Paul Basham Associates

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Great Wilsey Park Hydraulic Modelling Report

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For and on behalf of Wallingford HydroSolutions Ltd.

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Contents

1 Introduction

1.1 Background

Wallingford HydroSolutions Ltd has been commissioned by Paul Basham Associates to undertake bespoke hydraulic modelling to assess flood risks at a proposed residential development site in Wilsey, Haverhill (NGR: 569086, 245529).

Detailed pluvial modelling is required to confirm and quantify the flood risk to the site indicated in the Environment Agency's Risk of Flooding from Surface Water (RoFSW) mapping. The model will also be used to assess if there are any increased third-party impacts due to the development.

1.2 Methodology

A 2D pluvial hydraulic model of the catchment containing the proposed development has been constructed to assess surface water flood risk across the site and third-party impacts associated with the development.

The pluvial model has been produced using TuFLOW hydraulic modelling software with rainfall inputs derived using ReFH2. Ground levels for the model have been informed by LiDAR data published in 2022. The 1.0% AEP, 1.0% AEP + CC and 0.1% AEP events were assessed for several storm durations in line with the EA national scale RoFSW flood map.

1.3 Data Sources

The data sources used to inform the hydraulic model are as follows:

- LiDAR data from the National LiDAR Programme[1].
- FEH Web Service catchment data[2].
- Client supplied topographical data[3].
- Site xml files – final and existing contours, and site layout.

1.4 Assumptions

- Topographic data accurately represents major flow pathways.
- Where appropriate, sewer losses have been modelled as a continuing loss from the system in urban areas based on the surface material type.
- Sensitivity analysis is appropriate to test model robustness and uncertainty.
- A blockage assessment is not required.

[1] LiDAR Compositive Digital Terrain Model (DTM) – 1m. Environment Agency. November 2024. Available at: https://environment.data.gov.uk/survey
[2] UK Centre for Ecology and Hydrology, FEH Webservice. Available at: https://fehweb.ceh.ac.uk/Map
[3] Topographical Survey (Land to the South of Great Wilsey Farm). Interlock Surveys Ltd. February 2025. Dwg. No. 141025 3D.

2 Site Description

2.1 Site Location

The proposed development is located on the outskirts of Wilsey approximately 590 m west of Calford Green and 440 m north of the Stour Brook at the closest point. An unnamed watercourse also flows along the eastern boundary of the development into the Stour Brook approximately 1 km south of the site.

Figure 1 – Site Location

3 Hydrological Assessment

As this is a pluvial model, rainfall hyetographs are required for input into the 2D domain. The first stage in estimating the rainfall hyetograph was to derive a representative pluvial catchment area. The EA's Most Probable Overland Flow Pathway dataset[4] has been used to derive the catchment area. LiDAR data, existing surface water flood maps, and the FEH Web Service have subsequently been used to check the size of the pluvial catchment area. As the catchment derived is small (< 5 km^2) and has been manually defined, point data based on the catchment's centroid have been used to extract FEH point data from FEH Web Service to define the pluvial catchment. An overview of the catchment and the point location is shown in Figure 2.

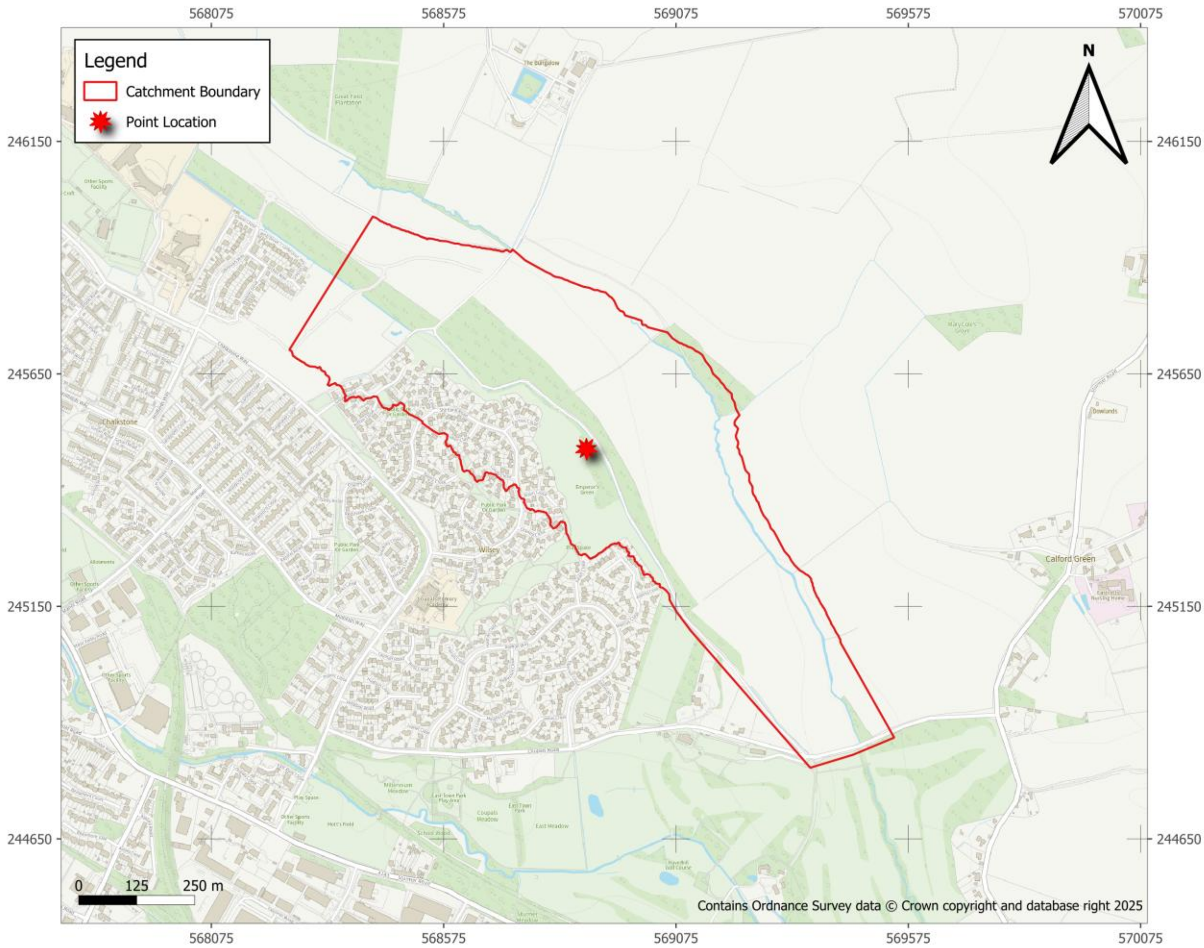

Figure 2 – Diagram illustrating the pluvial catchment and location of point data

Two separate hyetographs representing net rural rainfall and net urban rainfall were derived for each storm duration for each of the return periods being assessed. In terms of duration, hyetographs were derived for the following storms:

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[4] Overland Flow Pathways. Environment Agency. 2024.

- Duration = 1hr, Timestep = 4min
- Duration = 3hr, Timestep = 12min
- Duration = 6hr, Timestep = 24min
- Duration = 9hr, Timestep = 36min
- Duration = 12hr, Timestep = 48min
- Duration = 18hr, Timestep = 72min

The method for estimating the rainfall hyetographs is detailed in the full hydrology report attached as Appendix 1. The peak net rainfall across a timestep for the 0.1% AEP event for each duration applying the summer seasonality is also shown in Table 1. This net rainfall represents the amount of rainfall that contributes to surface runoff during the 0.1% AEP event for the specified duration.

Table 1 – Peak net rainfall

	1 Hour Peak Net Rainfall (mm)	3 Hour Peak Net Rainfall (mm)	6 Hour Peak Net Rainfall (mm)	9 Hour Peak Net Rainfall (mm)	12 Hour Peak Net Rainfall (mm)	18 Hour Peak Net Rainfall (mm)
	1000yr	1000yr	1000yr	1000yr	1000yr	1000yr
Urban Summer	8.06	20.71	25.45	28.36	30.30	32.65
Rural Summer	5.98	8.96	11.82	13.66	15.00	16.61

4 Hydraulic Model Build

4.1 2D Domain

Details on the 2D model build are presented in the section. The 2D domain was modelling using TuFLOW's latest 2023 release (2023-03-AE), which was run on a Graphic Processing Unit (GPU) using the Heavily Parallelised Compute (HPC) solver.

4.1.1 Model Extent

As detailed in section Figure 3, the 2D domain was defined using the EA' s Probable Overland Flow Pathways and Detailed Watershed dataset[4]. The 2D model extent was extracted from the detailed watershed that covered all the probable overland flow pathways entering the site. The selected section of the detailed watershed was then buffered by 50 m creating a final 2D model extent covering an area of 0.74 km^2. This ensured that the model was sufficiently large to capture all overland flow routes potentially impacting the site.

The model extent described above is shown in Figure 3. The extent of urbanisation was derived using the material layers obtained from Ordnance Survey (OS) Vector Tile API layer[5]. The OS layer IDs and symbol codes were used to classify the rural and urban areas throughout the model extent. The active area was deemed to be predominantly rural, with a localised built-up area (a section of the Wilsey area) along the western boundary. This was confirmed as a suitable representation of the land uses within the active area by conducting a visual comparison of the OS Vector Tile API layer against satellite imagery and UKCEH Land Cover Map[6].

[5] OS Vector Tile API. Ordnance Survey Ltd 2025. Available at: https://www.ordnancesurvey.co.uk/products/os-vector-tile-api

[6] Land Cover Map 2021 10m Web Map Service. UKCEH. September 2022.

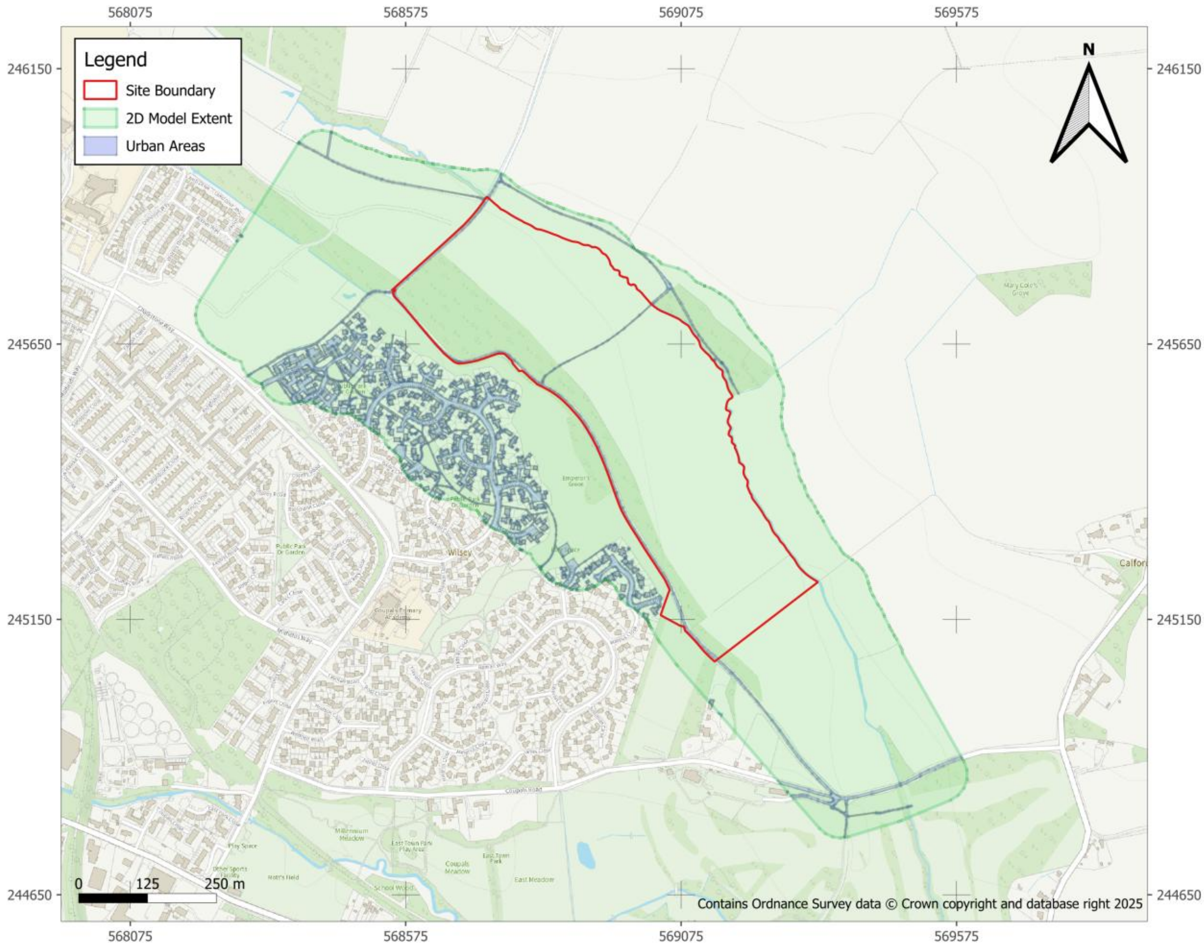

Figure 3 – 2D Model Extent

4.1.2 Grid Size and Orientation

A 1 m grid size was used to define resolution of the model which is considered sufficient to represent major flow pathways between buildings based on the recommended method statements for the Risk of Flooding from Surface Water (RoFSW) mapping[7]. Sub-grid sampling was also used to improve model accuracy as it allows TuFLOW to sample multiple points within a single cell using the LiDAR/topographic input producing more detailed cross-section of flow paths.

The orientation of the grid is defined by the location line, which is digitised so that the orientation of the computational area of the model is predominantly pointing east so TuFLOW will report the component of flow perpendicular to the line, i.e., flows from north to south within the catchment.

4.1.3 LiDAR Data

The topography of the active area is defined using 1 m resolution LiDAR Composite DTM data[1] flown in 2022. This is considered adequate to represent the key flow routes across the modelled area.

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[7] Risk of flooding from surface water – understanding and using the map. Environment Agency. 2025.

4.1.4 DTM Modifications

Existing buildings within the active area have been represented by an increase in Manning's n value and a direct modification to the DTM using a Z shape layer. The Z shape layer instructed TuFLOW to increase the DTM at the buildings' location (see Figure 4) by 300 mm to represent the floor slabs through which flows should not pass (unless depths exceed 300 mm).

Existing survey data was also incorporated into the model to more accurately represent sections of the ditches/watercourses flowing through the active area, see Figure 4. The topography of the ditches was extracted from land xml files provided by the client and saved as raster files, which were then read into the model using 'Read Grid Zpts' commands. This instructed TuFLOW to update the Z points (elevation values) of the 2D computational grid with the assigned Z points from the ditch surface rasters.

Figure 4 – Buildings and ditch extents used for DTM modification.

4.1.5 Boundary Conditions

Rainfall versus time (RF) flow boundaries

Rainfall versus time (RF) flow boundaries were used to apply the hyetographs derived in section 3. The hyetographs are applied as rainfall depth in mm vs timestep, i.e., each rainfall depth is the amount of rain that fell in millimetres between the previous time step and the current one. The RF flow boundaries were applied using a 2d_rf layer containing rainfall polygons used to apply rainfall spatially based on rural and urban extents. The RF flow boundaries applied to the model domains is shown in Figure 5 below.

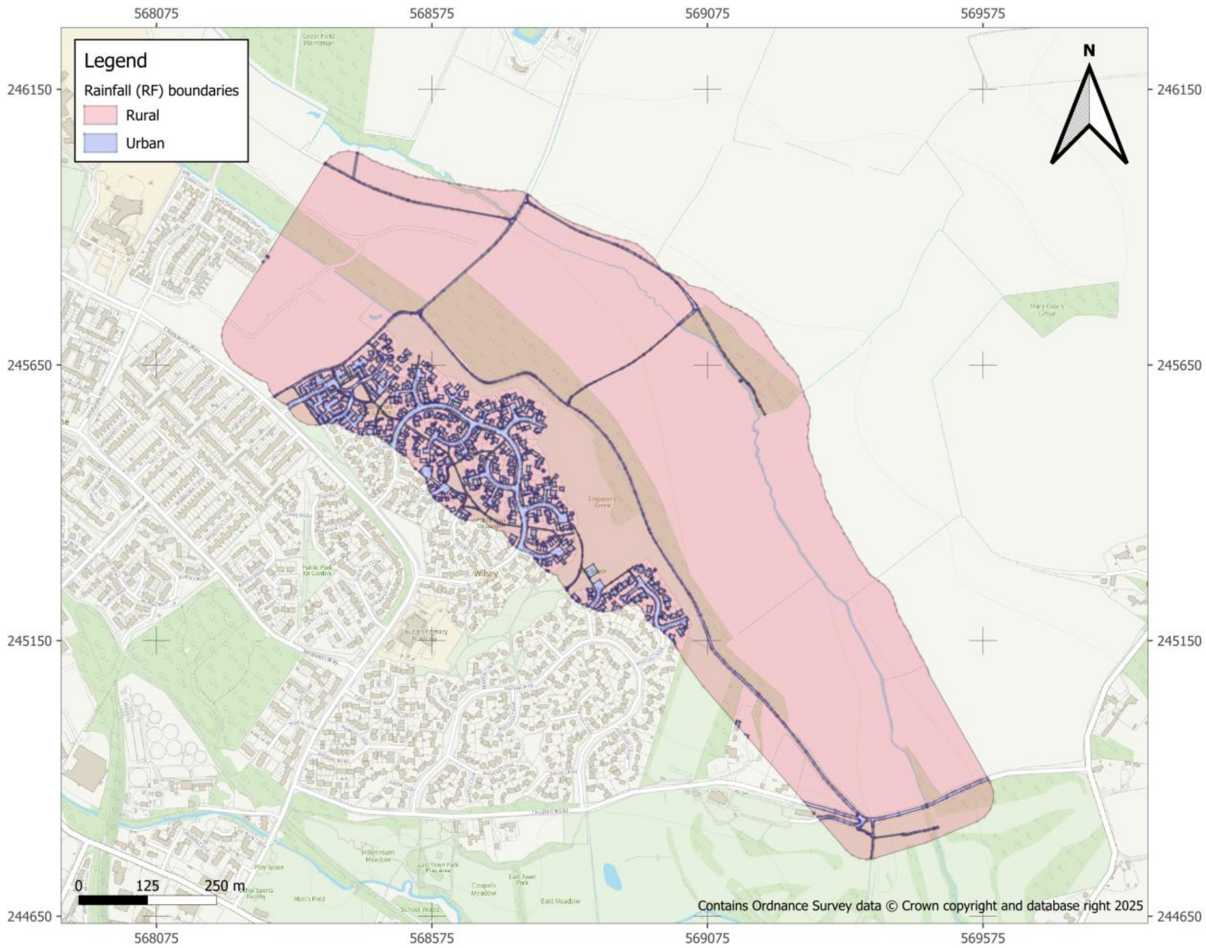


Figure 5 – Rainfall (RF) boundaries

Stage-discharge (HQ) boundaries

HQ boundaries were used to allow water to escape the model when it reached the downstream boundaries of the active area to prevent glass walling. The gradient of the terrain at these downstream locations was determined and used to set the 'b' value in the 2D boundary. All areas where the terrain gradient was calculated and applied at a boundary location is shown in Table 2 and Figure 6.

Table 2 – Gradient within HQ boundaries

HQ Boundary	Gradient, b (m/m)
A	0.023
B	0.051
C	0.055
D	0.070
E	0.017

Figure 6 – Stage-discharge boundaries

4.1.6 Initial Conditions

The initial conditions within the model are initial water levels for the surface water areas identified from the OS Vector API Tile layer. The initial water levels were applied in still waterbodies using the levels extracted for from the LiDAR data.

4.1.7 Surface Roughness

The floodplain roughness has been defined using the Manning's n roughness coefficient. As detailed in section 4.1.1, a combination of OS and satellite mapping has been used to identify the different land uses within the model extent.

2D material files have been produced with an associated material code referenced within a TuFLOW materials file (.tmf) where they are assigned appropriate roughness coefficient. These are detailed Table 3 below.

Table 3 – Manning's Roughness Coefficient

Land Use	Material ID	Manning's Coefficient
Field/Natural Land (pasture high grasses)	1	0.035-0.4*
Water bodies	2	0.025
Buildings	3	0.500
Other manmade structures	6	
Railways	4	0.040
Roads	5	0.020
Driveways/Paths	11	
Woodland	7	0.10-0.40**
Gardens	8	0.40-0.50†
Bushes	10	0.070-0.40††
Lawns	12	0.035

* For field/natural land (pasture high grasses) (mat. ID 1) the n value was varied based on the flood depth, where for a flood depth < 0.1 m a n value of 0.4 was applied and for a depth > 0.4 m a n value of 0.035 was applied. For flood depths between 0.1 m and 0.4 m, the n value is interpolated between 0.4 and 0.035 according to bed resistance depth interpolation.

** For woodland (mat. ID 7) the n value was varied based on the flood depth, where for a flood depth < 0.1 m a n value of 0.4 was applied and for a depth > 0.4 m a n value of 0.1 was applied. For flood depths between 0.1 m and 0.4 m, the n value is interpolated between 0.4 and 0.1 according to bed resistance depth interpolation.

† For gardens (mat. ID 8) the n value was varied based on the flood depth, where for a flood depth < 0.1 m a n value of 0.4 was applied and for a depth > 0.4 m a n value of 0.5 was applied. For flood depths between 0.1 m and 0.4 m, the n value is interpolated between 0.4 and 0.5 according to bed resistance depth interpolation.

†† For bushes (mat. ID 10) the n value was varied based on the flood depth, where for a flood depth < 0.1 m a n value of 0.4 was applied and for a depth > 0.4 m a n value of 0.070 was applied. For flood depths between 0.1 m and 0.4 m, the n value is interpolated between 0.4 and 0.070 according to bed resistance depth interpolation.

For these land uses, varying the roughness according to the flood depth is required to represent the changes in material resistance across the floodplain as flood depth increases. This is in accordance with the guidance from the SuDS manual on the impact of flow depth on hydraulic roughness[8]. This includes flooded vegetation and woodland areas becoming smoother, i.e. providing less resistance to flow, as water depth increases.

[8] The SuDS Manual (C753). CIRIA. 2015.

5 Design Runs

5.1 Summary of Design Runs

The hydraulic model has been run for each of the following Annual Exceedance Probability (AEP) events to determine the critical duration:

- 1.0% AEP 1 hr, 3 hr, 6 hr, 9 hr and 12 hr (Summer)
- 1.0% AEP + climate change allowance 1 hr, 3 hr, 6 hr, 9 hr and 12 hr (Summer)
- 0.1% AEP 1 hr, 3 hr, 6 hr, 9 hr and 12 hr (Summer)

The climate change allowance has been applied based on the EA's guidance[9] for using peak rainfall intensity allowances to assess surface water flood risk. A peak rainfall climate change allowance of 25% was applied for the Combined Essex Management Catchment that the site falls within.

5.2 Critical Storm Event

The critical duration, i.e. the event that results in the greatest flood depth, was identified for the design event (1.0% AEP + CC) by comparing the rainfall duration runs against each other to determine which resulted in the greatest flood risk. The critical duration analysis showed a variety of durations as maximum across the model extent. However, the 3 hr duration was deemed as the critical duration as it showed up most predominantly within the site boundary as surface water flood routes. These flood routes were deemed the most critical flood risk to the area.

5.3 Confidence in Baseline Model Results

The modelled flood extents for the critical 3 hr rainfall duration for the 1.0% AEP and 0.1% AEP events are shown in Figure 7 and Figure 8, respectively. To give confidence in the modelled results for the baseline scenario, a like for like comparison of the modelled flood extents with the EA RoFSW mapping can be seen in the figures below. To facilitate the like for like comparison, flood depths below 75 mm were filtered from the modelled flood results to replicate the process used for the EA RoFSW flood maps as detailed in the EA guidance[7].

When comparing the flood extents differences can be discerned between the model and RoFSW extents for both return periods, particularly along the eastern boundary of the site, where the RoFSW extents are larger than the modelled extents. The observed differences between the datasets are potentially due to incorporating survey data into the baseline model, which improved the representation of the ditches/watercourses, allowing for more in channel conveyance compared to the national scale modelling. However, it is evident that the flood extents and locations are similar, which gives confidence in the modelled results for the baseline model. Additionally, there is good agreement between the mappings for the overland flow route through the site.

[9] Flood Risk Assessments: Climate Change Allowances. Environment Agency. May 2022.

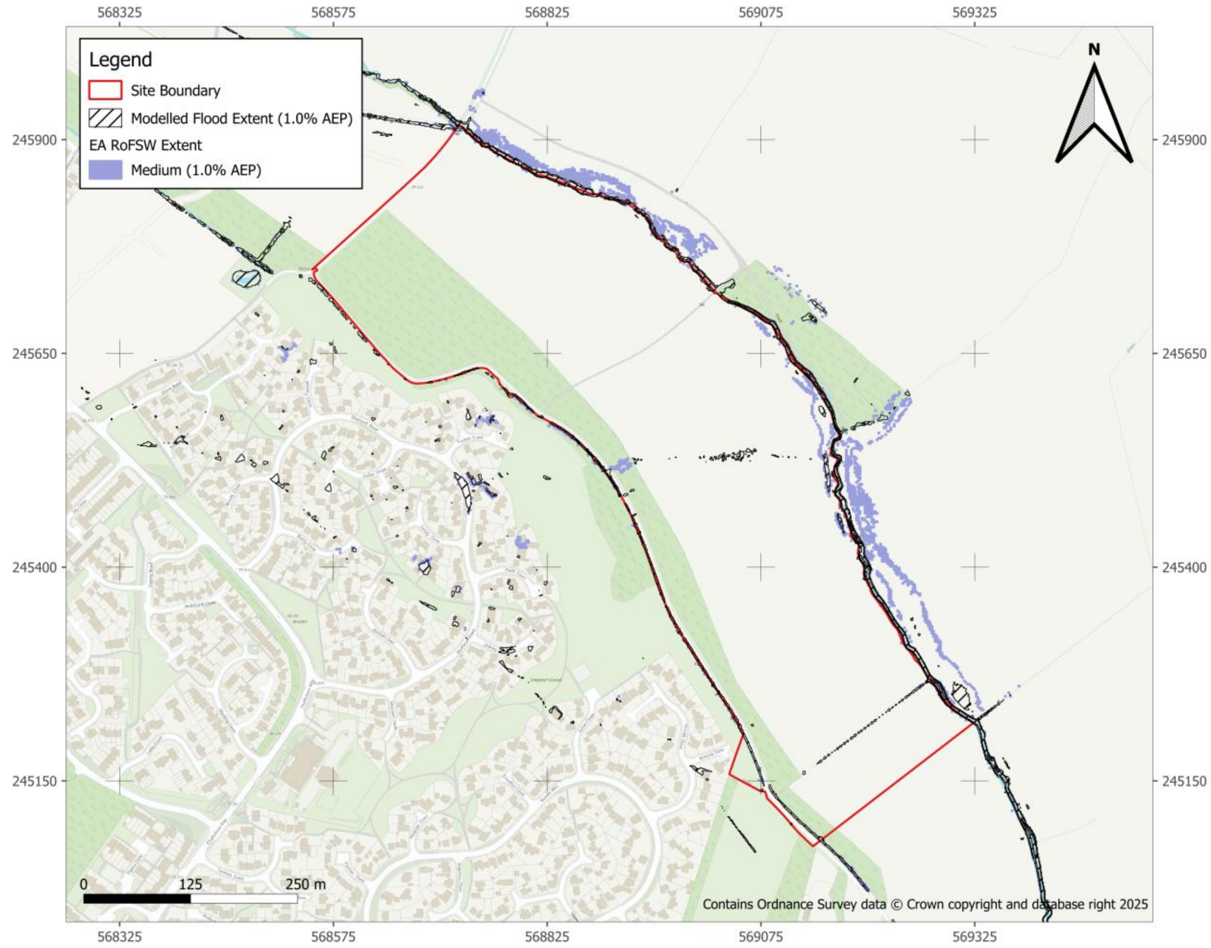


Figure 7 – Baseline 1.0% AEP 3-hr flood extent vs EA RoFSW extent for medium (between 1.0% and 3.3%) chance of flooding

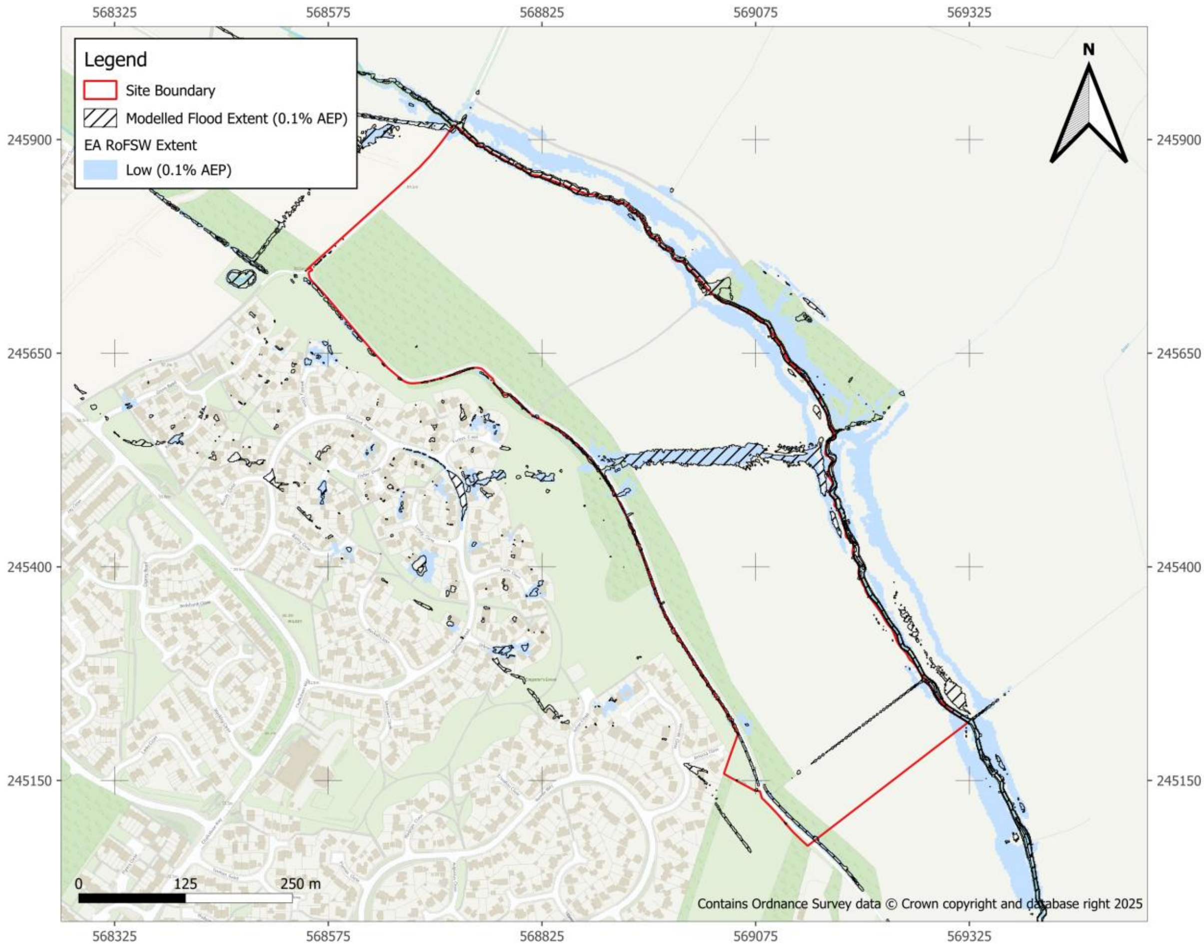


Figure 8 – Baseline 0.1% AEP 3-hr flood extent vs EA RoFSW extent for low (between 0.1% and 1.0%) chance of flooding

5.4 Baseline Results for Design Event

Figure 9 shows the baseline modelled maximum flood depths for the 1.0% AEP plus climate change design event. It indicates the centre of the site is at risk of flooding due to a surface water flow route resulting from the overtopping of the ditch along western boundary of the site. The maximum flood depth associated with the design event along this flow route is 0.2 m.

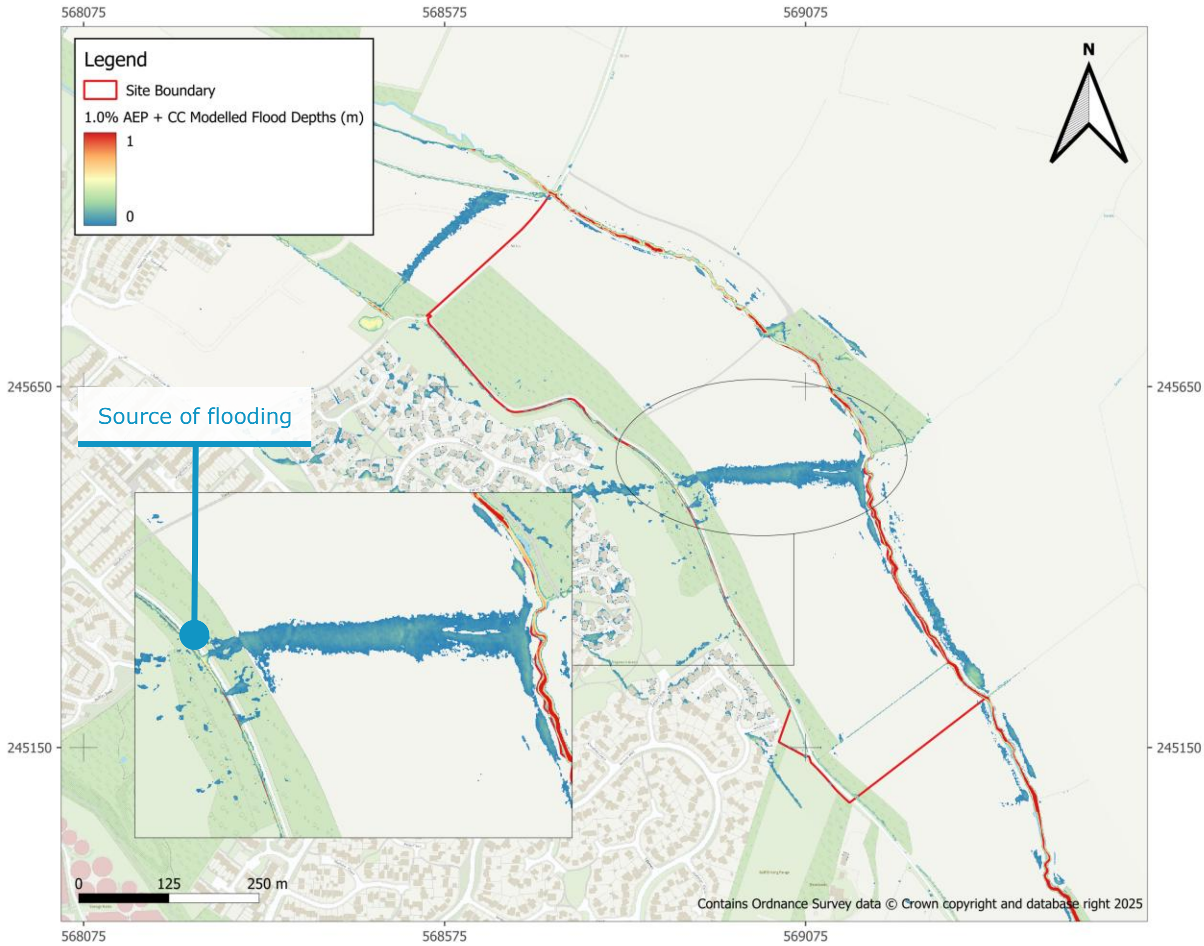


Figure 9 – Baseline modelled maximum flood depths for design event (1.0% AEP + CC event)

6 Post-Development Modelling

6.1 Post-Development Model Build

To prevent flows overtopping the ditch along the western boundary of the site from entering the development area, the client is proposing to reprofile the ground within the development area. The ground reprofiling aims to divert the surface water flow route, shown in the baseline mapping, to alternative locations outside the development area. Figure 10 shows the proposed contours for the reprofiling of the development area within the red line boundary.

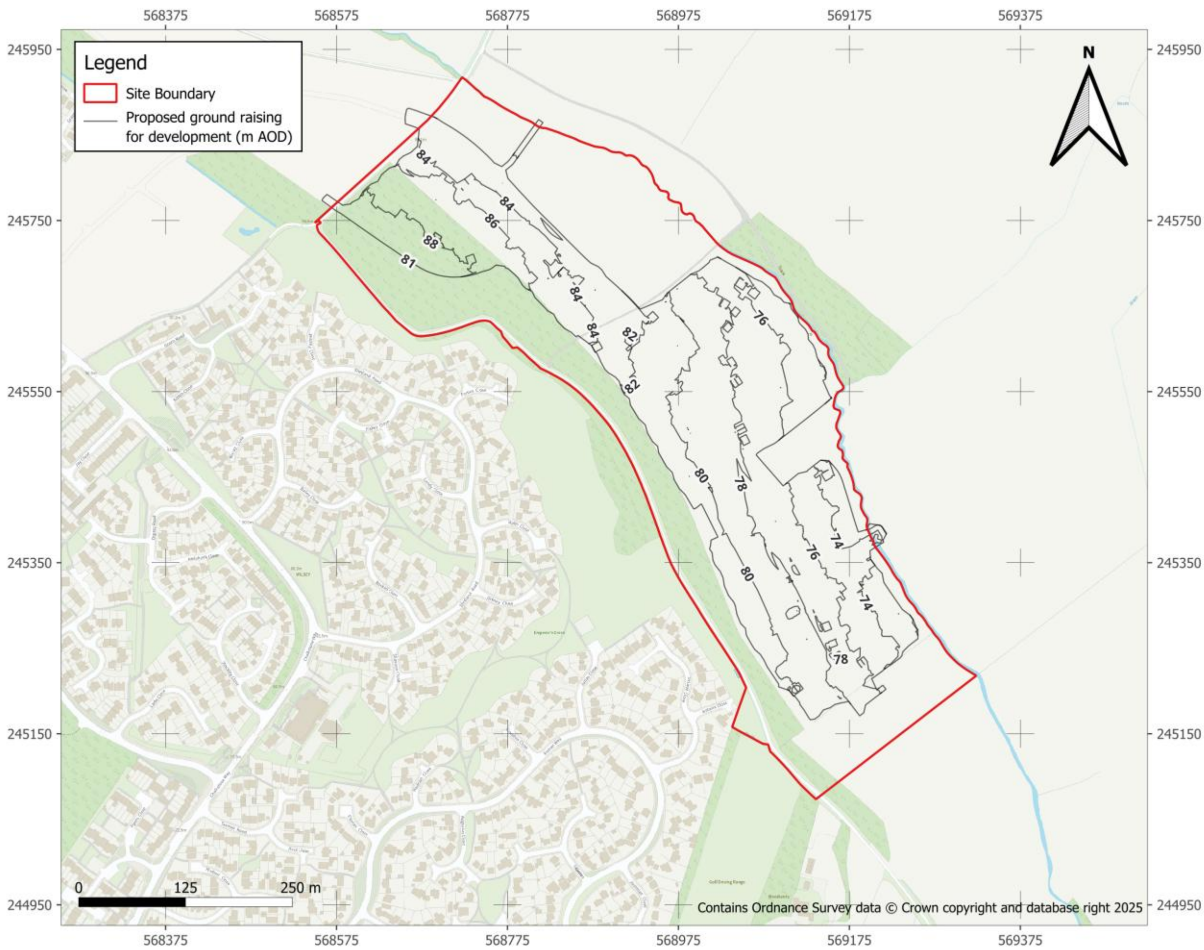


Figure 10 – Proposed contours for ground reprofiling

The proposed levels from the contours shown in Figure 10 were incorporated into the model using an S3D final surface XML file[10] to create the post-development scenario. The post-development model was used to confirm and quantify flood risk to the development and to determine if it resulted in increased third-party impacts.

[10] 1028.5003 - S3D Final Surface XML (25-03-19).

6.2 Post-development Results for Design Event

Figure 11 shows the post-development modelled results based on the incorporated ground raising. It indicates that the flooding from the ditch running along the western boundary will continue to overtop and enter the site despite the proposed reprofiling of the development area. After overtopping and entering the site, the flooding is diverted south along a roadway in the centre of the site. Therefore, a flood mitigation scheme is required to reduce flood risk to the development. This is outlined in section 7.

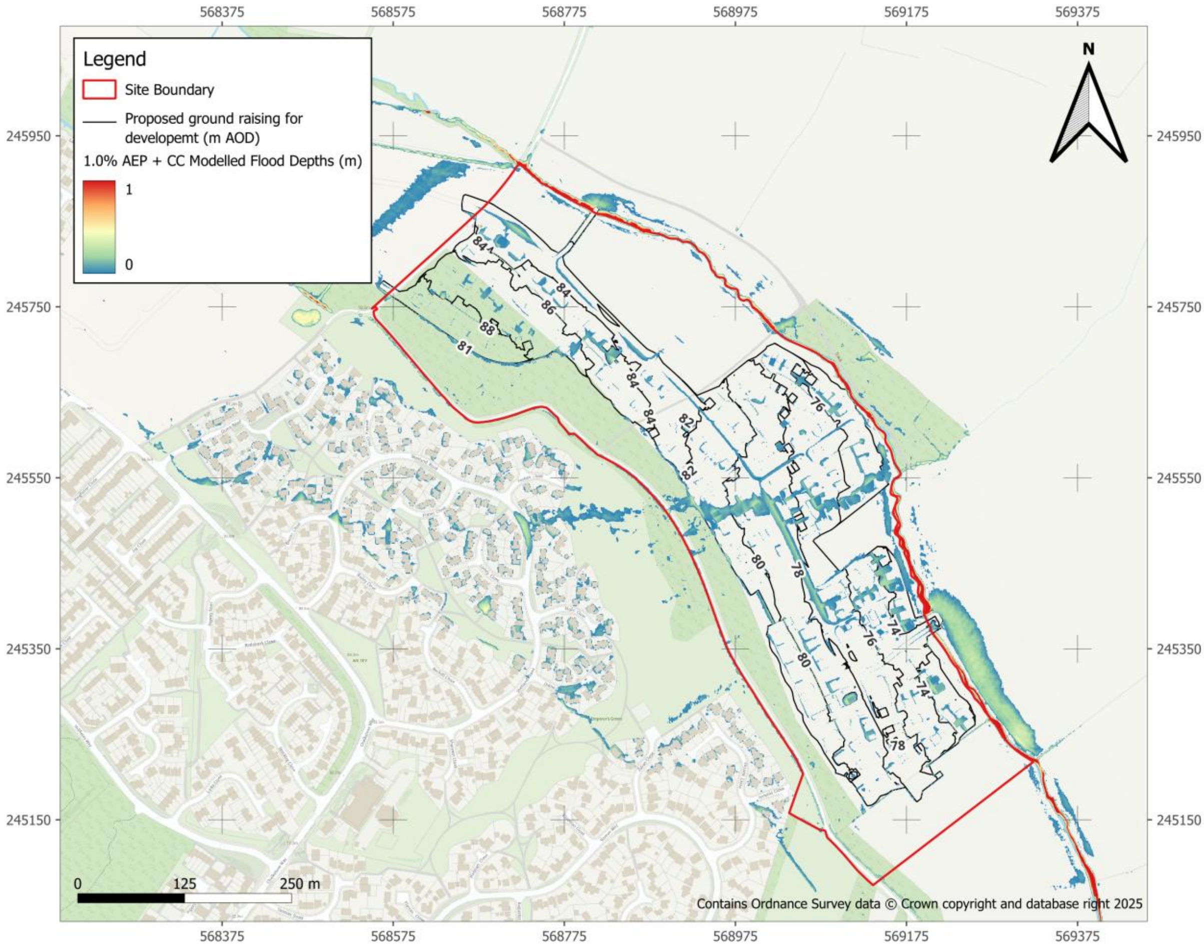


Figure 11 – Post-development modelled maximum flood depths for design event (1.0% AEP + CC event)

7 Mitigation Modelling

Mitigation options were developed to reduce the flood risk to the development, and their effectiveness was assessed through mitigation modelling. These options include a swale with a perforated pipe underdrain parallel to the ditch where overtopping is indicated. The swale will primarily act as a retention feature, intercepting flows that overtop the ditch to prevent the site from flooding. The perforated pipe underdrain will then convey flow to the south.

Flows retained by the swale will pass through its base into a perforated pipe, which will convey flows towards a watercourse along the eastern boundary of the site. The filter drain and perforated pipe will fall towards the south with a relatively flat gradient based on the adverse gradient at this location. At the southern end of the swale, the perforated pipe will transition to a culvert and continue towards the discharge point at the watercourse flowing along the eastern boundary of the site.

As part of the mitigation modelling, several diameter sizes were assessed for the perforated pipe and diversion culvert. A diameter of 450 mm was determined to be the most feasible option, providing sufficient capacity to convey flows to the discharge location while reducing flood risk to the site during the design event.

In addition to the swale and diversion culvert, an existing 300 mm pipe outfall from the existing ditch has been incorporated to discharge into the swale/filter drain. The exact location and depth of the pipe will need to be confirmed at the detailed design stage.

The options considered for the swale and filter drain are outlined below.

7.1 Mitigation Options

7.1.1 Option 1 – Swale with a constant depth of 600 mm

Option 1 consists of a swale with a constant depth of 600 mm and a perforated underdrain, sloping to the south parallel to the ditch where overtopping is indicated, see Figure 12. A high point along the path of the swale and underdrain creates an adverse gradient, which would prevent effective flow in the downstream sections. Therefore, the swale is shortened, and the underdrain continues as a culvert, which then conveys flows west-to-east across the centre of the site to the discharge point. The constant depth of 600 mm for the swale is in accordance with guidance from the Suffolk Flood Risk Management Strategy[11], which states that swales with filter drains should have a maximum depth of 600 mm and a maximum base width of 500 mm.

[11] Suffolk Flood Risk Management Strategy. Suffolk Flood Risk Management Partnership. February 2023.

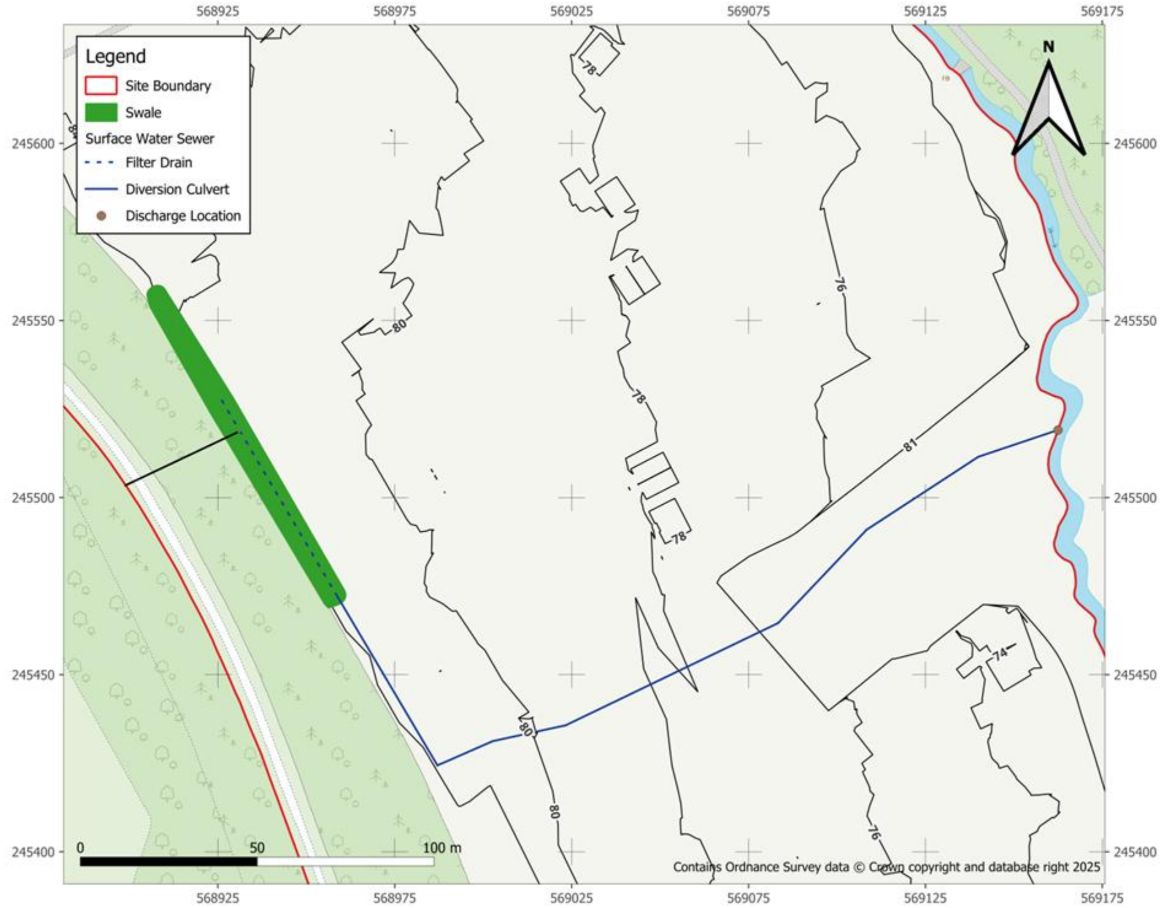


Figure 12 – Mitigation Option 1

7.1.2 Option 2 – Swale with 1:500 slope

Figure 13 shows the layout for Option 2, which consists of a swale that slopes at 1:500 to the south to overcome the adverse gradient resulting from the high point, where ground level is rising while the bed of the swale is falling. The perforated underdrain discharges to the culvert at the southern end of the swale, and the culvert conveys flows to the discharge point along the watercourse at the eastern boundary of the site. However, the deeper excavation required to overcome the high point will result in earthworks within the wooded area.

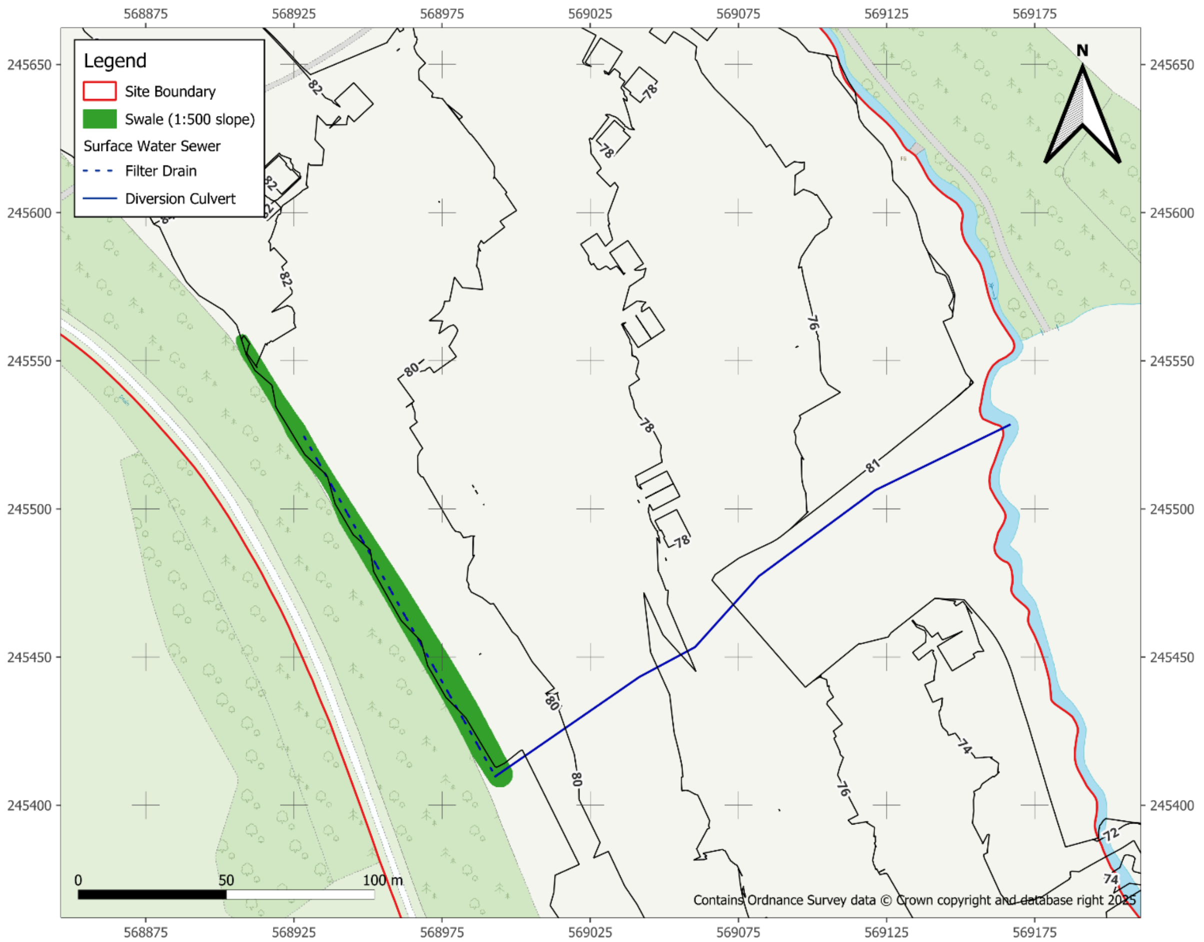


Figure 13 – Mitigation Option 2

7.1.3 Option 3 – Bund/Trench and swale system

Option 3 consists of a bund with an excavated trench at its base, which transitions to a swale, see Figure 14. The bund and the trench will run parallel to the ditch where overtopping is indicated. The trench will intercept and retain flows from the ditch, which will infiltrate into a perforated underdrain at the base of the trench. The bund/trench will transition to a swale, and the underdrain will continue sloping to the south at the base of the swale. Similar to the other options, the underdrain will transition to the diversion culvert, which conveys flows across the centre of the site to the discharge point along the eastern boundary of the site. This option reduces the earthworks required within the wooded area along the western boundary of the site.

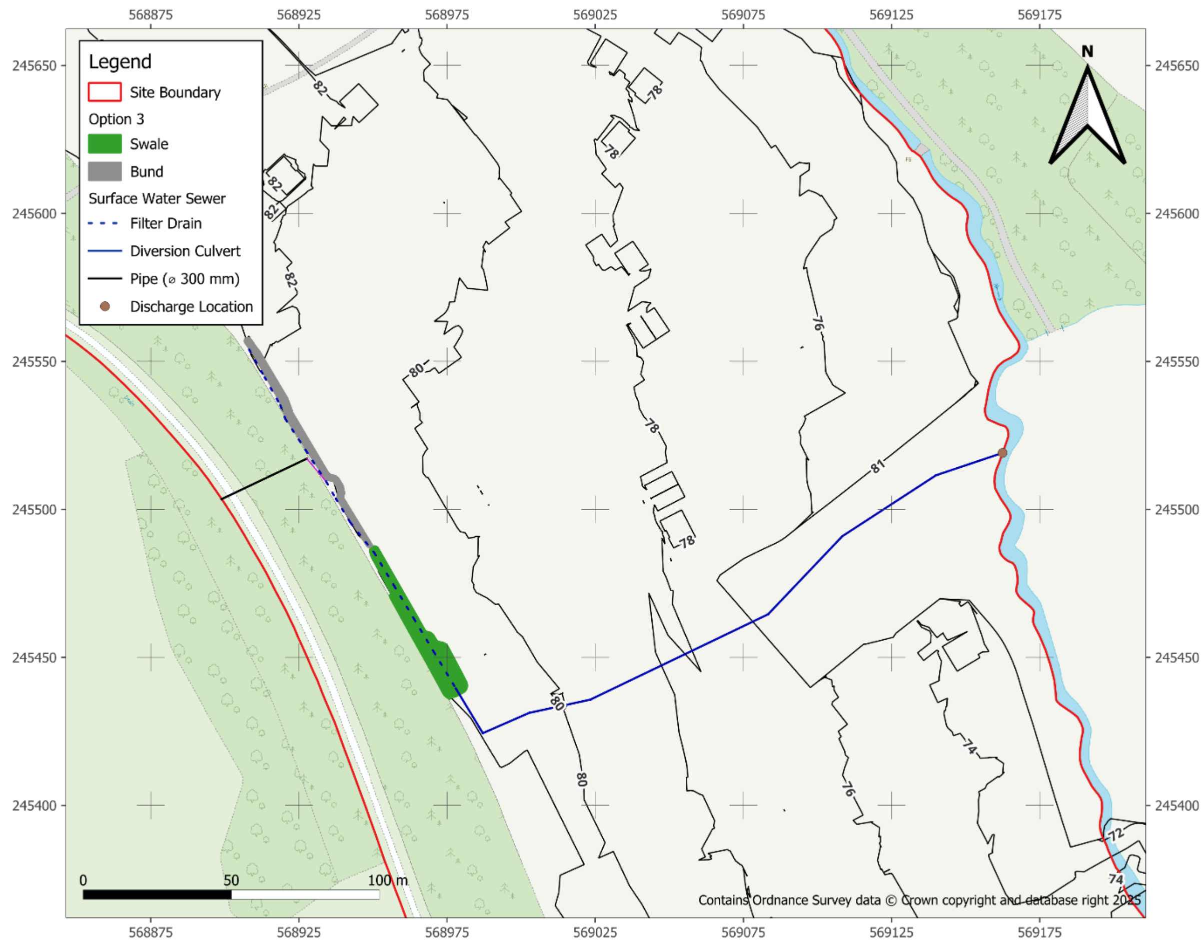


Figure 14 – Mitigation Option 3

7.2 Mitigation Model Results

7.2.1 Option 1

The mitigation results for Option 1 are shown in Figure 15, which indicates that the flood risk to the development has significantly decreased as there is a substantial reduction in flood extent and depths within the site boundary. The decrease in flood risk is due to the swale intercepting runoff that overtops the ditch and providing conveyance via its underdrain to the diversion culvert, which then directs flow to the watercourse along the eastern boundary of the site. Therefore, with the mitigation scheme in place, flooding remains largely confined to the drainage channels and isolated pockets in low-lying areas. The development surface water drainage (not included in this model) will be designed to manage the localised areas of surface water pooling.

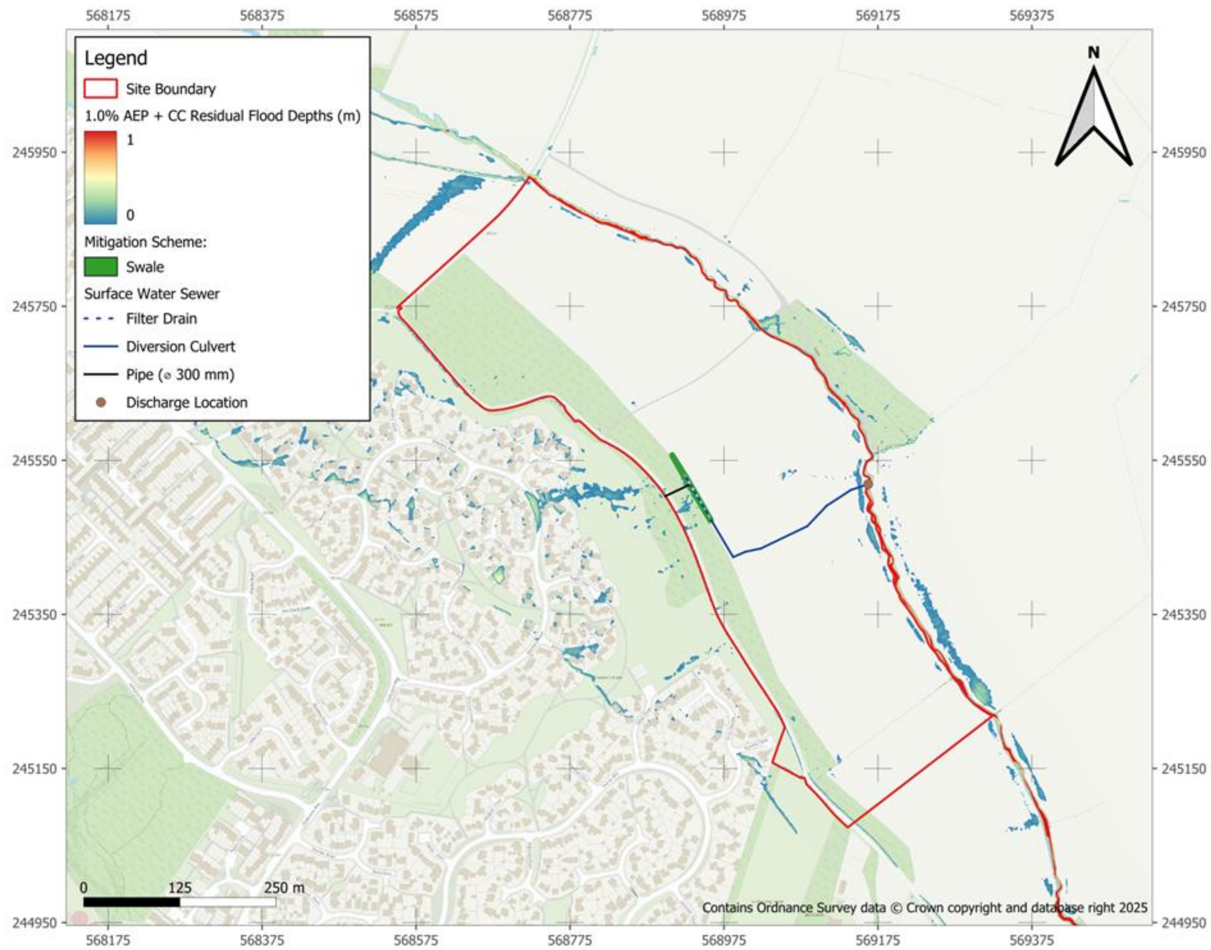


Figure 15 – Mitigation results (Option 1)

A depth change analysis was conducted to quantify the reduction in flood risk achieved by the proposed mitigation Option 1, and to verify whether the flood reduction benefits are realised without increasing impacts on third parties. Figure 16 shows the results of the depth change analysis, which indicates significant reductions in maximum flood depths of up to 100 mm along the surface water flow route identified in the baseline results. These reductions are attributed to the alternative conveyance route provided by the swale and diversion culvert. It also demonstrates that there is no significant increase in third-party impacts associated with the development, with the proposed mitigation scheme in place.

Figure 16 – Depth Change Analysis (Option 1 vs Baseline) for design event

7.2.2 Option 2

The mitigation results for Option 2 are shown in Figure 17. Similar to Option 1, the results indicate that the flood risk to the development has significantly decreased, as there is a substantial reduction in flood extent and depths within the site boundary. Therefore, with the mitigation scheme in place, flooding remains largely confined to the drainage channels, with isolated pockets in low-lying areas. The development surface water drainage (not included in this model) will be designed to manage the localised areas of surface water pooling. However, this option requires deeper excavation within the wooden area; therefore, it is not the preferred option.

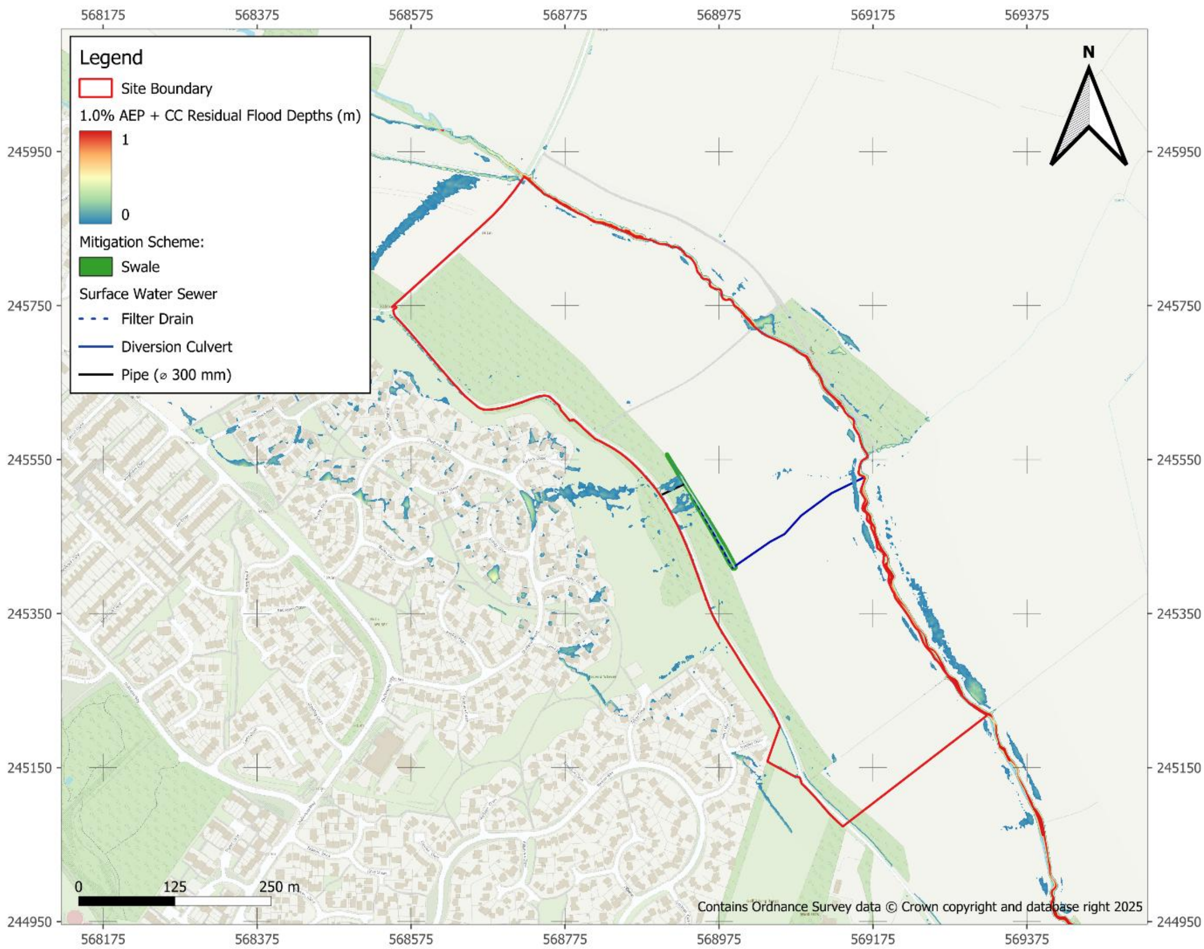


Figure 17 – Mitigation results (Option 2)

A depth change analysis was conducted to quantify the reduction in flood risk achieved by the proposed mitigation Option 2, and to verify whether the flood reduction benefits are realised without increasing impacts on third parties. Figure 18 shows the results of the depth change analysis, which indicates significant reductions in maximum flood depths of up to 100 mm along the surface water flow route identified in the baseline results. Similar to Option 1, the reduction in flood depths is attributed to the alternative conveyance route provided by the swale and diversion culvert. It also demonstrates that there is no significant increase in third-party impacts associated with the development, with the proposed mitigation scheme in place.

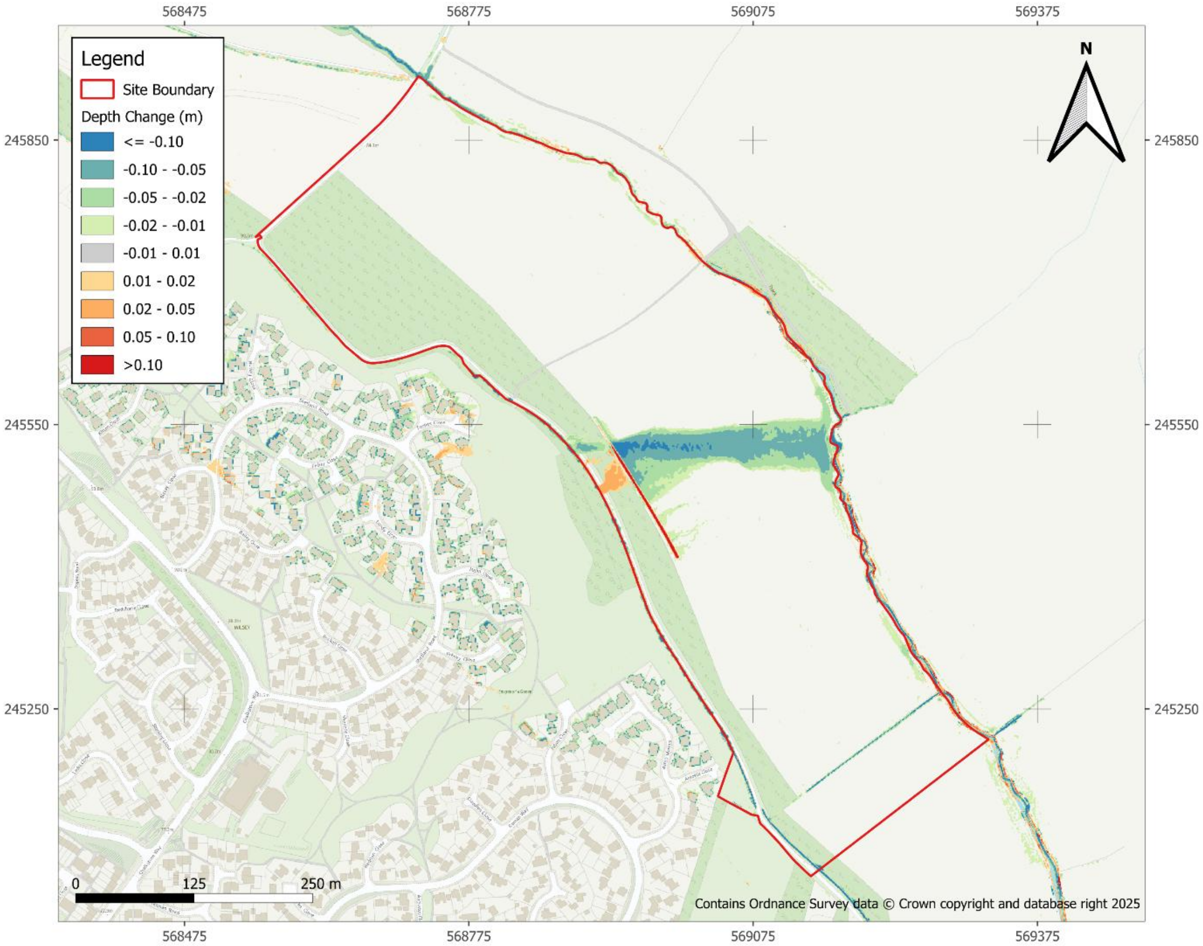


Figure 18 – Depth change analysis (Option 2 vs. Baseline) for design event

7.2.3 Option 3

The mitigation results for Option 3 are shown in Figure 19. Similar to Options 1 and 2, the results indicate that the flood risk to the development has significantly decreased, as there is a substantial reduction in flood extent and depths within the site boundary. Therefore, with the mitigation scheme in place, flooding remains largely confined to the drainage channels, with isolated pockets in low-lying areas. The development surface water drainage (not included in this model) will be designed to manage the localised areas of surface water pooling. This option is the preferred option as it requires the least excavation and reduces the impact of the layout within the wooded area while mitigating flood risk to the development area.

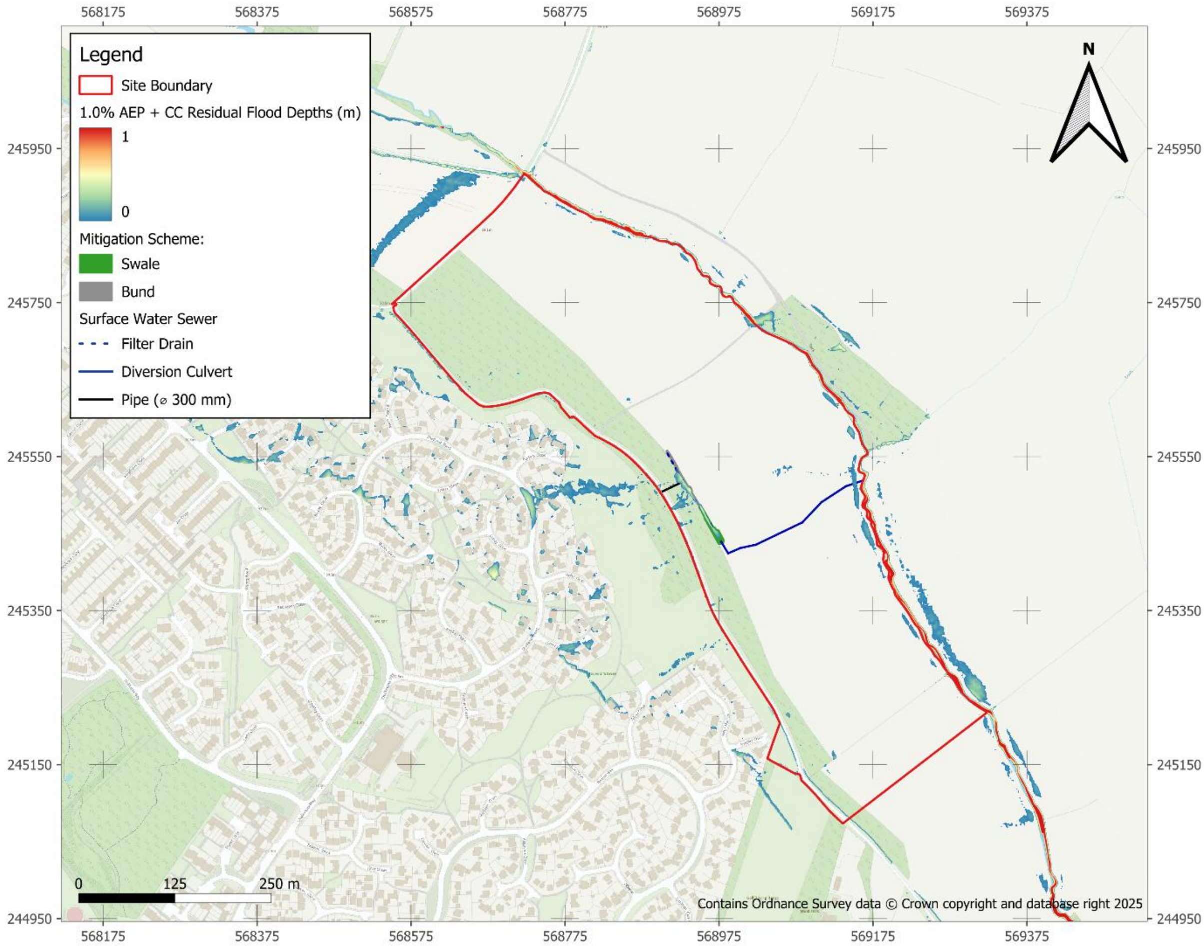


Figure 19 – Mitigation results (Option 3)

A depth change analysis was conducted to quantify the reduction in flood risk achieved by the proposed mitigation Option 3, and to verify whether the flood reduction benefits are realised without increasing impacts on third parties. Figure 20 shows the results of the depth change analysis, which indicates significant reductions in maximum flood depths of up to 100 mm along the surface water flow route identified in the baseline results. Similar to Options 1 and 2, the reduction in flood depths is attributed to the alternative conveyance route provided by the swale and diversion culvert, and the trench at the base of the bund. It also demonstrates that there is no significant increase in third-party impacts associated with the development, with the proposed mitigation scheme in place.

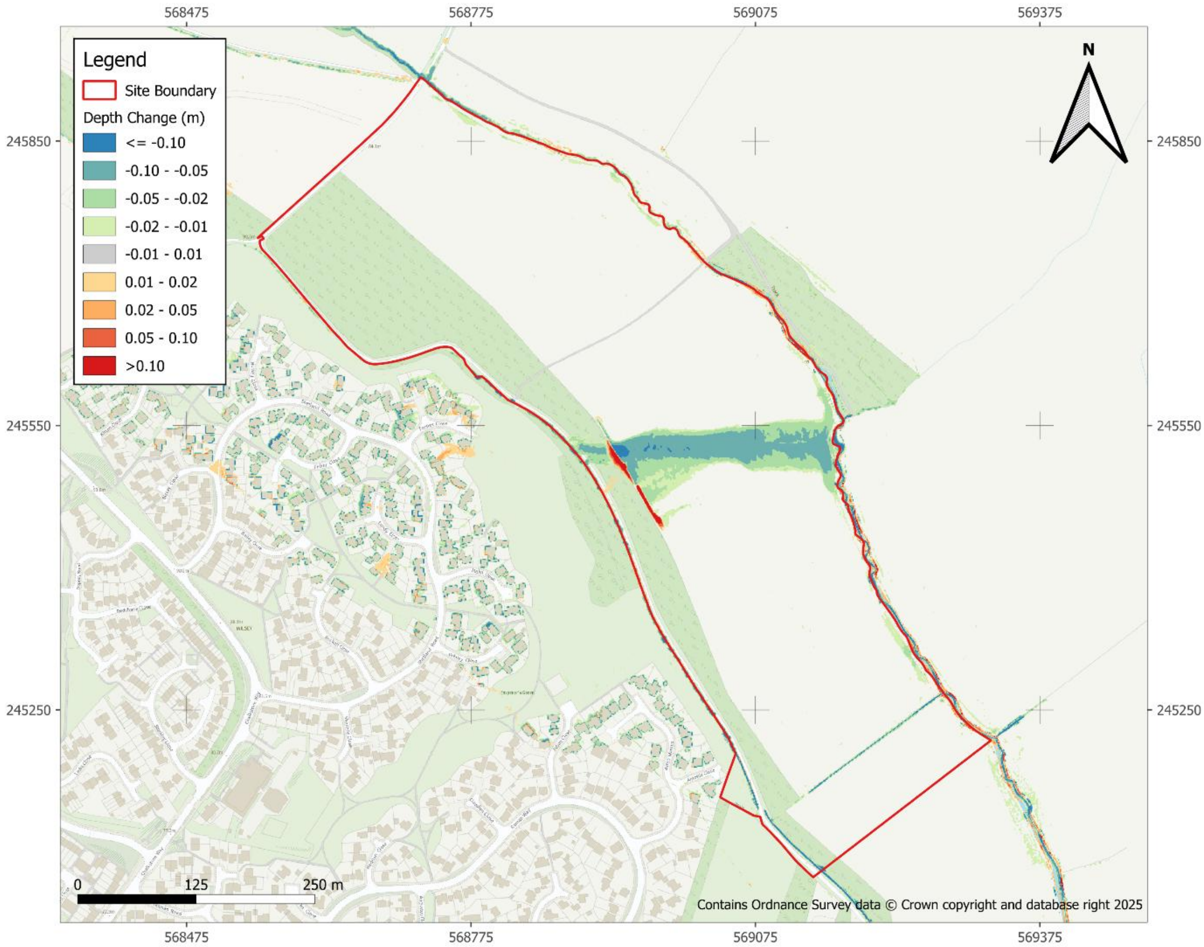


Figure 20 – Depth change analysis (Option 3 vs. Baseline) for design event

8 Sensitivity Analysis

Sensitivity analysis has been carried out on the baseline 1.0% AEP event using the following parameters:

- Manning's n value: Roughness coefficients for the floodplains have been increased and decreased by ±20%, respectively.
- Rainfall Input – the sensitivity of the model to changes in rainfall has been assessed by comparing modelled flood depths during the 1.0% AEP and 1.0% AEP plus climate change events.

The results of the sensitivity analysis have been assessed at key points within the model domain. The sensitivity sample points used to extract flood depths at these locations are shown in Figure 21.

Figure 21 – Sensitivity Sample Points

The following sections compare the results of the respective sensitivity analysis to the baseline flood levels.

8.2 Manning's n Value

Table 4 – Manning's n value sensitivity analysis

Sensitivity Point	1.0% AEP Flood Depths (m AOD)				
	Baseline	n values + 20%	Difference: Baseline vs n value + 20% (m)	n values - 20%	Difference: Baseline vs n value – 20% (m)
1	0.392	0.394	0.002	0.390	-0.002
2	0.126	0.129	0.003	0.123	-0.003
3	0.123	0.125	0.002	0.120	-0.003
4	0.139	0.141	0.002	0.137	-0.002

As shown in Table 4, the modelled flood depths at the sensitivity sample points demonstrate an acceptable response to variations in floodplain roughness. The results indicate that flood depths increase with higher roughness values and decrease when lower roughness values are applied.

8.3 Rainfall Input

The results of the rainfall sensitivity analysis are shown in Table 5 below. The flood depths are based on the 1.0% AEP and 1.0% AEP + climate change rainfall events.

Table 5 – Rainfall input sensitivity analysis

Sensitivity Point	1.0% AEP Flood Depths (m AOD)	1.0% AEP + CC Flood Depths (m AOD)	Difference (m)
1	0.392	0.417	0.025
2	0.126	0.153	0.027
3	0.123	0.133	0.010
4	0.139	0.158	0.019

As shown in Table 5, the modelled flood depths at the sensitivity sample points increase in response to higher rainfall inputs. These increases are within the expected range, consistent with the model's sensitivity parameters.

9 Model Stability and Limitations

9.1 2D Model Stability and Limitations

One of the main indicators of model stability with the HPC solver is the timestep selected by TuFLOW for the model runs. It is recommended that the timestep should not be less than one-tenth of the TuFLOW classic timestep. In this case, a timestep of 0.5s has been selected for the classic TuFLOW solver, and the grid resolution has been set to 1m. Hence, the HPC timestep should not fall below 0.05s.

To confirm the stability of the model, the evolution of the 2D timestep (adaptive timestep size - dtStar) throughout the model run was graphically reviewed. In addition to dtStar, three control numbers (Nu, Nc and ND) were reviewed and included in the plot to determine if any hydraulic conditions were limiting the timestep. The definition and stability criteria for the parameters are shown in Table 6 below.

Table 6 – Stability Parameters and Indications

Parameter	Definition	Stability Insight
dStar	Adaptive timestep size (s)	Lower values indicate potential instabilities requiring smaller timesteps to solve. Very low or rapidly changing dtStar means potentially poor model stability.
Nu	Upwind weighting factor (0 to 1)	Close to 0 = central differencing (more accurate, less diffusive). Close to 1 = fully upwind (more stable but less accurate). Increased Nu implies TuFLOW is applying more upwinding to maintain stability. Values > 1 indicate that the velocity is unusually high, or the cell size is too small for the modelled velocity.
Nc	Cell Stability counter (0 to 1)	Represents the proportion of cells passing all numerical stability checks. Values close to 1 = good stability. Values > 1 can be caused by a large depth-to-cell-size ratio. Dips in Nc indicate parts of the domain are struggling.
Nd	Depth ratio stability control	Tracks variation in cell depth over time. Higher values generally reflect more stable flows. Dips could suggest instability or abrupt changes in flow depth. Values > 0.3 suggest there is potentially poor boundary setup or insufficient SX cells linked to the 1D structure, or the cell size is too small.

The plot of the model stability indicators (see Figure 22) was reviewed, and they were found to be acceptable, as:

- dtStar is always above 0.05s, suggesting the model is dynamically adjusting time steps in response to changing flow conditions.
- Nu values are closer to 0, though some increases indicate the solver is applying more upwinding. However, the values eventually reduced, showing improved conditions.
- Nc increases and stabilises at 1.0, showing all cells are stable, but dips throughout the simulation, indicating surcharging or flow transition. Nc recovers to 1.0, showing the model regains full stability.
- Nd values show general consistency with mild variation indicating flow depths are evolving smoothly.

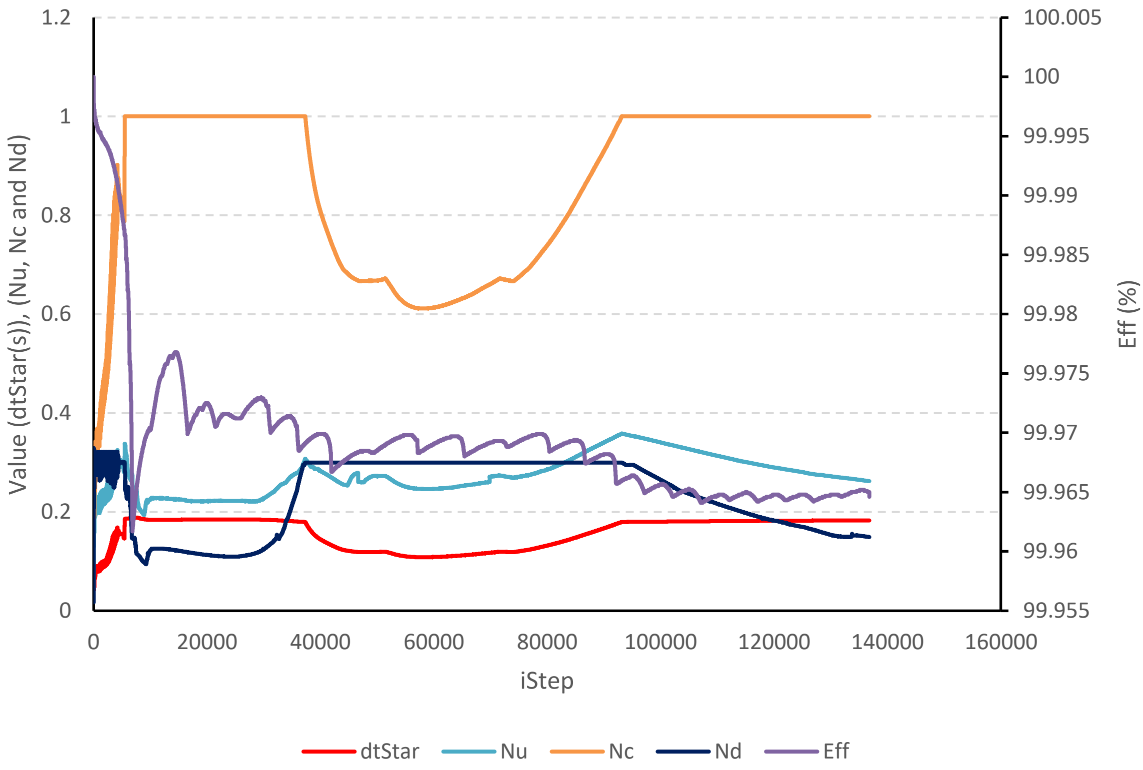


Figure 22 – Model Stability Indicators

9.2 Checks and Warning Messages

The check and warning messages present in the TuFlow log file upon completion of the model run for the design 1.0% AEP + climate change event are summarised in Table 7.

Table 7 – TuFLOW check and warning messages

ID	Count	Description	Comment
Check 2118	11	Lowered SX ZC Zpt by 0.13 m to 1D node bed level.	A 'Z' flag has been used to adjust the cell centre (ZC) elevation at each cell at/along the 2D SX object to below the 1D node bed elevation where ZC is higher.
	2	Lowered SX ZC Zpt by 0.73 m to 1D node bed level.	
Check 2541	27	BC polyline selects no cells, using midpoint.	The source area (SA) polygon centroids fell within the active model domain, and their centroid were used to choose the nearest cell as a connection between the 1D and 2D cells.
Check 2583	14	Material ID 1 contains a Manning's n value (0.400) greater than Wu n limit (0.100) - n value will be limited in Wu formulation	Values have been reviewed and are considered acceptable.
Warning 3526	2	SGS Sample GRID Distance command is ignored in SGS Approach == Method C	Method C sub-grid sampling was used, where SGS Sample Frequency or SGS Sample Target Distance is used instead of SGS Sample Distance.
Check 3548	3	Setting SGS Sample Distance Target to minimum grid zpt resolution of 0.09996.	SGS Sample Target Distance set to the minimum raster grid resolution to compute the sampling frequency.

10 Conclusions

This report has detailed the methodology used to develop the pluvial model required to confirm and quantify the flood risk to the site indicated in the Environment Agency's Risk of Flooding from Surface Water (RoFSW) mapping. It also details the mitigation options that were considered and assessed to manage the indicated flood risk to the site. The report is summarised below:

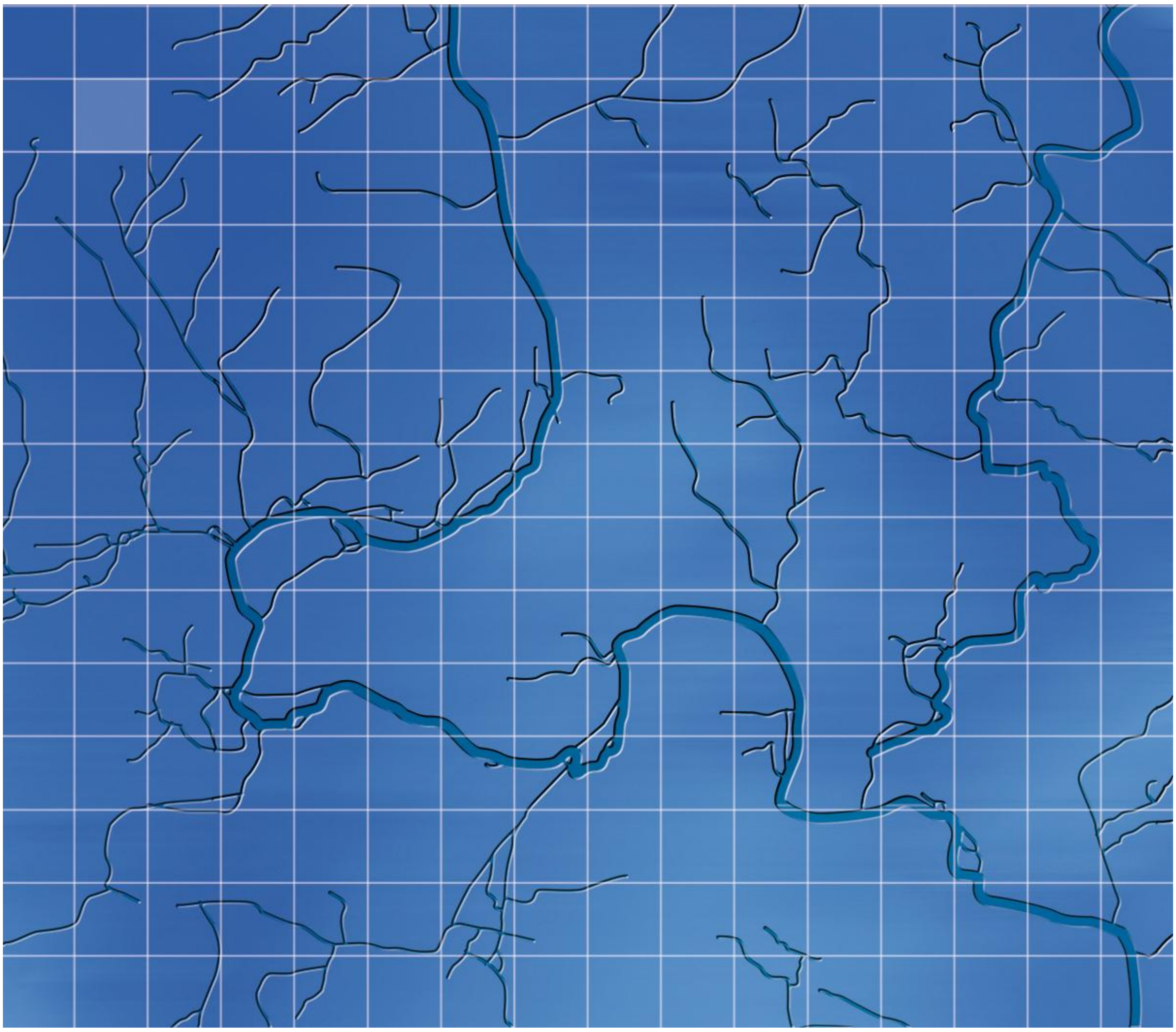
- A 1D/2D ESTRY TuFlow model was produced to identify and quantify flood risk to the site and to assess the proposed mitigation options to manage any identified flood risks.
- To establish the baseline conditions, the model has been run for the 1.0% AEP, 1.0% AEP + 25% climate change allowance and the 0.1% AEP events. The model results are presented in the report.
- The baseline results indicated the centre of the site is at risk of flooding due to a surface water flow route resulting from the overtopping of the ditch along western boundary of the site. The maximum flood depth associated with the design event along this flow route is 0.2 m.
- A post-development scenario was produced by incorporating the proposed ground levels into the baseline model. However, it indicates that the flooding from the ditch will continue to overtop and enter the site despite the proposed reprofiling of the development area. After overtopping and entering the site, the flooding is diverted south along a roadway in the centre of the site.
- Mitigation Options 1, 2 and 3 were designed to reduce the flood risk to the development, and their effectiveness was assessed through mitigation modelling.
- The options used different variations of a swale with a perforated underdrain to intercept and retain flows overtopping ditch. The retained floodings then passes through the base on the swale into the underdrain. The underdrain then conveys flows to a culvert at the southern end of the swale, which then continues diverts these flows west-to-east across the centre of the site to the discharge location along the eastern site boundary.
- Option 1 consists of a shortened swale with a constant depth of 600 mm, Option 2 consists of a swale with a 1:500 slope (to overcome a high point along the path of the swale), and Option 3 consists of a bund/trench and swale combination. The layouts for the options are shown in the report.
- The mitigation modelling for these three options indicates flood risk to the development has significantly decreased as there is a substantial reduction in flood extent and depths within the site boundary.
- The decrease in flood risk is due to the swale intercepting runoff that overtops the ditch and providing conveyance via its underdrain to the diversion culvert, which then directs flow to the watercourse along the eastern boundary of the site.
- The depth change analysis undertaken for each of the options confirms the reduction in the indicated flood risk, as there are reductions in maximum flood depths of up to 100 mm along the surface water flow route identified in the baseline results. It also demonstrates that there is no significant increase in third-party impacts resulting from the options.
- Of the options considered, Option 2 is the least preferred as it requires deeper excavation depths, which encroach into the wooden area along the western site boundary. Option 1 requires less excavation due to the shortened swale; however, *Option 3 is the preferred option as the bund/trench combination with the swale requires the least excavation and reduces the encroachment onto the wooden area, therefore reducing the impact of the layout.*

Appendix 1 - Great Wilsey Park Pluvial Assessment Report

Paul Basham Associates

March 2025

Pluvial Assessment for Great Wilsey Park Modelling



Wallingford HydroSolutions Limited

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Paul Basham Associates

Pluvial Assessment for Great Wilsey Park Modelling

Document issue details

WHS10203

Version	Issue date	Issue status	Prepared By	Approved By
1.0	26/03/2025	Draft	Ajani Jacobs (*Consultant*)	Daniel Hamilton (*Principal Consultant*)

For and on behalf of Wallingford HydroSolutions Ltd.

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Contents

1 Introduction

Wallingford HydroSolutions have been commissioned by Paul Basham Associates to undertake detailed pluvial hydraulic modelling. This is in order to assess pluvial flood risk at a proposed residential development site in Wilsey, Haverhill (NGR: 569086, 245529). In this regard, a pluvial assessment is required to derive rainfall hyetographs as input into the hydraulic model.

2 The Catchment

As the study requires pluvial modelling of the catchment it was first necessary to derive a pluvial catchment area. The EA's Most Probable Overland Flow Pathway dataset[1] has been used to derive the catchment area. LiDAR data, existing surface water flood maps, and the FEH Web Service have subsequently been used to check the size of the pluvial catchment area.

The catchment derived is also used to define the active area within the model domain, allowing all the rainfall which may have an impact on the site to be included in the assessment. The overall size of the catchment was estimated to be 0.54 km^2.

As this is a small catchment (< 5 km^2) and has been manually defined, FEH point data has been obtained from the catchment centroid and was extracted from the FEH Web Service[2] to define the pluvial catchment. The FEH point data has BFIHOST19 value of 0.361, indicating a substrate of low to moderate permeability. This was cross checked against the underlying geology to confirm that it captured the geological variation in the area. Based on BGS Geology Viewer[3], the catchment is characterised by bedrock consisting of various formations of chalk formation, which has relatively high permeability. There are also predominantly superficial deposits of diamicton throughout the catchment with deposits of clay, silt, sand and gravel along the watercourse channel. The Landis Soilscapes Map[4] was also reviewed to confirm the drainage capacity of the underlying soils. The map shows the catchment consists of clayey, some loamy soils with slightly impeded drainage. Though BGS shows the catchment to be underlain by a substrate with relatively high permeability, the clayey soils help explain the low BFIHOST19 value, which is considered to be representative. Table 1 provides information about the point data.

Table 1 – Point descriptors

Point Descriptor	
BFIHOST19	0.361
PROPWET	0.26
SAAR6190	583 mm

The catchment area is shown in Figure 1, with the point location marked.

[1] Overland Flow Pathways. Environment Agency. 2024.
[2] FEH Web Service. Available at: https://fehweb.ceh.ac.uk/GB/map
[3] British Geological Survey (BGS) Geology Viewer (2023). Available at: https://geologyviewer.bgs.ac.uk/
[4] Landis Soilscapes Viewer (2009). Available at: https://www.landis.org.uk/soilscapes/

Figure 1 – Diagram illustrating the Pluvial catchment and location of point data

3 Outline Methodology

3.1 Rainfall Data

The point FEH22 DDF rainfall was obtained from the FEH Web Service at the same location as the point descriptor data. The rainfall model is produced at a 1km resolution. As the catchment is relatively small, the analysis has been based upon the single point descriptor.

3.2 Derivation of Net Urban & Rural Runoff

Separate parts of the model domain will be defined as being urban or rural and separate urban and rural input rainfall hyetographs were produced as input to the hydraulic model.

The ReFH2.3 software was used to derive the hyetographs. The rainfall-runoff methods underlying ReFH2.3 are those first published by Kjelsden[5], which were subsequently updated in 2015 and 2019 and implemented within the software as described in the WHS technical guidance[6]

[5] Kjeldsen, T. R. 2007. The revitalised FSR/FEH rainfall-runoff method. Supplementary Report No.1. CEH.
[6] https://www.hydrosolutions.co.uk/software/refh-2/

ReFH2.3 uses the point descriptors to calculate the rainfall, and loss parameters, which are used to derive net rainfall. Three assumptions were adopted:

- Rainfall, infiltration and other losses are modelled in ReFH2
- The resulting Net rainfall (runoff) estimate by ReFH is applied to the 2d model domain
- Sewer losses are modelled in TUFLOW rather than ReFH2

Two separate hyetographs were derived for each storm duration, representing net rural rainfall, and net urban rainfall. In terms of duration, hyetographs were derived for the following storms:

- Duration = 1hr, Timestep = 4min
- Duration = 3hr, Timestep = 12min
- Duration = 6hr, Timestep = 24min
- Duration = 9hr, Timestep = 36min
- Duration = 12hr, Timestep = 48min
- Duration = 18hr, Timestep = 72min

Two storm profiles were also considered, the 75% winter profile (generally recommended for rural catchments) and the 50% summer profile (recommended within urban catchments). These were applied to both the rural and urban analysis. Therefore, for each duration event there were a total of four separate hyetographs. However, for the final model runs only the summer profiles were used as they had higher rainfall volumes which allowed for a more conservative assessment of flood risk.

The hyetographs are defined over the area of the pluvial catchment (0.51 km^2). The Areal Reduction Factor (ARF) is set to 1 and the seasonal correction factor (SCF) is kept at its default values as defined for the catchment area and seasonality applied.

For the urban hyetographs, the impervious runoff factor (IRF) and impervious factor were both set at 1.0 to match the values used by the EA[7] in their latest national hazard mapping. All other urban parameters were maintained at their default values.

4 Results

The table below shows the peak rainfall across a timestep for each duration applying the summer seasonality. The results are derived from the net rainfall values extracted from ReFH2.

Table 2 – Peak net rainfall

	1 Hour Peak Net Rainfall (mm)	**3 Hour Peak Net Rainfall (mm)**	**6 Hour Peak Net Rainfall (mm)**	**9 Hour Peak Net Rainfall (mm)**	**12 Hour Peak Net Rainfall (mm)**	**18 Hour Peak Net Rainfall (mm)**
	1000yr	1000yr	1000yr	1000yr	1000yr	1000yr
Urban Summer	8.06	20.71	25.45	28.36	30.30	32.65
Rural Summer	5.98	8.96	11.82	13.66	15.00	16.61

[7] EA Overarching document 'Flood Maps for surface Water: How were they produced' available from https://www.gov.uk/government/publications/flood-maps-for-surface-water-how-they-were-produced