Paul Basham Associates

Great Wilsey Park Channel Crossing Assessment

Document issue details

WHS10203

Version	Issue date	Issue status	Prepared By	Approved By
1	20/11/2025	Draft	Ajani Jacobs (*Consultant*)	Brett Park (*Principal Consultant*)
2	24/11/2025	Final	Ajani Jacobs (*Consultant*)	Brett Park (*Principal Consultant*)

For and on behalf of Wallingford HydroSolutions Ltd.

WHS

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Contents

1 Introduction

1.1 Background

Wallingford HydroSolutions Ltd (WHS) has been commissioned by Paul Basham Associates to undertake bespoke hydraulic modelling to inform the size of 2 No. proposed crossing points over an existing ditch to allow access to the future development area northeast of the proposed residential development site in Wilsey Haverhill (NGR: 569086, 245529).

Detailed fluvial modelling is required to quantify the flows within the ditch where the crossings are proposed, ensuring an accurate representation of flows from the larger contributing catchment.

1.2 Methodology

A 1D-2D fluvial model of the catchment containing the proposed development has been constructed for the channel crossing assessment and to assess the likelihood of any associated third-party impacts.

The hydraulic model was produced using ESTRY-TuFLOW hydraulic modelling software, with flow inputs estimated from a hydrological assessment of the ditch running along the eastern boundary of the Site. The 1D model component was informed by survey data provided by the client.

1.3 Data Sources

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

- Peak Flow Assessment (attached as Appendix 1).
- LiDAR data from the National LiDAR Programme[1]
- Site XML file[2] – final and existing contours, and site layout
- Client supplied topographical surveys[3,4] (attached as Appendix 2)

1.4 Assumptions

The model was built based on the following assumptions:

- The LiDAR and survey datasets are suitable for informing the hydraulic model.
- It is acceptable to generate the river cross-sections from the topographical survey data of the channel.
- Sensitivity analysis is appropriate to test model robustness and uncertainty.

[1] LiDAR Compositive Digital Terrain Model (DTM) – 1m. Environment Agency. November 2024. Available at: https://environment.data.gov.uk/survey
[2] S3D Final & Existing Contours. Paul Basham Associates. March 2025. Dwg. No. 1028.5003
[3] Topographical Survey. Interlock Surveys. February 2025. Dwg. No. 141025 3D
[4] Topographical Survey. Survey Solutions. August 2025. Dwg. No. 79930BWLS-01

2 Site Description

2.1 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

To estimate the peak flows and hydrographs for input into the hydraulic model, the catchment area has been defined using a single inflow at the downstream boundary of the model at E: 569331, N: 245207. The peak flows were estimated by applying the FEH methods based on the catchment descriptors for the derived catchment, obtained from the FEH Web Service. The FEH methods used include the statistical method applied in WINFAP-FEH 5 and the rainfall-runoff method applied in ReFH2.3. A diagram showing the catchment used is shown in Figure 2.

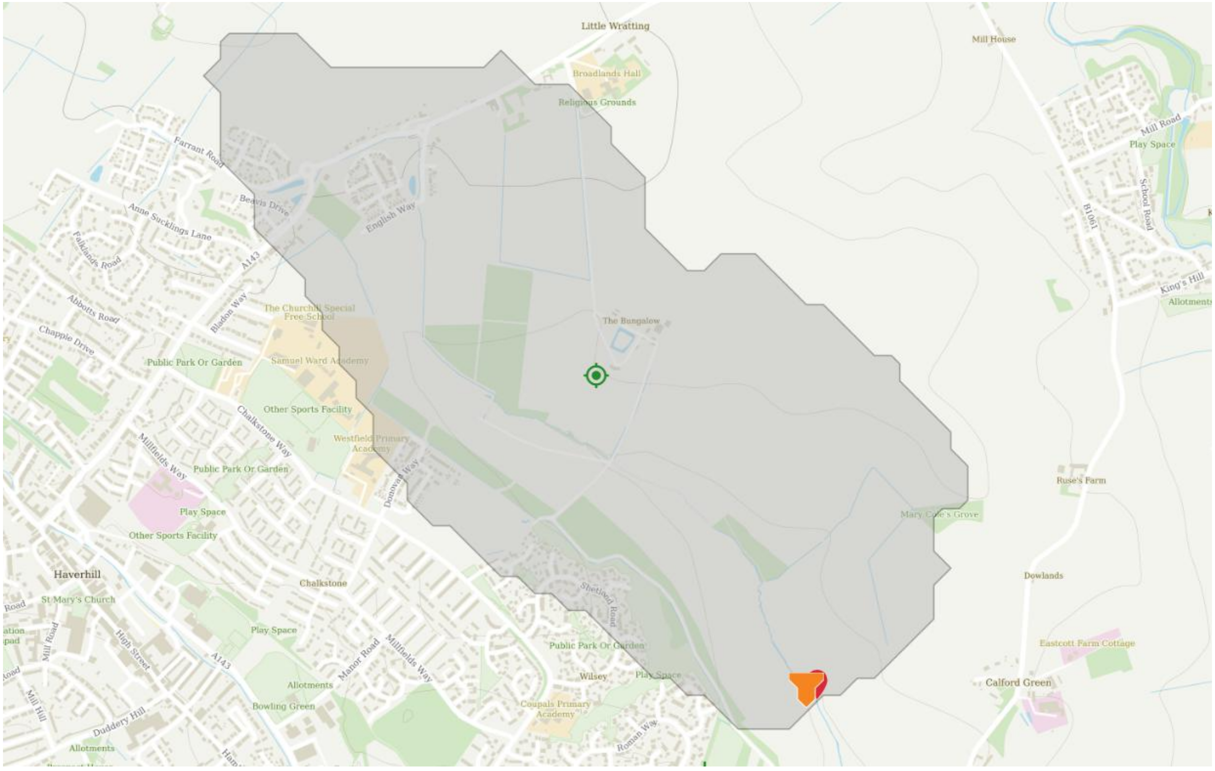


Figure 2 – The FEH Web Service derived catchment is shown by the grey boundary. Contains OS data © Crown Copyright (2023) Contains CEH data © and database right NERC (CEH) 2023

A potential field drain sub-catchment with a different outlet from the main watercourse was identified within the western boundary of the FEH catchment. However, due to the interconnectivity of the field drains and overland flow routes between the two catchments, it was not possible to delineate the full length of this sub-catchment without detailed survey data. As uncertainty exists, this sub-catchment has been conservatively included within the FEH catchment boundary to provide a precautionary approach regarding flood risk.

The method for the peak flow estimate is detailed in the full hydrology assessment report attached as Appendix 1, and the final peak flow estimates are presented in Table 1.

Table 1 – Final Peak Flow Estimates

Return Period (years)	Peak Flow estimate (m^3/s)
2	0.921
25	1.890
50	2.150
100	2.433
500	3.401
1000	4.210

4 Hydraulic Model Build

4.1 1D Domain

The 1D domain was modelled using the TuFlow ESTRY using the latest software release.

4.1.1 Model Extent

The 1D model extent has been modelled as a 1.5 km section for the watercourse along the eastern boundary of the Site based on the upstream and downstream model extents at NGR: 568531, 246026 and NGR: 569412, 244910, respectively. The 1D model extent and network are shown in Figure 3.

Figure 3 – 1D Model Extent

4.1.2 Watercourse Channel Geometry

The channel geometry of the watercourse was informed by survey data provided by the client, where:

- Cross-sections were extracted from a ground model[2] of the existing site provided in land XML format. The cross-sections were extracted at 100 m intervals to ensure the channel geometry was appropriately represented in the model.
- Invert levels for the structures and geometry for existing structures were informed by the site topographical survey[3] and a separate survey/assessment[4] of the structures.

4.1.3 Boundary Conditions

The inflows estimated from the peak flow assessment were applied to the 1D model using a Flow-Time (QT) boundary at the upstream extent of the 1D network.

A rating curve, i.e., Stage-Flow (HQ) boundary, was applied at the downstream extent of the 1D network. The water levels for the HQ boundary were extracted from the cross-section data at the downstream extent of the model and are the levels at which the hydraulic properties are calculated.

The flows are based on the hydraulic property of this downstream section (i.e., the conveyance), which is multiplied by the square root of the slope from the last two river sections of the channel. The rating curve used as the downstream boundary is shown in Figure 4.

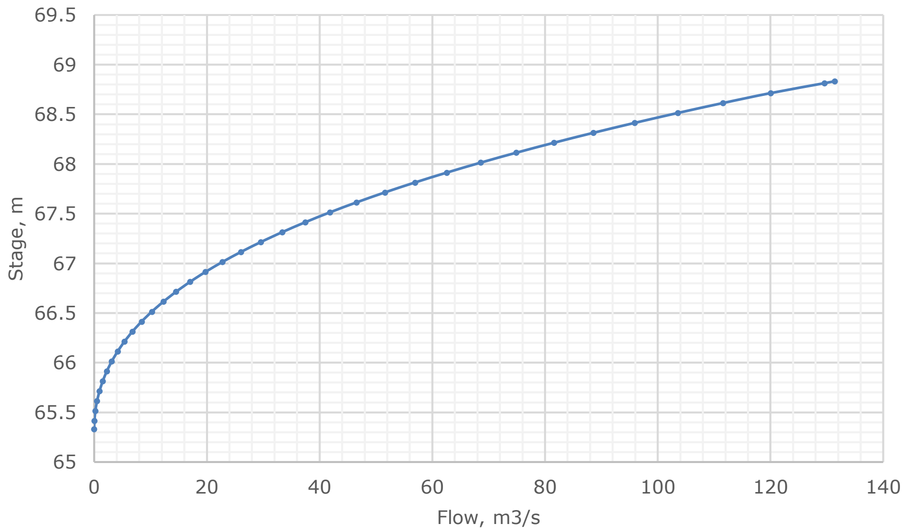


Figure 4 – HQ Downstream Boundary

4.1.4 Initial Conditions

1D initial conditions were not required for the model runs. However, the model was run for 4 hr before the start of the flood event to 'pre-wet' the model.

4.1.5 River Channel Roughness

The Manning's values used to represent the surface roughness are summarised in Table 2. The selected values are based on engineering judgement and published guidelines values[5].

Table 2 – Summary of Manning's n values for 1D network

Domain	Description	Manning's n value
1D	Natural Channel (Bed) – clean winding, some pools and shoals	0.04
1D	Banks – very weedy reaches	0.10

[5] Open-channel Hydraulics. Chow, V T (1959).

4.1.6 River Channel Structures

The representation of river channel structures in the 1D network was guided by the structure sections and topographical surveys provided by the client. A total of 6 structures were surveyed however, only 3 were represented in the model. A summary of the structures is provided in Table 3 and the survey data is attached as Appendix 2.

Table 3 – Summary of in-channel structures

Node	NGR	1D Structure Type	Modelled	Photo
Structure1	568725, 245917	Irregularly shaped culvert (I)	Y	
Structure2	569023, 245723	Circular culvert (C)	Y	

Structure3	569135, 245620	Footbridge (represented in model as increased channel roughness, n = 0.05)	Y	
Structure4	569180, 245560	-	N culvert on lateral ditch – outside model extent	
Structure5-6	569469, 244853	-	N outside model extent	

4.2 2D Domain

4.2.1 Extent

The 2D model extent is shown in Figure 5. The active area has been digitised around the maximum flood extent, which at this location is the 1.0% Annual Exceedance Probability (AEP) event, including an allowance for climate change.

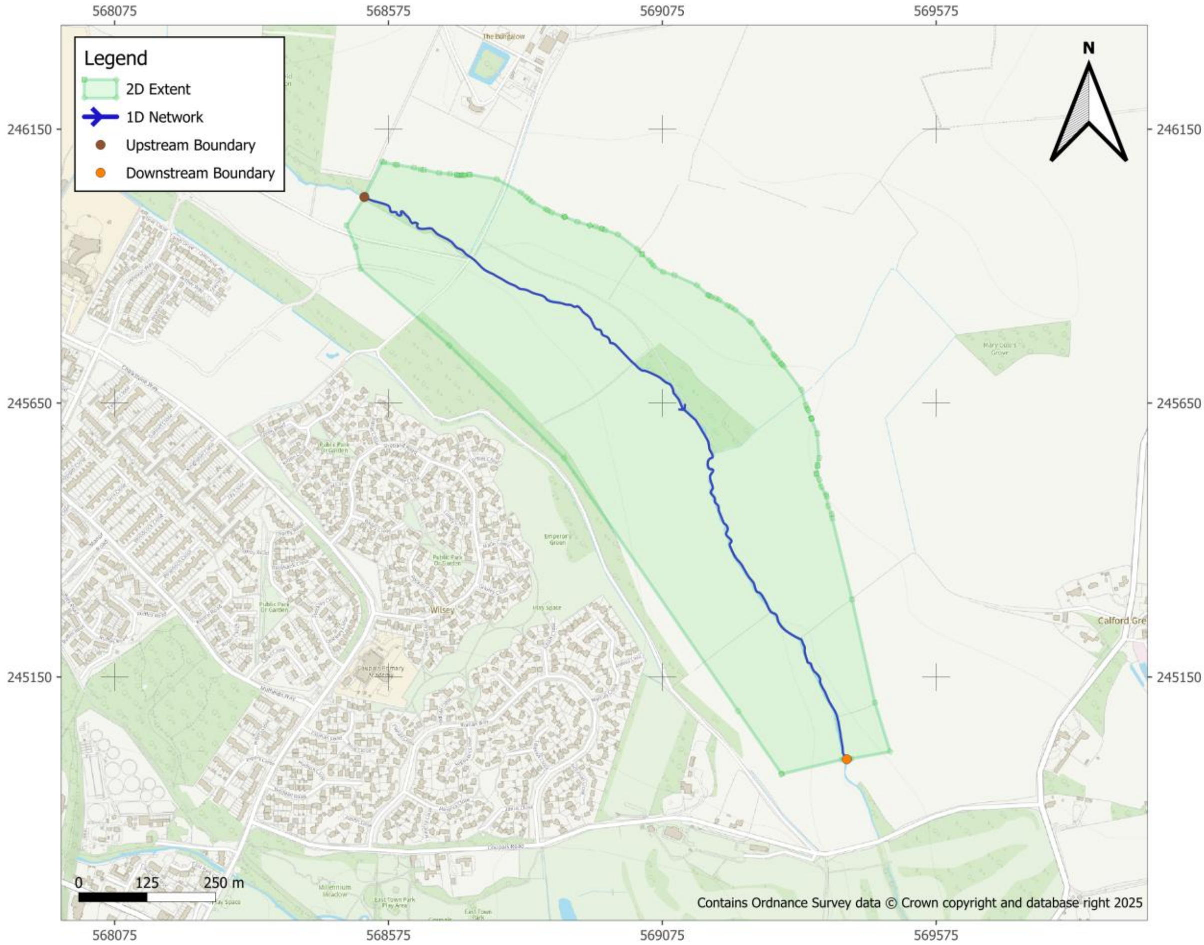


Figure 5 – 2D Model Extent

4.2.2 Grid Size and Orientation

The model grid size has been set to 2m, and the orientation of the grid is defined by the GIS location line in the TuFlow Geometry Control (TGC) file, which is digitised, allowing the orientation of the grid to align with the predominant flow direction of the watercourse.

4.2.3 LiDAR Data

LiDAR data[1] at a resolution of 1 m has been used to inform the topography of the floodplain.

4.2.4 DTM Modifications

The ground model of the Site was incorporated into the model to more accurately represent the existing levels across the Site and surrounding areas, including the section of the ditch flowing along the eastern boundary of the Site. As the ground model was in Land XML format, it was read into the model using the 'Read TIN Zpts' commands. This instructed TuFlow to assign the elevation values from the ground model to the Z points of the 2D computational area.

4.2.5 Boundary Conditions

2D Stage-Flow (HQ) boundaries were applied to allow water to escape the model when it reached the downstream extent 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 layer. All areas where the terrain gradient was calculated and applied at a boundary location are shown in Table 4 and Figure 6.

Table 4 – Gradient within HQ boundaries

HQ Boundary	Gradient, b (m/m)
A	0.0137

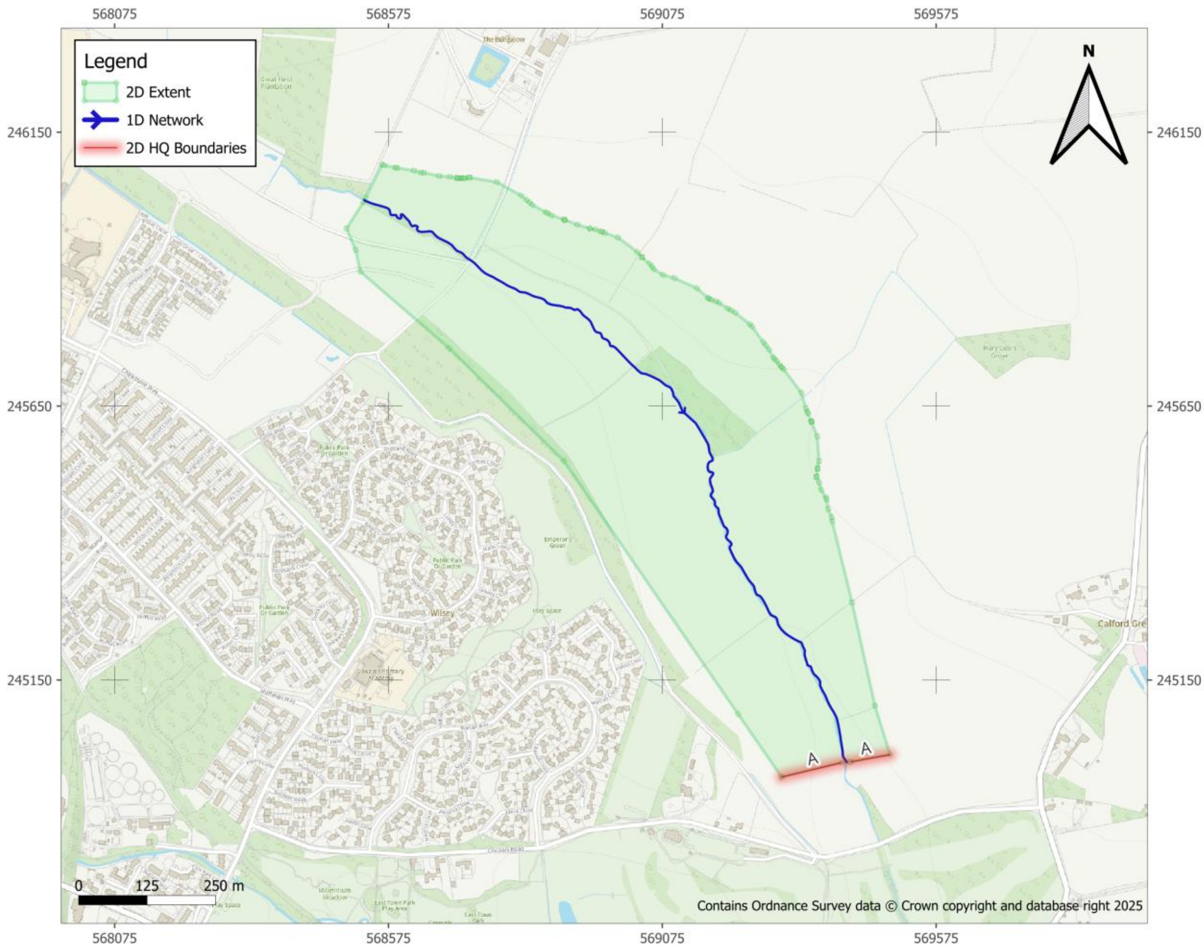


Figure 6 – 2D HQ Boundaries

4.2.6 2D Initial Water Levels

The initial conditions within the model are the initial water levels for the watercourse, represented using a 2D_IWL layer. The levels for this layer are extracted from the LiDAR data.

4.2.7 Surface Roughness

The floodplain roughness has been defined using Manning's n roughness coefficient. As detailed in section 4.1.5, 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 an appropriate roughness coefficient. These are detailed in Table 5 below.

Table 5 – 2D domain surface roughness values

Land Use	Material Code	Manning's n value
Building	1	0.300
General surface (yards, lawns, fields)	2	0.040
Surface Water	6	0.025
Woodland	9	0.100
Roads	11	0.025
Hard Standing	14	0.030

5 Design Runs

5.1 Summary of Design Runs

The hydraulic model has been run for each of the events below to obtain baseline results.

- 1.0% AEP event
- 1.0% AEP event plus a 25% central climate change allowance
- 0.1% AEP event

The climate change allowance has been applied based on the EA guidelines[6] for climate change estimation for the Combined Essex Management Catchment.

5.2 Confidence is Baseline Model Results

To establish confidence in the modelled results for the baseline scenario, a like-for-like comparison was undertaken between the modelled flood extents and the EA Risk of Flooding from Rivers and the Sea (RoFRS) map, as shown in Figure 7 and Figure 8.

Differences can be observed between the baseline model and the RoFRS flood extents for both the 1.0% and 0.1% AEP events, particularly along the southern section of the watercourse, where the RoFRS maps show more out-of-bank flows. The differences are likely due to the inclusion of detailed survey data into the baseline model, which improved the representation of the watercourse and bank levels. This enhancement allowed for greater in-channel conveyance compared to the national-scale RoFRS model.

Despite these differences, the overall flood extents and locations are broadly similar for the 1.0% AEP event. However, the flood extent and location for the baseline 0.1% AEP event are notably smaller than those for the EA RoFRS 0.1% AEP event. These variations are expected and do not undermine confidence in the baseline model, as it uses high-resolution LiDAR and ground survey data, which are inherently more accurate than the national dataset.

[6] Climate change allowances for peak river flow in England. Environment Agency. Available at: https://environment.data.gov.uk/hydrology/climate-change-allowances/river-flow

Figure 7 – 1.0% AEP Modelled Flood Extent vs EA RoFRS 1.0% AEP Flood Extent

Figure 8 – 0.1% AEP Modelled Flood Extent vs EA RoFRS 0.1% AEP Flood Extent

5.3 Baseline Scenario Modelled Results

Figure 9 shows the baseline scenario for the design 1.0% AEP + climate change and the 0.1% AEP events. It indicates that the flows mostly remain in the channel during the design events, and out-of-bank flooding is limited – mainly near the upstream section at the northeastern corner of the Site and around the central crossing. The 0.1% AEP flood depths show deeper flooding near the channel, with limited spread into adjacent land along the site boundary.

Figure 9 – Baseline Results

6 Post-Development Modelling

A post-development model scenario was generated by incorporating the proposed ground model and the watercourse crossings into the baseline model, see Figure 10.

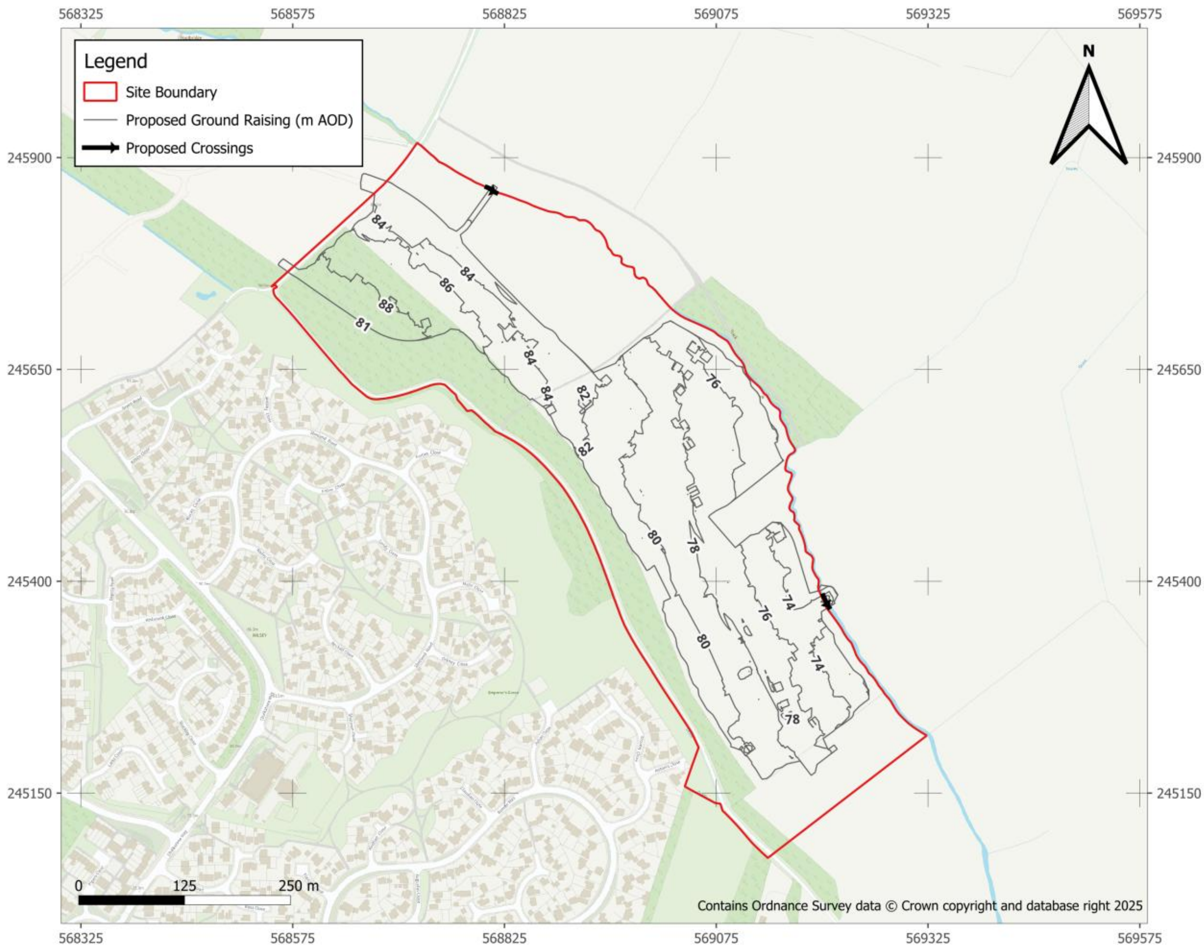


Figure 10 – Post-development scenario

The options considered and assessed for the proposed crossings are outlined below.

- Option 1: 1.05 m circular culverts
- Option 2: 1.0 m wide x 1.5 m high box culverts
- Clear span bridge with a soffit set above the 1.0% AEP + climate change event flood level, including a 300 mm freeboard.

The post-development results for Options 1 and 2 are shown and discussed in the section below. For Option 3, as the soffit of the bridge would be set 300 mm above the maximum flood level for the 1.0% AEP + climate change event, it has been assumed that the bridge deck will have no impact on flows through the ditch. Therefore, it has not been modelled.

6.2 Post-Development Results

6.2.1 Option 1

Figure 11 shows the post-development modelled results based on the incorporated ground raising and the proposed crossing points as 1.05 m circular culverts (i.e., Option 1). It indicates that flows will remain mostly contained within the channel during the design 1.0% AEP plus climate change events, with the proposed crossings in place. This means that the 1.0% AEP + climate change extent remains broadly similar in shape and coverage compared to the baseline scenario. However, the 0.1% AEP flood extent shows an increase in lateral spread, resulting in localised flooding from the proposed crossings during this exceedance event.

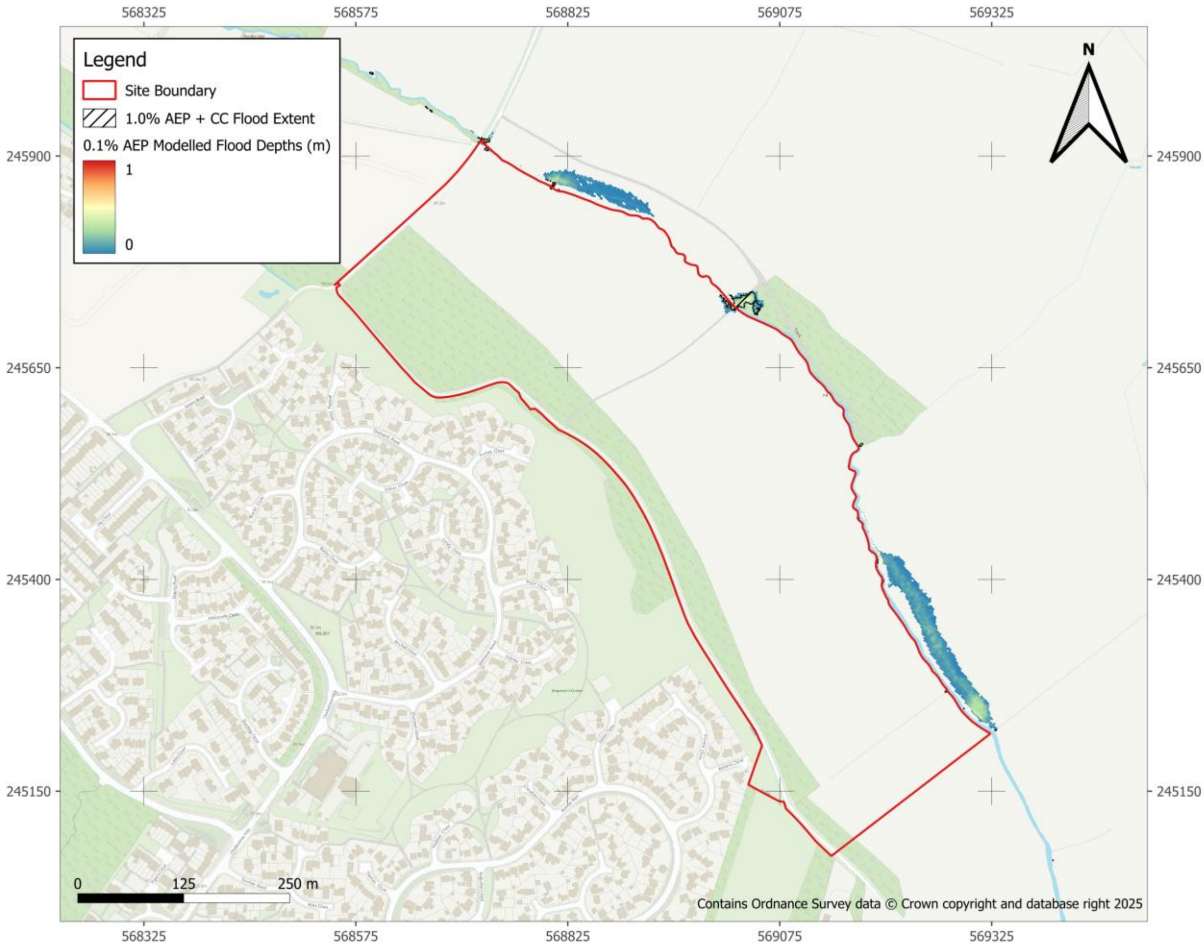


Figure 11 – Post-development model results for 1.05 m culverts (Option 1)

A depth change analysis was undertaken to assess the impacts associated with 1050 mm dia. culverts as the proposed crossings. Figure 12 shows the depth change plots for the 1.0% AEP + CC and 0.1% AEP events, respectively. For the 1.0% AEP + climate change event, the differences between the baseline and post-development scenarios are minimal along the watercourse and surrounding areas. The changes are confined to localised areas at the central existing crossing, where increases in depth reach up to approximately 0.2 m. For the 0.1% AEP event, the changes in flood depths are more significant with the proposed crossings in place, as peak flows from this extreme event exceed the capacity of the 1050 mm culverts.

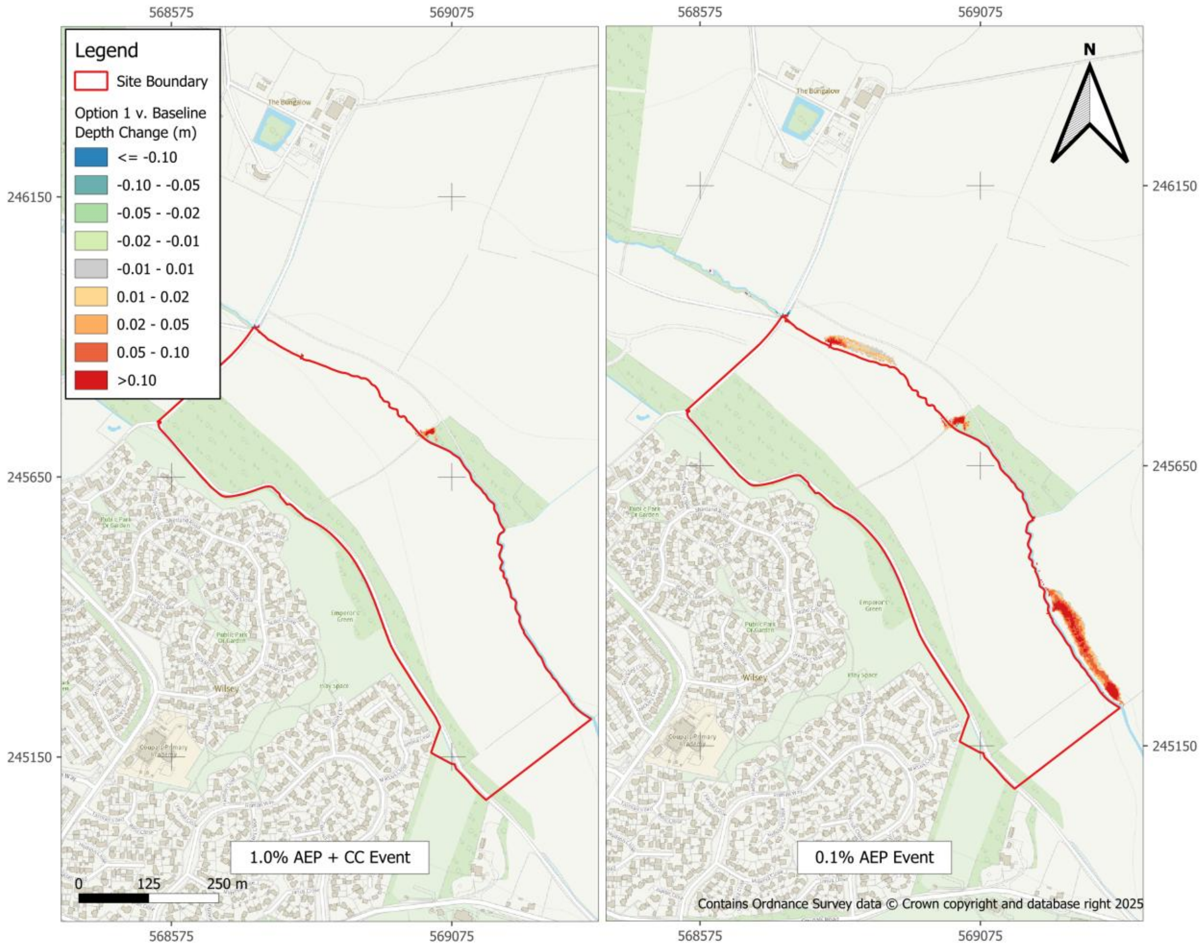


Figure 12 – Option 1 vs. baseline depth change analysis

6.2.2 Option 2

Figure 13 shows the post-development modelled results based on the incorporated ground raising and the proposed crossing points as 1.0 m width x 1.5 m high box culverts (i.e., Option 2). It indicates that flows will remain mostly contained within the channel during the design events, with the proposed crossings in place (i.e., no out-of-bank flows are shown at the proposed crossings). The 1.0% AEP + climate change extent remains broadly similar in shape and coverage compared to the baseline scenario, and the 0.1% AEP flood extent shows no significant increase in lateral spread. This is likely due to the 1 m x 1.5 m box culverts being larger than the existing ditch, therefore presenting little to no obstruction to flow.

Figure 13 – Post-development Results for 1m x 1.5m box culvert (Option 2)

A depth change analysis was undertaken to assess the impacts associated with the box culverts as the proposed crossings. Figure 14 shows the depth change plots for the 1.0% AEP + CC and 0.1% AEP events, respectively. The plots indicate that differences between the baseline and post-development scenarios are minimal along the watercourse and surrounding areas. For the 1.0% AEP + climate change event, changes are confined to localised areas near the central existing crossing, where increases in depth reach up to approximately 0.2 m. Similarly, for the 0.1% AEP event, there are increased impacts confined to a localised area at the northern proposed crossing. The variations in flood depths are highly localised and occur near hydraulic structures, likely due to adjustments in culvert representation along the watercourse.

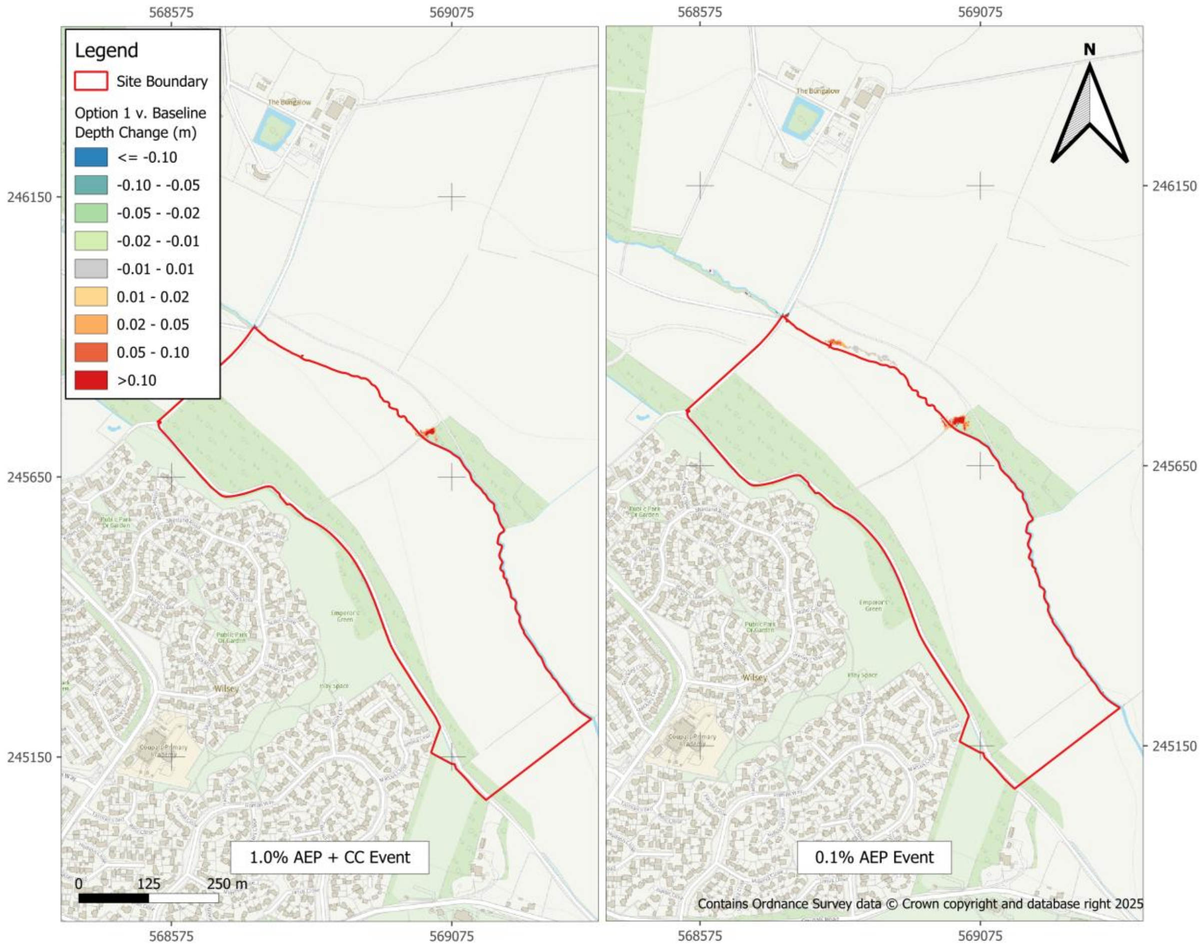


Figure 14 – Option 2 vs. baseline depth change analysis

6.3 Option evaluation and selection

Of the options considered, Option 3 (the clear span bridge) would have the least impact. However, based on engineering and cost constraints, Option 1 is the preferred alternative. Proceeding with Option 1 for the proposed crossings would require an agreement from the landowner to accept the potential impacts during the exceedance flood event identified in the study.

7 Sensitivity Analysis

Sensitivity analysis has been undertaken on the baseline 1.0% AEP climate change event using the following parameters:

- Manning's n value: Roughness coefficients for the channel, floodplains and structures have been increased and decreased by ±20%, respectively.
- Flow input: the sensitivity of the model to changes in flow is assessed by comparing modelled flood depths during the 1.0% AEP and 1.0% AEP plus climate change design events.

The results of the sensitivity analysis have been assessed at key points within the domain. The sensitivity sample points used to extract flood depths at these locations are shown in Figure 15.

Figure 15 – Sensitivity Sample Points

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

7.2 Roughness Coefficient

Table 6 show the results of the sensitivity analysis on the roughness coefficient. The sensitivity analysis indicates negligible variation in flood depths at Point 1, confirming low sensitivity to changes in Manning's n at this location.

The analysis at Point 2 shows there is a small impact under ±20% roughness adjustments, indicating some sensitivity to roughness changes. This is likely due to proximity to structures and flow constrictions.

Although there is a small impact at Point 2, the model is not considered to be unacceptably sensitive to changes in Manning's values, as the impact on modelled flood levels is below the freeboard allowance of 300mm.

Table 6 – Manning's n value sensitivity analysis

Sensitivity Points	1.0% AEP Flood Depths (m AOD)				
	Baseline Scenario (BSC)	SEN(n+): n values +20%	Difference: SEN(n+) vs. BSC	SEN(n-): n values -20%	Difference: SEN(n-) vs. BSC
1	0.237	0.240	0.003	0.236	-0.001
2	0.204	0.106	-0.098	0.324	0.120

7.3 Flow Input

Table 7 show the results of the sensitivity analysis on the peak flow inputs. The analysis indicates the modelled flood depths increase in response to higher peak inflows. These increases are within the expected range, consistent with the model's sensitivity parameters.

Table 7 – Peak flow input sensitivity analysis

Sensitivity Points	1.0% AEP Flood Depths (m AOD)	1.0% AEP + CC Flood Depths (m AOD)	Difference (m)
1	0.237	0.259	0.022
2	0.204	0.211	0.007

8 Model Stability and Limitations

8.1 2D Model Stability and Limitations

One of the main indicators of model stability with the HPC solver is the timestep selected by TuFlow. It is recommended that the timestep should not be less than one-tenth of the value used within a TuFlow Classic model. In this case, a timestep of 1.0s would be selected for a grid resolution of 2m; therefore, for a stable model the timestep should not be less than 0.1s.

To confirm the stability of the model, the evolution of the 2D timestep (dtStar) throughout the model run was reviewed to ensure that is stayed above 0.1s. 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 8 below.

Table 8 – Stability Parameters and Indicators

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 16) was reviewed, and they were found to be acceptable, as:

- dtStar is always above 0.1s, 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.
- Nc increases and stabilises at 1.0, showing all cells are stable.
- Nd values show general consistency with mild variation, indicating flow depths are evolving smoothly.

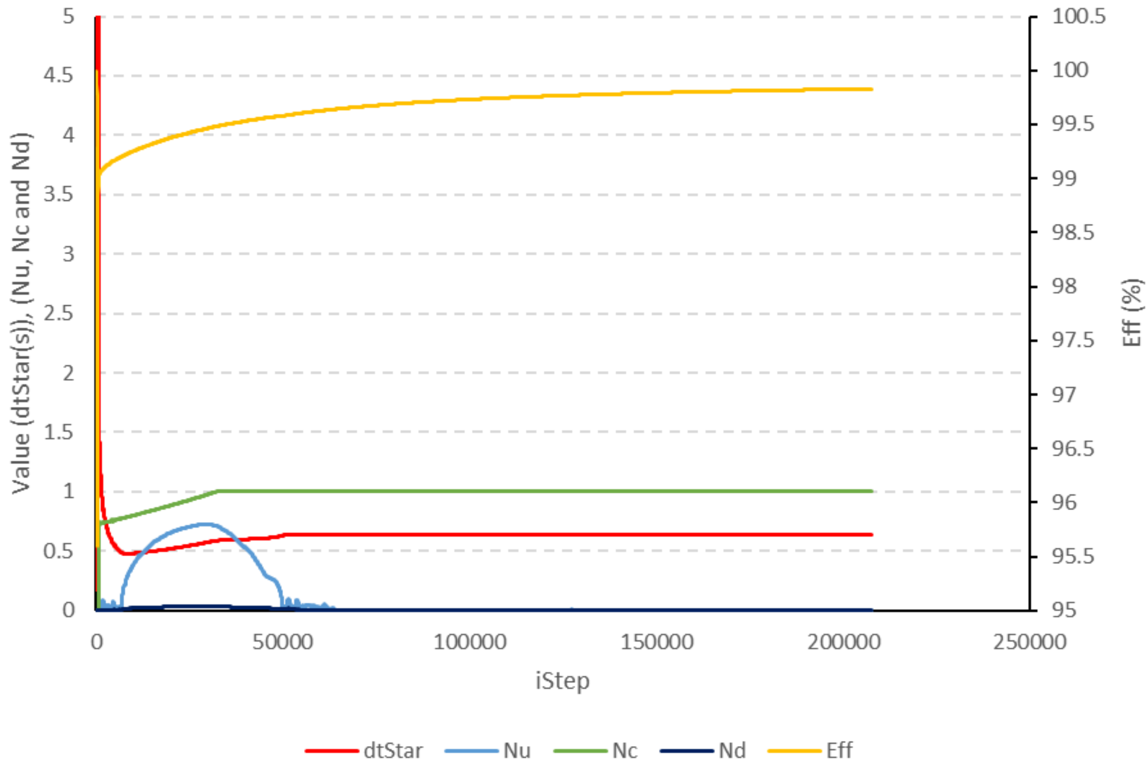


Figure 16 – Model Stability Indicators

8.2 Check 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 9. The messages have been reviewed and are considered acceptable.

Table 9 – TuFlow checks and warning messages

<table>
<tr><th>ID</th><th>Count</th><th>Description</th><th>Comment</th></tr>
<tr><td rowspan="2">Warning 1100</td><td>4</td><td>Structure 1 crest/invert is below bed of primary channel</td><td rowspan="2">Levels confirmed using survey data provided by the client.</td></tr>
<tr><td>4</td><td>Structure 2 crest/invert is below bed of primary channel</td></tr>
<tr><td>Check 1152</td><td>3</td><td>For channel Structure1, using centre cross-section and ignoring end cross-section(s).</td><td>A cross-section at the centre of Structure1 (as opposed to channel ends) takes priority, and end-section(s) are being ignored.</td></tr>
<tr><td>Check 1284</td><td>3</td><td>Connecting a 1D boundary to 2D HX link</td><td>Model configuration at downstream boundary to allow flows from the channel out of the active model area.</td></tr>
<tr><td rowspan="2">Check 2109</td><td>3</td><td>Raised HX ZC Zpt by 0.09m to 1D bed level.</td><td rowspan="2">A ‘Z’ flag has been used to adjust the cell centre (ZC) elevation at each cell at/along the 2D HX object to below the 1D node bed elevation where ZC is higher</td></tr>
<tr><td>3</td><td>Raised HX ZC Zpt by 0.00m to 1D bed level.</td></tr>
<tr><td>Warning 2550</td><td>2</td><td>5 instability timestep corrections recorded at cell [0529;0439]</td><td>The HPC solver detected instabilities at the cell, and several timestep corrections were made for the solution to remain stable.</td></tr>
<tr><td rowspan="3">Warning 2583</td><td>2</td><td>Material ID 1 has a Manning's n value (0.300) greater than Wu n limit (0.100) - n value will be limited in Wu formulation.</td><td rowspan="3">Values have been reviewed and are considered acceptable.</td></tr>
<tr><td>2</td><td>Material ID 5 has a Manning's n value (0.300) greater than Wu n limit (0.100) - n value will be limited in Wu formulation.</td></tr>
<tr><td>2</td><td>Material ID 18 has a Manning's n value (0.500) greater than Wu n limit (0.100) - n value will be limited in Wu formulation.</td></tr>
</table>

9 Conclusions

This report has detailed the methodology used to produce the hydraulic model, which informed the sizing of the proposed watercourse crossings over an existing ditch to allow access to the future development area northeast of the proposed greenfield residential development site in Wilsey Haverhill.

The report is summarised below:

- A 1D/2D ESTRY TuFlow model was produced to determine the size of the proposed crossings and to assess the likelihood of any increased third-party impact.
- The 1D network for the modelled watercourse was informed by survey data provided by the client.
- The 1D/2D hydraulic 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 model results indicated the flows remained mostly contained within the channel during the design events.
- The baseline model aligns well with the EA Flood Map for Planning for risk of flooding from rivers and the sea, indicating that it is appropriate for sizing the crossings and assessing third-party impacts.
- Post-development scenarios were established by incorporating the proposed ground model and crossing points into the baseline model. Three options were considered for the crossing points: Option 1 – 1050 mm circular culverts, Option 2 – 1 m width x 1.5 m height box culvert, and Option 3 – Clear span bridge with a soffit set above the 1.0% AEP + climate change event flood level, including a 300 mm freeboard.
- Options 1 and 2 were modelled; however, as the soffit of the bridge in Option 3 would be set 300 mm above the maximum flood level for the 1.0% AEP + climate change event, it has been assumed that the bridge deck will have no impact on flows through the ditch. Therefore, it has not been modelled.
- For Option 1, the post-development results indicate that flows will remain mostly contained within the channel during the design 1.0% AEP + CC event. However, the results indicate localised flooding around the proposed crossings due to the large flows associated with the exceedance 0.1% AEP event.
- The post-development results indicate that flows will remain mostly contained within the channel during the design events, with the proposed crossings in place (i.e., no out-of-bank flows are shown at the proposed crossings) for Option 2.
- A depth change analysis was undertaken to assess third-party impacts associated with the proposed crossings.
- The results indicate that changes are confined to localised areas near the centralised crossing, where increases in depth reach up to approximately 0.2 m for the 1.0% AEP + CC event during Options 1 and 2. It also shows increased impacts during the 0.1% AEP event for Option 1.
- A sensitivity analysis completed on the baseline model scenario for Manning's n values and changes in flow (using the climate change event) indicates that the model has negligible sensitivity to Manning's n changes upstream (close to the Structure 1) and is slightly sensitive to roughness changes at the central existing crossing, likely due to proximity to structures or flow constriction.
- Option 3 would have the least impact; however, when considering engineering and cost constraints, Option 1 is preferred. However, this would require an agreement from the landowner to accept the potential impacts during the exceedance flood event identified in this study.
- A review of the model stability and checks/warning messages indicates that the model is stable and suitable to inform the proposed crossing sizes and assess third-party impacts.

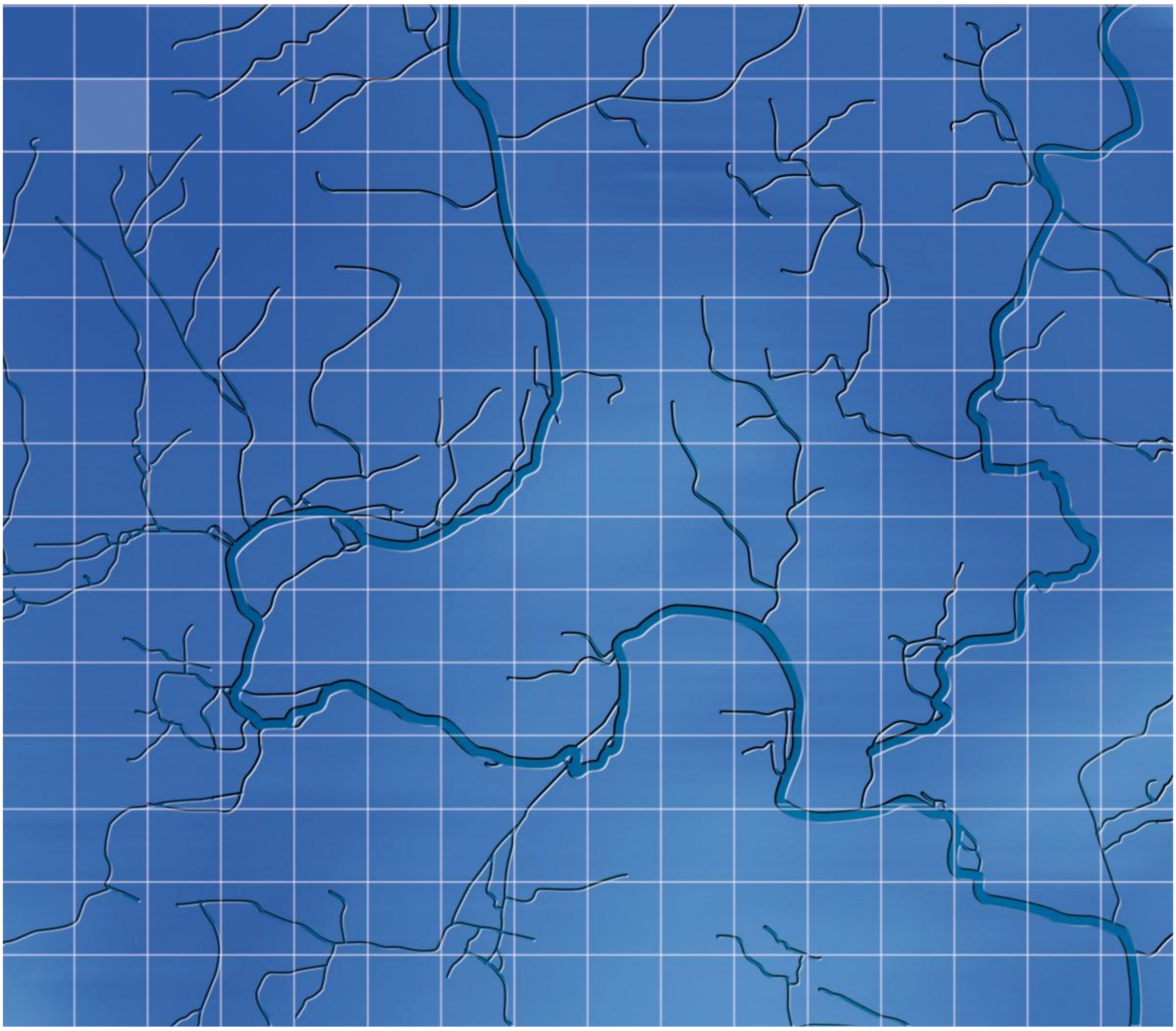
Appendix 1 – Peak Flow Assessment

Appendix 2 – Survey Data

Paul Basham

August 2025

Peak Flow Assessment for Great Wilsey Park Channel Crossings



Paul Basham

Peak Flow Assessment for Great Wilsey Park Channel Crossings Hydraulic Assessment

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WHS10203

Version	Issue date	Issue status	Prepared By	Approved By
1.0	26/08/25	Draft	Ajani Jacobs (*Consultant*)	Joel Leyshon-Jones (*Principal Consultant*) & Daniel Hamilton (*Principal Consultant*)

For and on behalf of Wallingford HydroSolutions Ltd.

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Contents

1 Requirements

The customer requires the development of design flood estimates to inform bespoke hydraulic modelling for sizing no. 2 watercourse crossings over an existing ditch. These crossings will enable access to a future development area northeast of the proposed residential development in Wilsey, Haverhill (NGR: 569086, 245529).

2 The Catchment

The catchment boundary is presented in Figure 1. The FEH Web Service[1] was used to derive the catchment descriptors. The area of the FEH Web Service boundary is 2.45 km^2. Table 1 presents the relevant catchment descriptors. The catchment has an URBEXT2000 value of 0.041, hence is classed as 'slightly urbanised' according to FEH guidance. The catchment is assigned a BFIHOST19 characteristic of 0.333 which is considered broadly representative. Whilst the majority of the catchment is underlain by chalk bedrock which is expected to be permeable, superficial deposits (till) of diamicton are present throughout the catchment along with clay, silt and gravel close to the watercourse. These are expected to limit permeability and in terms of soils, lime-rich loamy and clayey soils are present with impeded drainage. The catchment is not affected by the presence of lakes and is therefore assigned a FARL value of 1.

Table 1. Relevant catchment descriptors from the FEH Web Service[2]

Catchment Descriptor	Value
Area (km^2)	2.45
SAAR (standard average annual rainfall 1961 - 1990mm)	585 mm
BFIHOST19 (baseflow index derived from HOST soils data)	0.333
DPLBAR	1.46 km
DPSBAR	27 m/km
FARL (index of flood attenuation due to reservoirs and lakes)	1
FPEXT (extent of flood plain)	0.0378
PROPWET	0.26
URBEXT 2000	0.0408

[1] https://fehweb.ceh.ac.uk/GB/map
[2] *Results based upon FEH methodology and data, CEH (2015) 'CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, Oxon, UK'*

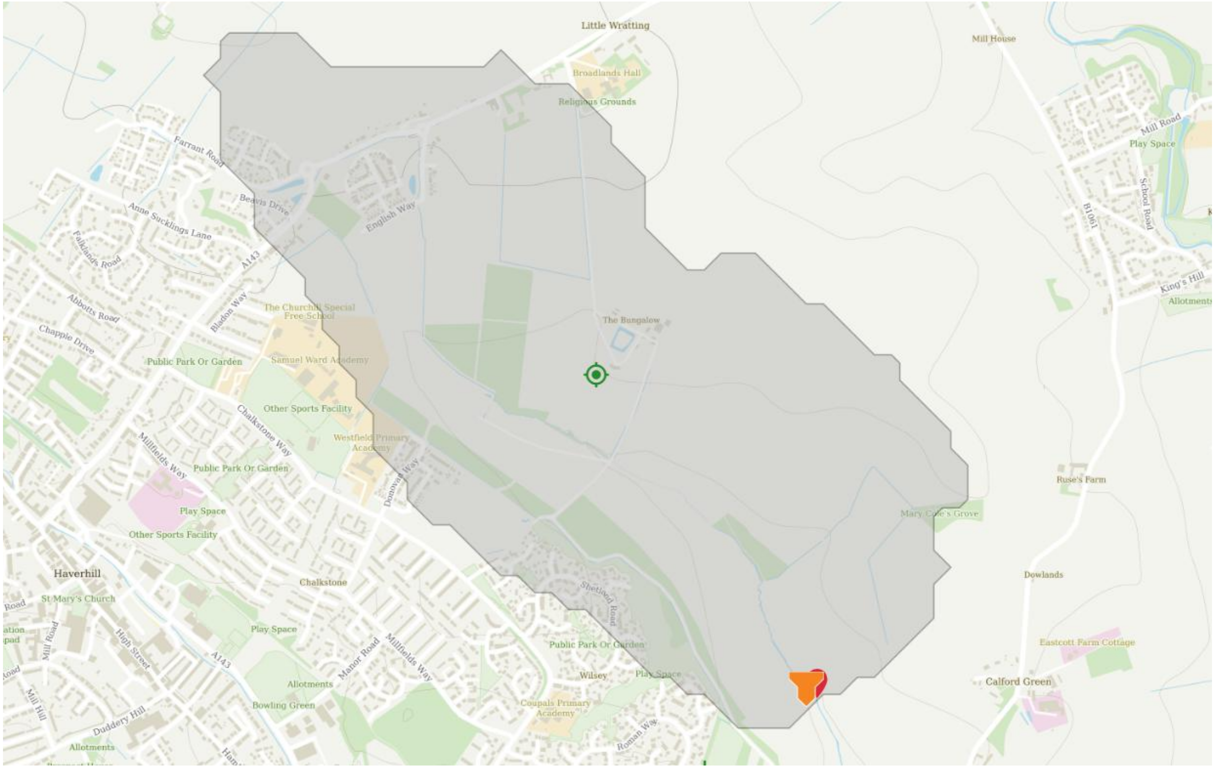

Figure 1 The FEH Web Service derived catchment is shown by the grey boundary. *Contains OS data © Crown Copyright (2023) Contains CEH data © and database right NERC (CEH) 2023*

A potential field drain sub-catchment with a different outlet from the main watercourse, was identified within the western boundary of the FEH catchment. However, due to the interconnectivity of the field drains and overland flow routes between the two catchments, it was not possible to delineate the full length of this sub catchment without detailed survey data. As uncertainty exists, this sub-catchment has been conservatively included within the FEH catchment boundary to provide a precautionary approach with regard to flood risk.

3 Outline of Methodology

The flood estimates have been developed using the Flood Estimation Handbook statistical and rainfall runoff methods. The statistical methods are those as published by the Institute of Hydrology in 1999[3] with updates included in the latest version of WINFAP-FEH 5[4] as described by Kjeldsen *et al.,*[5] and the WHS technical guidance[6]. These methods require the estimation of a normalised flood frequency curve, termed the flood growth curve and the estimation of the normalising variable; the median annual flood, QMED. The current version of NRFA Peak Flows dataset available for use in this study was NRFA Peak Flows v14.0[7].

The rainfall-runoff methods are those first published by Kjeldsen[8], which were subsequently updated in 2015 and 2019 and implemented within the ReFH2.3 software[9] as described in the WHS technical guidance[10]. The latest FEH22 rainfall model[11] has been used in the derivation of rainfall inputs for the catchment.

4 Peak Flow Estimation using the statistical method

As the site is ungauged, the approach adopted for estimating QMED has been to develop an FEH catchment descriptor-based estimate and to review the availability of potentially suitable donor catchments to form the basis of a data transfer exercise that would improve the QMED estimate. This is the standard application of the FEH methodology. The pooled methodology for estimation of the growth curve is then applied.

4.1 Derivation of the Median Annual Flood

The QMED for the location was first estimated from catchment descriptors ($QMED_{cds}$) as 0.515 m^3s^{-1}.

Estimates of QMED from observed data ($QMED_{obs}$) at donor stations can be used to adjust the estimate of $QMED_{cds}$ at the subject site. Possible donor catchments are initially sought on the basis of being geographically close. Along with geographical distance, the similarity of catchment descriptors; Area, SAAR, FARL and BFIHOST19 with the target catchment are also considered. By default, 6 stations are selected for donor transfer.

The general pattern of $QMED_{cds}$ to the $QMED_{obs}$ for the six closest stations is mixed, with the catchment descriptor equation overestimating the $QMED_{obs}$ for three of the six stations and underestimating it for the remaining three stations.

A station 1km downstream of the target catchment outlet, the Stour Brook @ Sturmer (36011) was not originally included within the donor group on the basis of having a high URBEXT2000 value (0.100). Given that this catchment is otherwise extremely similar to the target catchment, and that

[3] Robson, A. and Reed, D., 1999. *Flood Estimation Handbook Volume 3: Statistical Procedures for Flood Frequency Estimation.* Institute of Hydrology, Wallingford, pp338.
[4] https://www.hydrosolutions.co.uk/software/winfap-5/
[5] Kjeldsen, T.R., Jones, D.A., and Bayliss, A.C., 2008. *Improving the FEH statistical procedures for flood frequency estimation.* Environment Agency, Bristol, pp137.
[6] https://www.hydrosolutions.co.uk/software/winfap-4/literature/
[7] https://nrfa.ceh.ac.uk/peak-flow-dataset
[8] Kjeldsen, T. R. 2007. The revitalised FSR/FEJ rainfall-runoff method. Supplementary Report No.1. CEH.
[9] https://www.hydrosolutions.co.uk/software/refh-2/
[10] https://www.hydrosolutions.co.uk/software/refh-2/supporting_literature/
[11] https://fehwebdocs.hydrosolutions.co.uk/DDF-Science/FEH22/

the target catchment itself is slightly urbanised (0.041), this was considered to be justification for raising the URBEXT2000 threshold to include this station. This station shows the catchment descriptor equation to underestimate $QMED_{obs}$. Whilst other stations in the donor group are potentially suitable for donor transfer, they are considered surplus to requirements given that 36011-Stour Brook at Sturmer lies in the same catchment, is located just downstream and has a long record length (54-years).

In this regard, the Stour Brook @ Sturmer (36011) was selected as the sole donor site. Table 2 presents the geographically closest ten stations.

When donor adjustment is applied the final rural QMED value is estimated as 0.599 m^3s^{-1}. The catchment has an URBEXT value of 0.041 with an associated urban adjustment factor of 1.036. Therefore, the QMED value when urbanisation is accounted for is 0.620 m^3s^{-1}. This value is used to scale the flood growth curves.

Table 2. List of donor catchments considered for QMED adjustment[12]

Station	Distance (km)	AREA	SAAR	FARL	URBEXT2000	BFIHOST19	Accept or Reject	Comments
FEH_Catchment_Descriptors_		2.45	585	1	0.041	0.333	Target Catchment	
36008 (Stour @ Westmill)	1.69	222.82	589	0.994	0.023	0.397	Reject	Significantly Larger Catchment
36011 (Stour Brook @ Sturme	3.15	34.242	592	0.999	0.1	0.363	Accept	Suitable catchment
36012 (Stour @ Kedington)	5.53	76.642	599	0.99	0.01	0.38	Reject	Suitable catchment but surplus to requirements
36010 (Bumpstead Brook @ B	5.69	27.547	588	0.999	0.007	0.367	Reject	Suitable catchment but surplus to requirements
36015 (Stour @ Lamarsh)	8.25	481.29	583	0.987	0.021	0.47	Reject	Significantly Larger Catchment
37012 (Colne @ Poolstreet)	9.38	64.49	574	0.992	0.009	0.384	Reject	Excluded as closer stations to the target catchment exist
36006 (Stour @ Langham)	10.93	571.362	580	0.985	0.019	0.505	Reject	Significantly Larger Catchment
33055 (Granta @ Babraham)	11.04	101.972	579	0.999	0.012	0.709	Reject	BFIHOST and Significantly Larger Catchment
37016 (Pant @ Copford Hall)	12.26	63.8	588	0.997	0.009	0.385	Reject	Excluded as closer stations to the target catchment exist
36002 (Glem @ Glemsford)	12.26	85.627	598	0.982	0.008	0.385	Reject	Excluded as closer stations to the target catchment exist

[12] *Results based upon FEH methodology and data, CEH (2015) 'CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, Oxon, UK'*

4.2 Derivation of the Growth Curve

Within the FEH methodology, flood growth curves for ungauged sites are formed by pooling annual maxima data from similar catchments, which are flagged as being suitable for pooling. A threshold of 500 station-years is required (a sum of record lengths). The default URBEXT threshold of 0.03 was raised to 0.100 to include the Stour Brook @ Sturmer (36011) which as mentioned lies downstream of the target site and is suitable for pooling. Thus an initial pooling group was formed for the development of the flood growth curve by pooling data from 12 catchments with a total of 513 station years. Note, the URBEXT

Three stations were removed due to being permeable catchments, these were Brompton Beck @ Snainton Ings (27073), Gypsey Race @ Kirby Grindalythe (26016), Heighington Beck @ Heighington (30013) and Water Forlornes @ Driffield (26014).

To ensure the pooling group met the threshold of 500-station years, the Hodge Brook @ Bransdale Weir (27010), the Bolingey Stream @ Bolingey Cocks Bridge (49005), Haddeo at Upton (45816), Black Burn @ Pluscarden Abbey (7011), Gogar Burn @ Turnhouse (19017) and Brox Burn @ Newliston (19014) were substituted into the pooling group.

A final pooling group containing 14 stations and 526 station years was derived. Table 3 shows the full list of pooling group members along with the reason for any stations being removed or retained. The distance shown is the distance from each candidate station to the sites in a similarity distance space (the FEH distance measure).

Table 3. Pooling group selection and reasons for retaining or removing from final pooling group.[13]

Station	Distance SDM	AREA (km^2)	SAAR (mm)	FARL	URBEXT 2000	Accept or Reject	Comments
27073 (Brompton Beck @ Snainton Ings)	1.116	8.600	721	1.000	0.008	**Reject**	Mainly permeable limestone catchment
23018 (Ouse Burn @ Woolsington)	1.189	10.137	670	0.977	0.1	**Accept**	
27051 (Crimple @ Burn Bridge)	1.446	8.172	855	1.000	0.006	**Accept**	
26016 (Gypsey Race @ Kirby Grindalythe)	1.651	15.850	757	1.000	0	**Reject**	Groundwater catchment. Daily flows are ephemeral and blocky.
25019 (Leven @ Easby)	1.753	15.880	830	1.000	0.004	**Accept**	
30013 (Heighington Beck @ Heighington)	1.804	23.88	605	0.962	0.079	**Reject**	Slow Responding Limestone catchment.
76011 (Coal Burn @ Coalburn)	1.828	1.630	1096	1.000	0	**Accept**	
36010 (Bumpstead Brook @ Broad Green)	1.914	27.547	588	0.999	0.007	**Accept**	More than 15% non-flood yrs
36011 (Stour Brook @ Sturmer)	2.087	34.242	592	0.999	0.1	**Accept**	
26014 (Water Forlornes @ Driffield)	2.129	32.415	721	1.000	0.007	**Reject**	Geology is chalk overlain by drift and gravels, ephemeral and largely non-responsive.
7009 (Mosset Burn @ Wardend Bridge)	2.138	28.295	803	0.998	0	**Accept**	
38020 (Cobbins Brook @ Sewardstone Road)	2.19	38.785	616	0.997	0.051	**Accept**	More than 15% non-flood yrs
27010 (Hodge Beck @ Bransdale Weir)	2.202	18.820	987	1.000	0.001	**Accept**	Substituted
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	2.229	16.800	1044	0.991	0.006	**Accept**	Substituted
45816 (Haddeo @ Upton)	2.234	6.808	1210	1.000	0.005	**Accept**	Substituted
7011 (Black Burn @ Pluscarden Abbey)	2.326	36.375	808	0.98	0.001	**Accept**	Substituted
19017 (Gogar Burn @ Turnhouse)	2.334	40.307	756	0.99	0.113	**Accept**	Substituted
19014 (Brox Burn @ Newliston)	2.371	37.330	826	0.989	0.115	**Accept**	Substituted

[13] *Results based upon FEH methodology and data, CEH (2015) 'CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, Oxon, UK'*

4.3 Results

A three-parameter generalised logistic (GL) distribution was used, it was preferred over the Kappa distribution which showed a slightly better fit. This was because the GL distribution allows for non-flood year adjustment and still shows a very good fit. A non-flood year adjustment is required given that two of the stations in the pooling group have more than 15% non-flood years. Figure 2 shows the estimated growth curve for the subject site and Table 4 presents the flood growth curve indexed by return period. The growth curve was also adjusted for urbanisation.

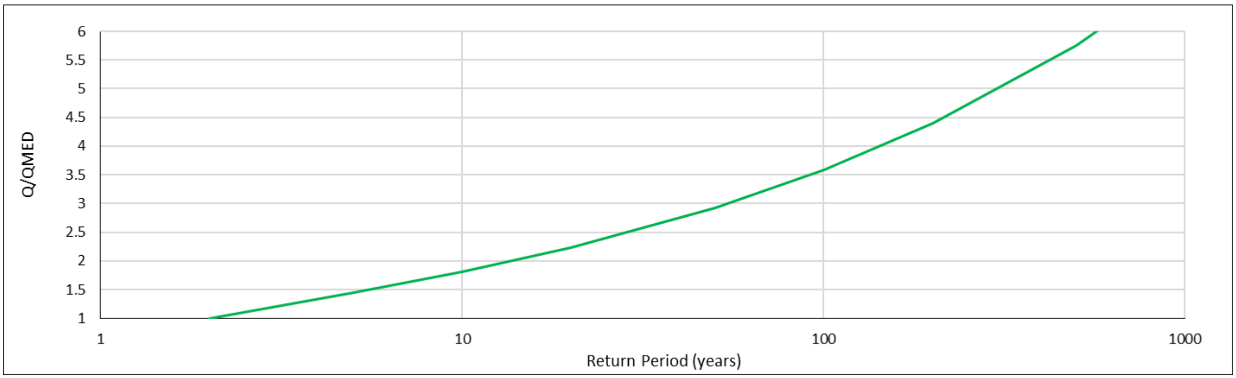

Figure 2 Growth curve

Table 4. Growth curve[14]

Return Period (years)	Growth Curve for Location
2	1.000
25	2.383
50	2.923
100	3.583
500	5.758
1000	7.073

The final peak flow is presented in Table 5. These represent the QMED value (2-year return period) scaled by the adjusted growth curve.

[14] *Results based upon FEH methodology and data, CEH (2015) 'CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, Oxon, UK'*

Table 5 Peak Flow Estimates as Returned by WINFAP 5.2 Software

Return Period (years)	Peak Flow estimate (m^3/s)
2	0.620
25	1.477
50	1.812
100	2.221
500	3.570
1000	4.385

5 Peak Flows Estimation using the Rainfall- Runoff methodology

The catchment was modelled using the ReFH 2.3 software. This uses standard design rainfall events and catchment descriptors to produce hydrographs for the site. The recommended duration and timestep of 6.5 hours and 0.5 hours, respectively, were used to define the rainfall event. Default parameters for urbanisation were used, and as the catchment is slightly urbanised, the final peak flows were sensitive to these.

Table 6 Peak Flow Estimates as Returned by ReFH 2 Software

Return Period (years)	Peak Flow estimate (m^3/s)
2	0.921
25	1.890
50	2.150
100	2.433
500	3.401
1000	4.210

6 Final Hydrology

The flood peaks estimated using the rainfall runoff methodology and the statistical method are closely matched. The final flood peak estimates have been taken from the results of the rainfall runoff method as the watercourse crossings are being designed for the 1.0% AEP plus climate change event, and the rainfall runoff method provides the highest peak flow for the 1.0% AEP event. The final flood peaks are presented in Table 7.

Table 7 Final Peak Flows Estimates[15].

Return Period (years)	Peak Flow estimate (m^3/s)
2	0.921
25	1.890
50	2.150
100	2.433
500	3.401
1000	4.210

[15] *Results based upon FEH methodology and data, CEH (2015) 'CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, Oxon, UK'*

Appendix 2 – Survey Data

Photos

Culvert 1 – Photo 1

Culvert 1 – Photo 2

Culvert 1 – Photo 3

Culvert 1 – Photo 4

Culvert 1 – Photo 5

Culvert 1 – Photo 6

Culvert 1 – Photo 7

Culvert 1 – Photo 8

Culvert 1 – Photo 9

Culvert 2 – Photo 1

Culvert 2 – Photo 2

Culvert 2 – Photo 3

Culvert 2 – Photo 4

Culvert 2 – Photo 5

Culvert 2 – Photo 6

Culvert 2 – Photo 7

Footbridge (Culvert) 3 – Photo 1

Footbridge (Culvert) 3 – Photo 2