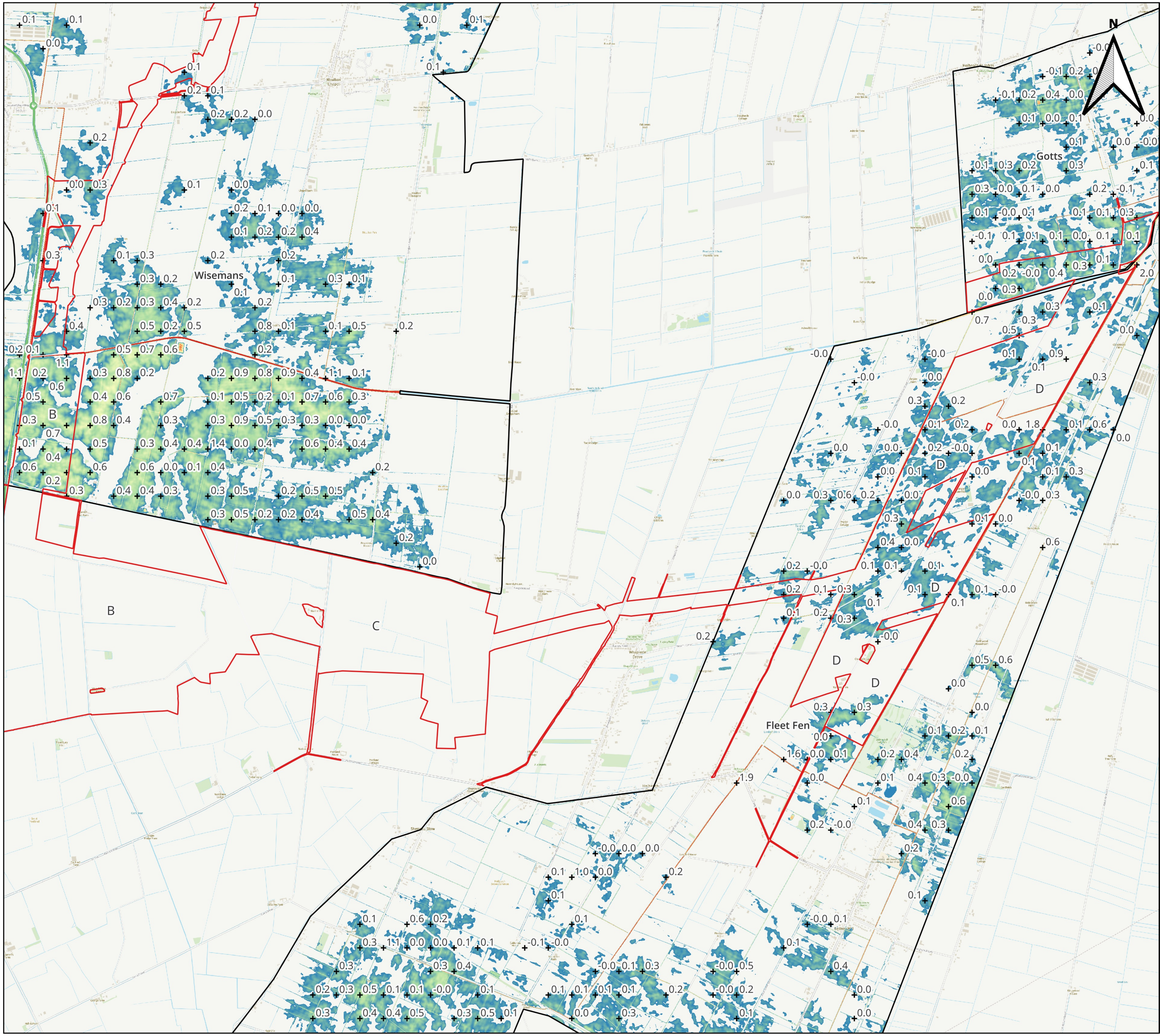
A new one-dimensional (1D) hydraulic model of the South Holland Main Drain (SHMD), including two of the South Holland Internal Drainage Board (SHIDB) pumped catchments (Fleet Fen – Catchment E and Gotts – Catchment S), was built to assess the worst-case flood risk scenario and to define the flood levels for the proposed Meridian Solar farm.

The fluvial and tidal 0.1% AEP plus climate change event was selected as the design event in accordance with the South East Lincolnshire Strategic Flood Risk Assessment and in agreement with the Environment Agency and the modelled flood levels during this event have been established.

Given the type of development, the modelling approach is relatively simple, and a number of conservative assumptions have been made. Although it is deemed that these assumptions are appropriate for this development, they may not be appropriate for other, higher risk development.

Appendix 1 – Peak Flow Assessment

Appendix 2 – Pumping Stations



Project :

Meridian Solar Farm
South Holland Main Drain

Client :

AECOM

Legend :

- Site Boundary
- SHIDB Catchments
- + Sampled max flood depths (m)

0.1% AEP + CC Modelled Flood Depths (m)

2

0

Scale :

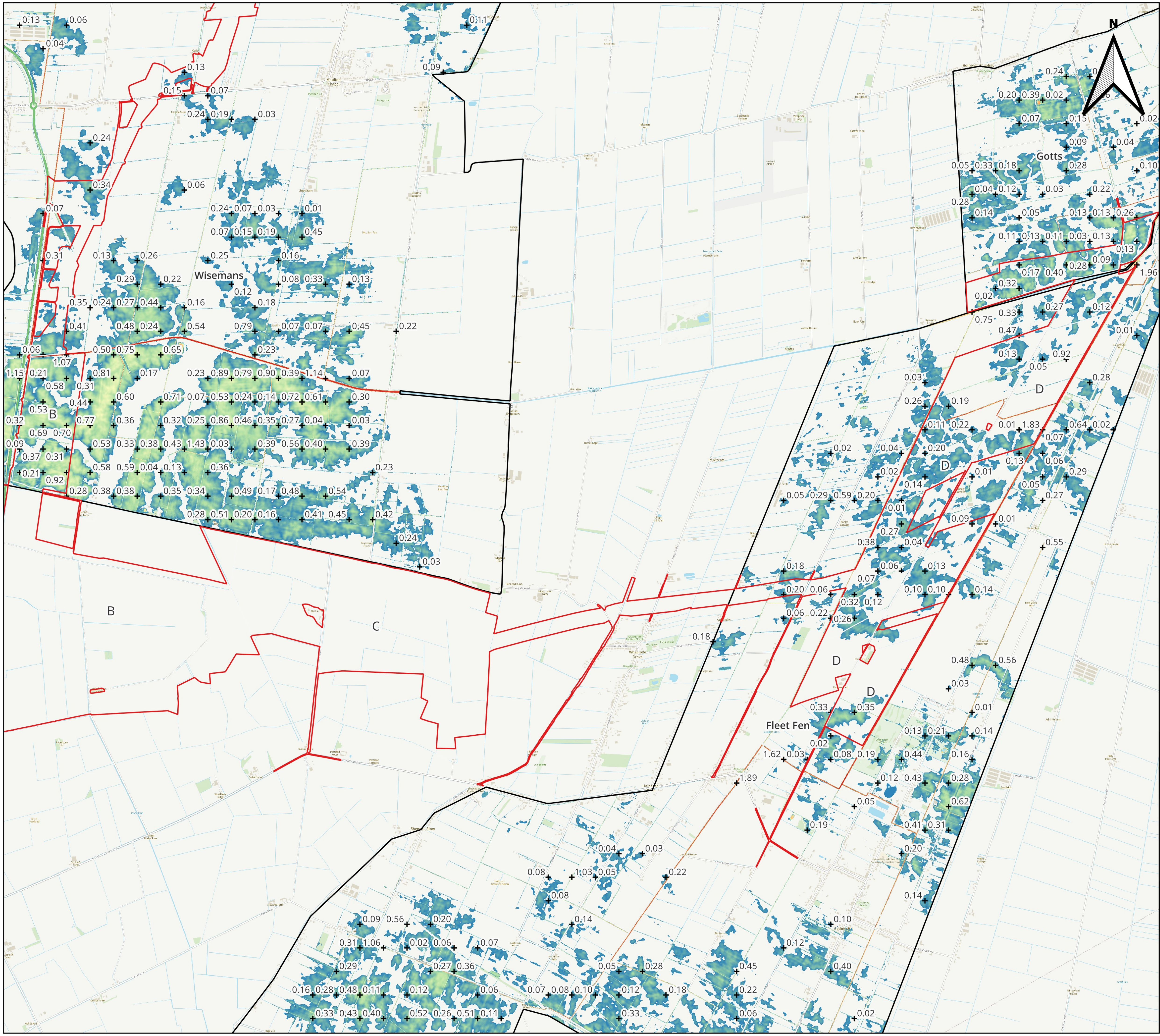
0 250 500 750 1,000 m

Title :

0.1% AEP Flood Event + CC Modelled Flood Depths (m) for Pump Failure Scenario

Drawing :	Rev :
WHS10217-T01-0002	3

WHS



Project :

Meridian Solar Farm
South Holland Main Drain

Client :

AECOM

Legend :

Site Boundary

SHIDB Catchments

+ Sampled max flood depths (m)

0.1% AEP + CC Modelled Flood Depths (m)

2

0

Scale :

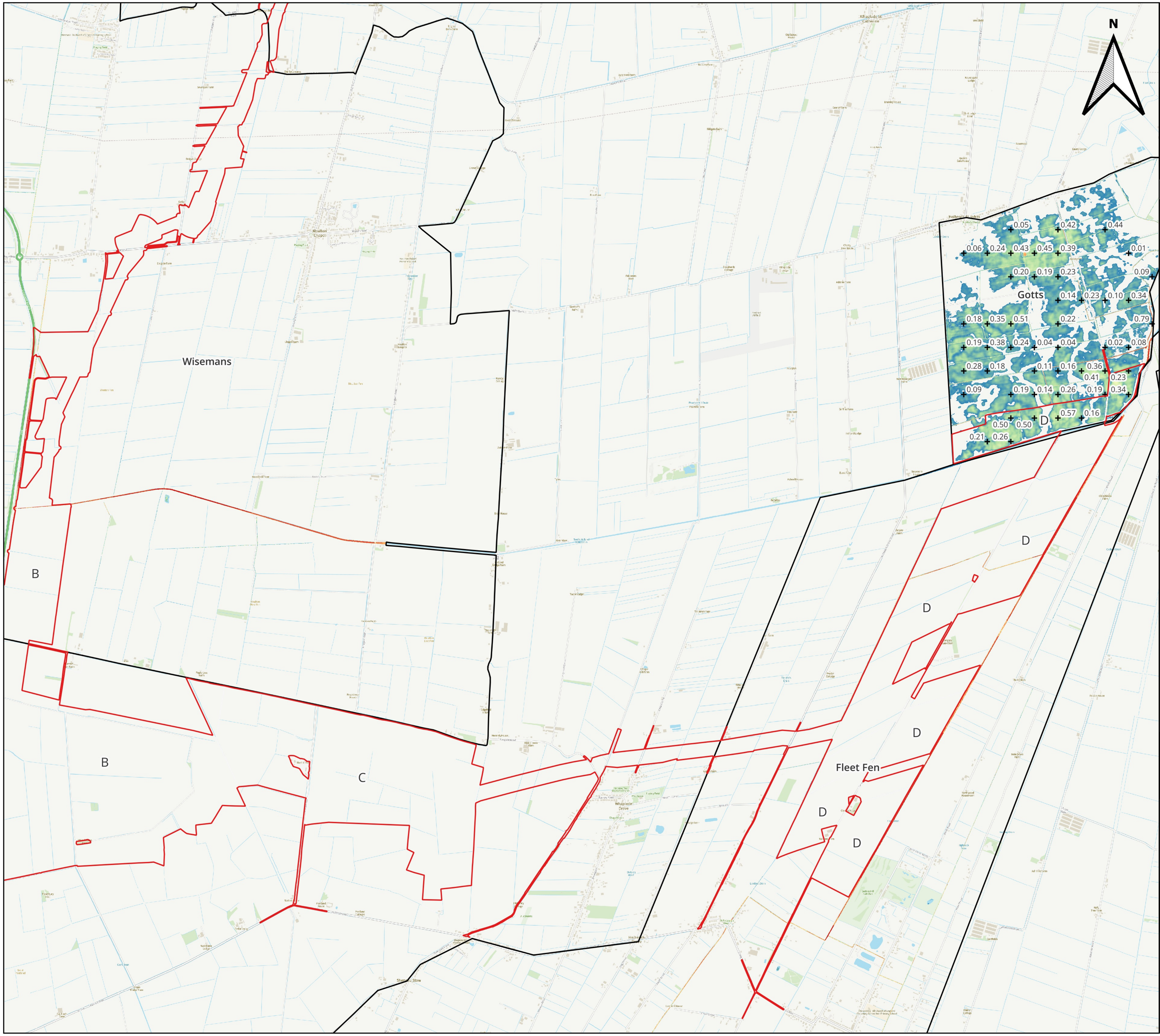
0 500 1,000 1,500 2,000 m

Title :

0.1% AEP Flood Event + CC Modelled Flood Depths (m) for Pump Failure Scenario

Drawing :	Rev :
WHS10217-T01-0002	4

WHS



Project :

Meridian Solar Farm
South Holland Main Drain

Client :

AECOM

Legend :

Site Boundary

SHIDB Catchments

3.33% AEP Modelled Flood Depths (m)

2

0

Scale :

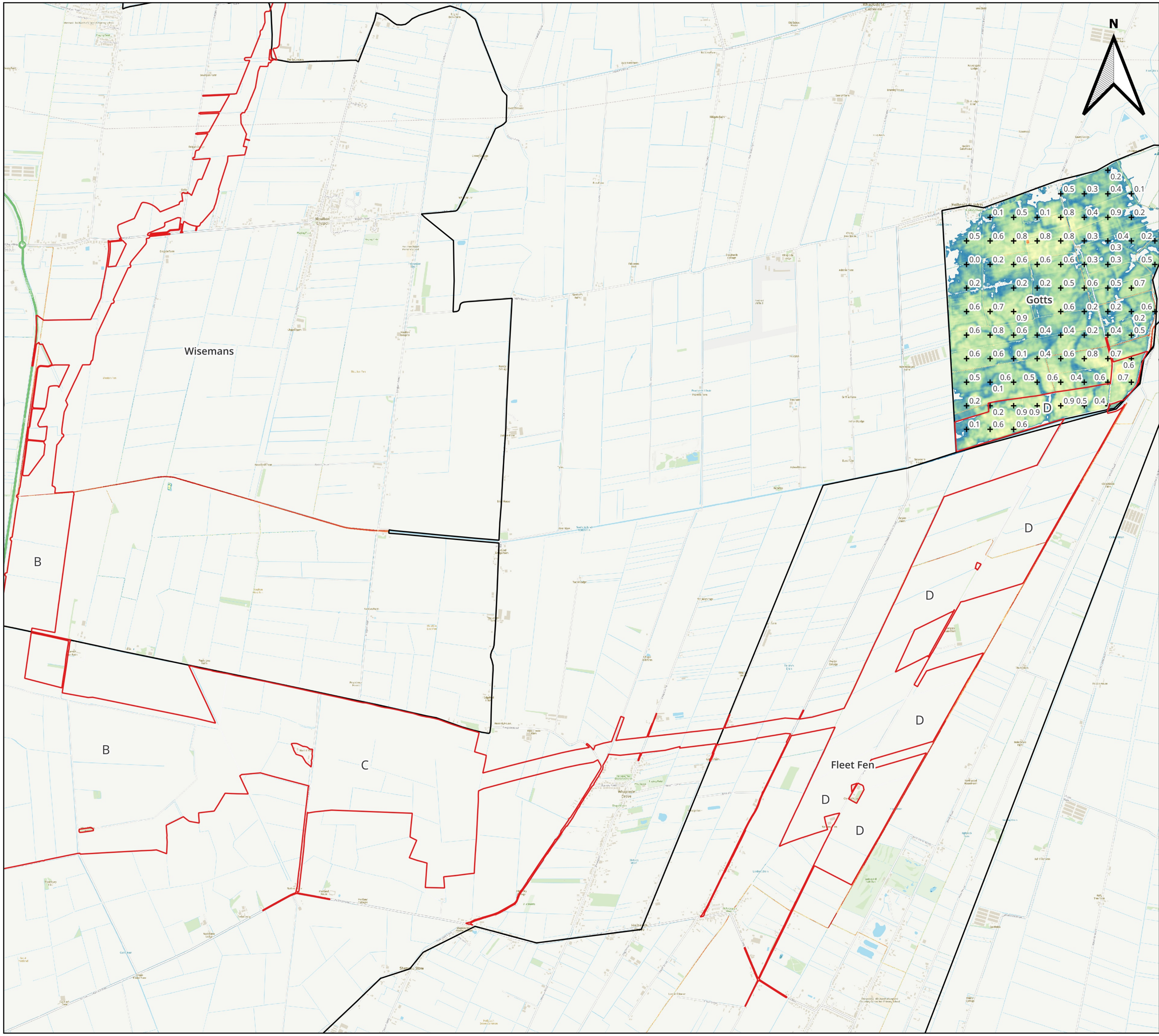
0 500 1,000 1,500 2,000 m

Title :

3.33% AEP Modelled Flood Depths (m)
Pumps Operational

Drawing :	Rev :
WHS10217-T01-0003	1

WHS



Project :

Meridian Solar Farm
South Holland Main Drain

Client :

AECOM

Legend :

Site Boundary
SHIDB Catchments
+ Sampled max flood depths (m)

1.0% AEP Fluvial / 0.5% AEP Tidal Event - Year 2125
Modelled Flood Depths (m)

2

0

Scale :

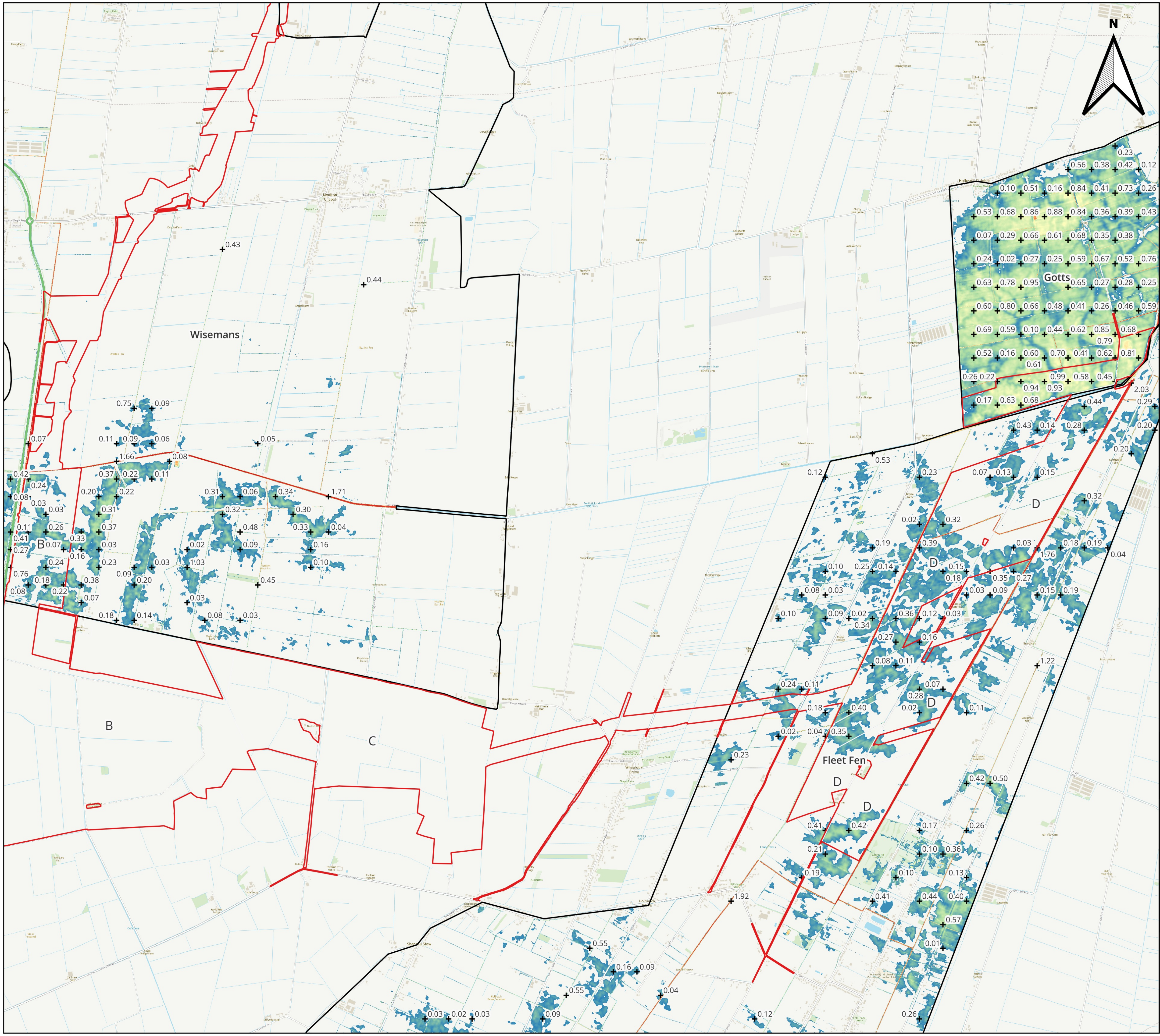
0 500 1,000 1,500 2,000 m

Title :

1.0% AEP Fluvial / 0.5% AEP Tidal Event - Year 2125
Modelled Flood Depths (m)
Pumps Operational

Drawing :	Rev :
WHS10217-T01-0004	1

WHS



Project :

Meridian Solar Farm
South Holland Main Drain

Client :

AECOM

Legend :

- Site Boundary
- SHIDB Catchments
- + Sampled max flood depths (m)

0.1% AEP Fluvial / 0.1% AEP Tidal Event - Year 2125
Modelled Flood Depths (m)

2

0

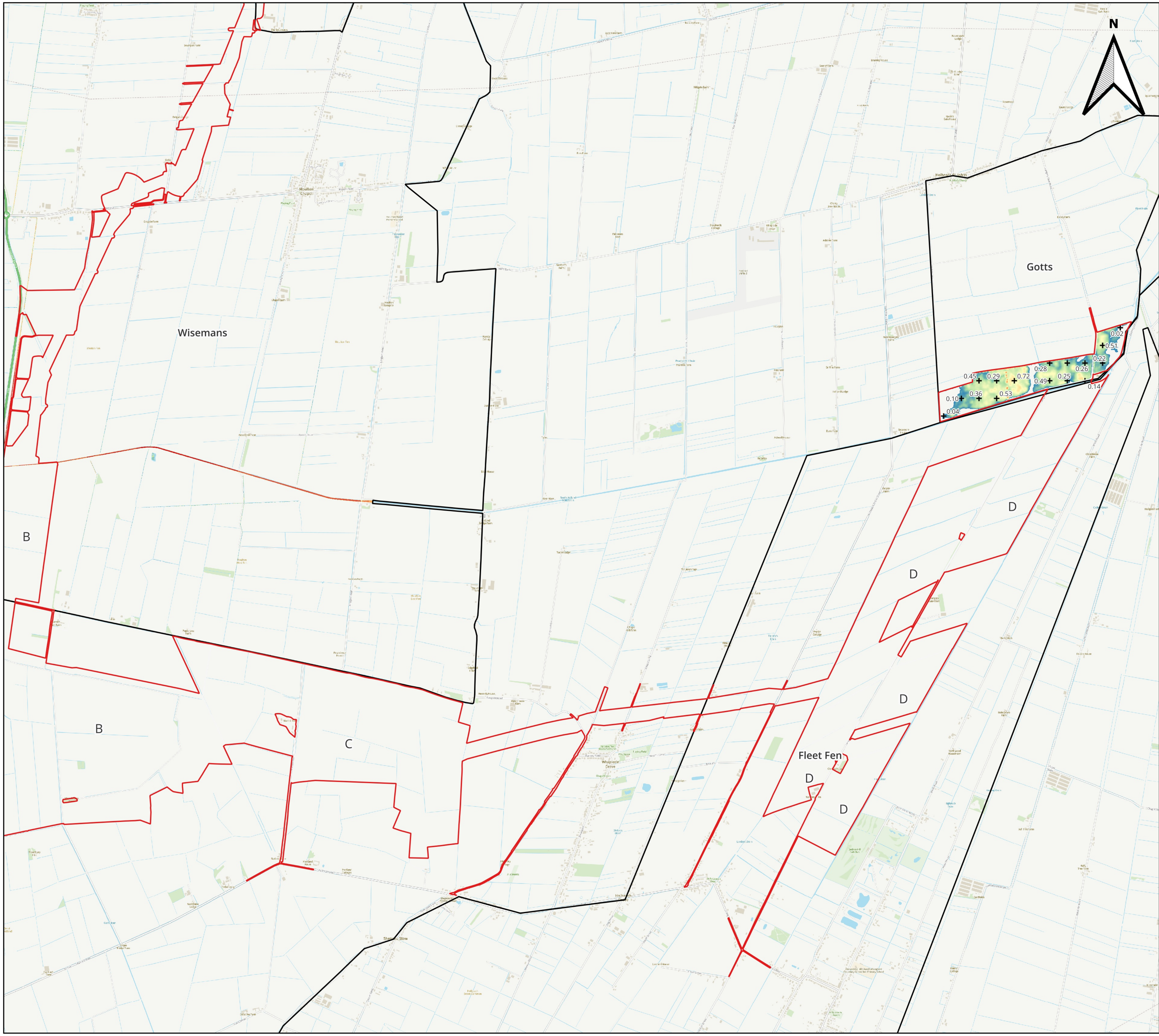
Scale :

0 500 1,000 1,500 2,000 m

Title : 0.1% AEP Fluvial / 0.1% AEP Tidal Event - Year 2125 Modelled Flood Depths (m) Pumps Operational

Drawing :	Rev :
WHS10217-T01-0005	1

WHS



Project :

Meridian Solar Farm
South Holland Main Drain

Client :

AECOM

Legend :

Site Boundary

SHIDB Catchments

+ Sampled max flood depths (m)

3.33% AEP Modelled Flood Depths (m)

1.4

0

Scale :

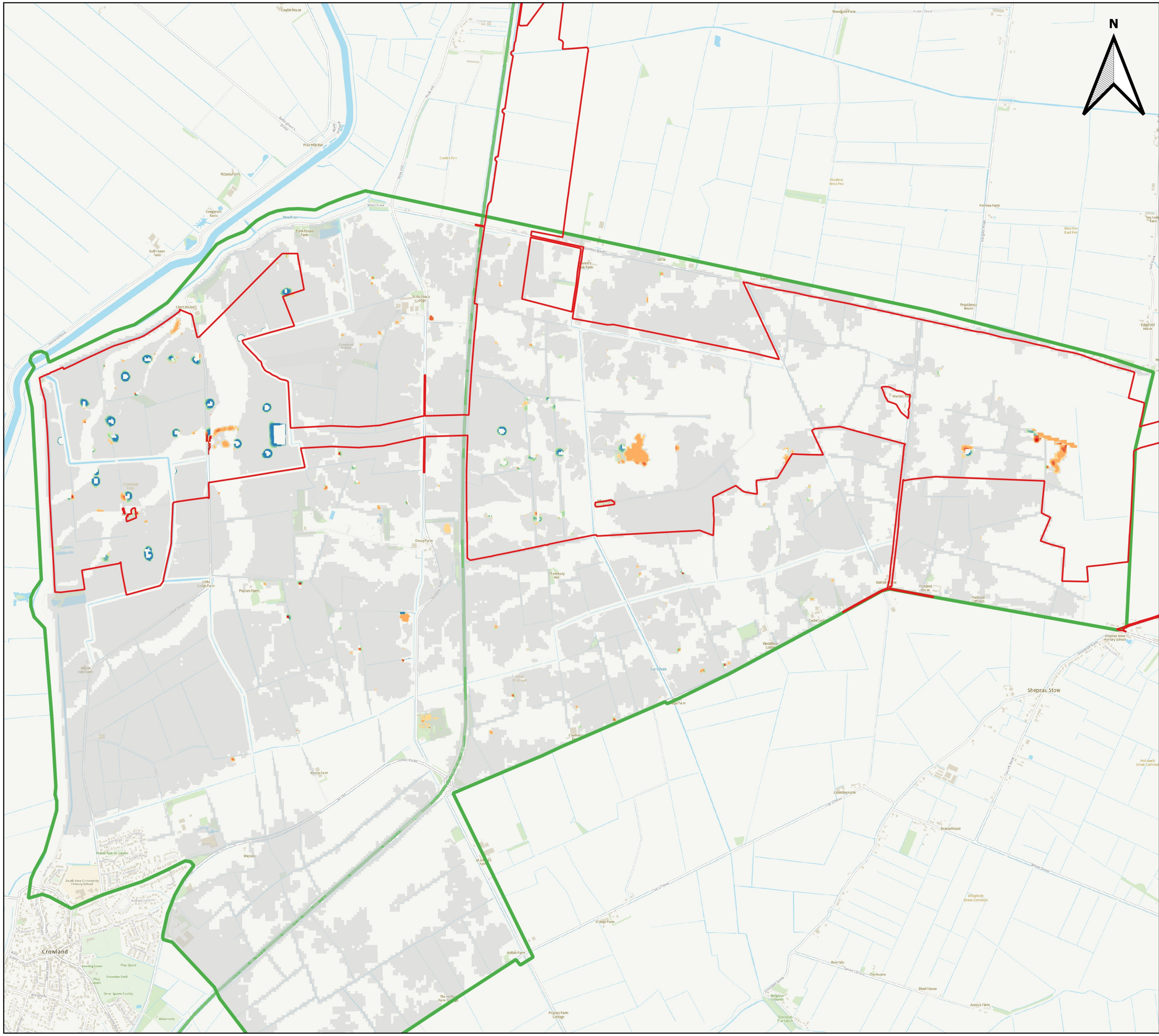
0 500 1,000 1,500 2,000 m

Title :

3.33% AEP Modelled Flood Depths (m)
Pumps Operational Scenario - with
Topo Survey

Drawing :	Rev :
WHS10217-T01-0006	1

WHS



Project :

Meridian Solar Farm

Client :

AECOM

Legend :

Site Boundary

Postland Catchment

Depth Change (m)

<= -0.10

-0.10 - -0.05

-0.05 - -0.02

-0.02 - -0.01

-0.01 - 0.01

0.01 - 0.02

0.02 - 0.05

0.05 - 0.10

>0.10

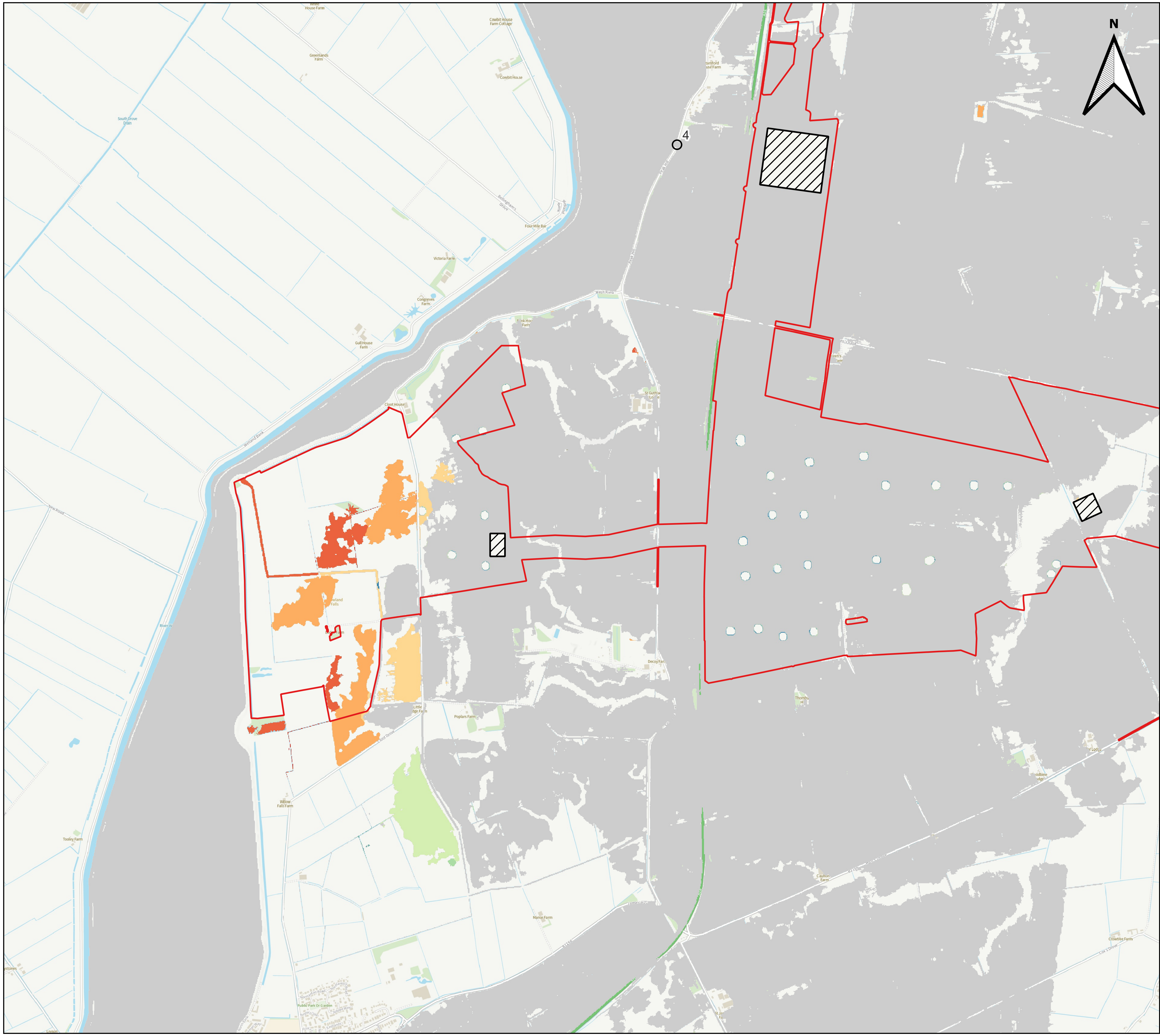
Scale :

0 500 1,000 m

Title : Post-development vs. Pump Failure Scenario 0.1% AEP Flood Event + Climate Change Depth Change

Drawing :	Rev :
WHS10217-T01-0007	1

WHS



Project :

Meridian Solar Farm

Client :

AECOM

Legend :

- Site Boundary
- Breach Location 4

Depth Change (m)

- <= -0.5
- -0.5 - -0.2
- -0.2 - -0.1
- -0.1 - -0.05
- -0.05 - 0.05
- 0.05 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- > 0.5

Scale :

0 500 1,000 m

Title :

1.0% AEP Flood Event + CC
Breach Location 4 Depth Change
Analysis

Drawing :	Rev :
WHS10217-T01-0008	1

WHS