A12 Chelmsford to A120 widening scheme TR010060

6.3 ENVIRONMENTAL STATEMENT

APPENDIX 14.5 FLOOD RISK ASSESSMENT – ANNEX L HYDRAULIC MODELLING REPORTS

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

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A12 Chelmsford to A120 widening scheme

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ENVIRONMENTAL STATEMENT

APPENDIX 14.5 FLOOD RISK ASSESSMENT – ANNEX L HYDRAULIC MODELLING REPORTS

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6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT BOREHAM BROOK MODELLING ANNEX

APFP Regulation 5(2)(e)

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0 Executive summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at Boreham Brook. The modelling and hydrology have been carried out in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1D only type using Flood Modeller software (Jacobs 2022) and was built from scratch.

0.1.2 Three events were simulated: the 5%, 1% and 1% AEP with 38% climate change. Flow reconciliation was carried out to match the design hydrology. No data was available for calibration, but the model was verified using Environment Agency (EA) flood maps. The model was then used to simulate baseline and with-scheme scenarios for the three events mentioned. The with-scheme scenario involved changes to river cross sections where the new highway embankment interfaces with the modelled flood outline. Sensitivity analysis of roughness, inflows and the downstream boundaries was carried out to further validate the results. The analysis shows that the proposed scheme has no significant impact on flooding at Boreham Brook and therefore no mitigation is proposed.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed scheme includes widening the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) is required for the proposed scheme to meet relevant local and national planning legislation and inform the design and planning process. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic models with associated catchment hydrology. Water levels for the baseline and with scheme scenarios both upstream and downstream of the proposed scheme and the associated flood extents were determined for a range of storm flood events at each watercourse crossing.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA are:

- Boreham Brook (this report)
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Five minor watercourses were also modelled for the proposed scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for Boreham Brook, to assess the baseline and “with scheme” scenarios. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 For the A12 crossing of Boreham Brook, the highway will be upgraded to three lanes per carriageway. This will require widening of the highway embankment on its south side. Changes include modifying the river section profiles near the A12 by superimposing the new embankment.

1.1.6 This report should be read in conjunction with the Flood Risk Assessment report (Jacobs 2022).

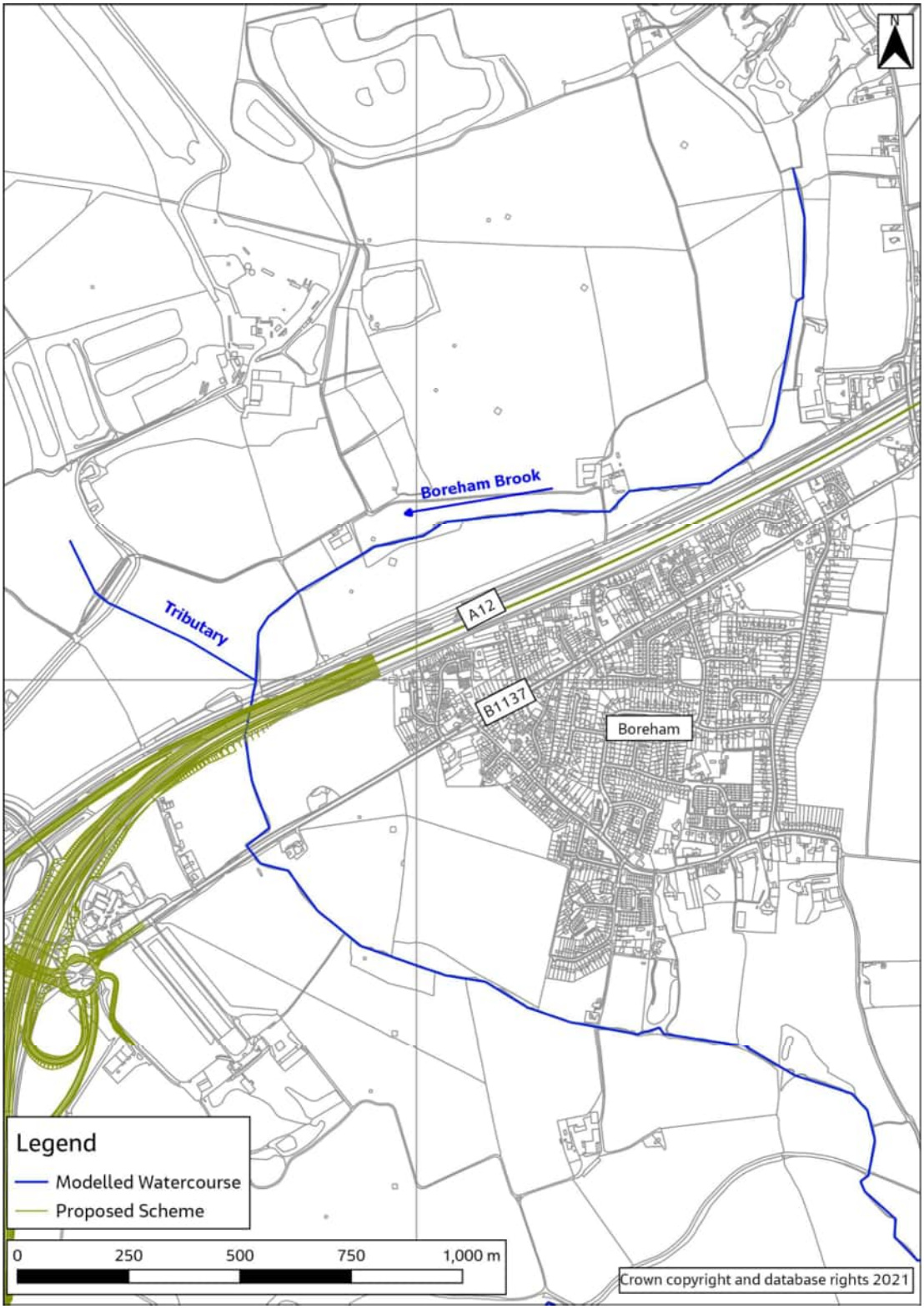
1.2 Methodology

1.2.1 The model was built using a one-dimensional (1D) schematisation, using the Flood Modeller Pro version 4.6 (Jacobs 2020) river modelling package. 1D representation of the floodplain using extended cross-sections was considered appropriate because the high ground levels on both sides of Boreham Brook mean that the predominant flow path from upstream to downstream is maintained for all flows that are considered. In addition, the EA flood maps (Plate 6.3) show the flood extent is aligned to the river channel.

1.3 Study area

1.3.1 Boreham Brook flows to the west of Boreham village in Essex, flowing from north to south under the A12 (Plate 1.1). To the north of the A12 there is a tributary, which joins Boreham Brook from the west. The model domain includes 2km of Boreham Brook upstream of the A12 and 1.3km downstream. In addition, 400m of the tributary is included in the model.

Plate 1.1 Boreham Brook study area



2 Input data

2.1.1 The data used to construct the hydraulic model for Boreham Brook is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR	2m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection and Ranging) data. Used to extend cross sections into the floodplain.	Environment Agency 2017
OS maps	Mastermap data	Highways England
Channel survey	In-channel cross sections and hydraulic structures	Jacobs August 2017
Hydrological analysis	Hydrological analysis carried out for Boreham Brook	Jacobs 2017 and update for climate change (2021)

3 Hydrology

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of the Boreham Brook for the 5% and 1% Annual Exceedance Probability (AEP) events (equivalent to the 20 and 100-year return periods). The 1% AEP event is also required including an allowance for climate change (plus CC) (Environment Agency 2021). For full details of the flow derivation methods and background hydrology refer to the Boreham Brook Flood Estimation Calculation Record (Annex A).

3.2 Hydrological considerations

3.2.1 Boreham Brook catchment (14.7km^2) is a lowland catchment located in the south east of England. The catchment is classified as essentially rural with small urban areas around Boreham. The Boreham Brook has numerous sand and gravel pits and waterbodies / pools which are assumed to be associated with the sand and gravel pits. The majority of these waterbodies have been assessed as being offline and therefore are not considered to unduly impact design peak flow estimates. The FEH Web Service records a FARL of 0.962 for the Boreham Brook catchment to the model extent. Some attenuation due to reservoirs / waterbodies is therefore likely to be occurring but the impact of such attenuation is not considered to have a major influence on the design peak flow estimates. The soils within the catchment are predominantly of the chalky till variety with glaciofluvial drift around the watercourse. The soils are noted as having the potential for slow permeability with some areas of clayey subsoils (Soil Survey of England and Wales 1983). The catchment BFIHOST (0.529) and SPRHOST (41.77) values are in the normal range and do not indicate a high permeability across the catchment.

3.3 Peak flow derivation

3.3.1 The Boreham catchment is classified as small (<25km^2). For such catchments where uncertainties may be large it is considered sensible to compare peak flows derived from two independent methods. Therefore, both the FEH Statistical and ReFH2.2 methods (ReFH2.2 was the available ReFH software at the time of the original 2017 hydrology assessment) were applied to estimate the design return period peak flows. More details can be found in Annex A.

3.3.2 The ReFH2.2 resulted in higher design peak flows for the Boreham Brook than the FEH Statistical method. Although analysis of a nearby gauge on the River Ter (station number 37003) indicated that the ReFH2.2 method appears to generally overestimate design peak flows for that gauge it was assessed as appropriately precautionary to average the two design peak flow estimates (ReFH2.2 and FEH Statistical) to generate the final design peak flows.

3.3.3 For hydraulic modelling of the Boreham Brook the final design peak flows estimated at FEP01 and FEP02 (refer to FEP1_BB and FEP2_BB

respectively in Plate 3.1) have been scaled by catchment area and applied at the top of the modelled tributaries. The residual flows have been applied as lateral inflows into the model between the top of the tributaries and FEP01 and FEP02, that is, confluence of the two tributaries. For the residual catchment south of the confluence (that is, south of FEP01 / FEP02), the derived flows have been applied as lateral inflows into the hydraulic model.

3.3.4 The flood flows at the three locations FEP01, FEP02 and FEP03 (for the lumped catchment up to those points) are presented in Table 3.1.

Table 3.1 Boreham Brook peak flows (average FEH Statistical and ReFH2.2) as a function of event rarity

Inflows	Peak Flows (m^3/s)										
AEP	50%	20%	10%	5%	3.33 %	2%	1.33%	1%	1% + 38% CC	0.5 %	0.1%
FEP01	0.74	1.03	1.23	1.43	1.56	1.75	1.92	2.06	2.84	2.51	3.93
FEP02	0.82	1.14	1.37	1.60	1.75	1.96	2.16	2.32	3.20	2.82	4.39
FEP03*	1.50	2.07	2.47	2.89	3.15	3.52	3.86	4.14	5.72	5.04	7.83

*The AEP flows at FEP03 are used to reconcile the modelled flow at the model downstream extent.

3.4 Inflow hydrograph shape

3.4.1 Given that the Boreham Brook catchment is ungauged it was assessed as appropriate to derive hydrograph shapes for input into the hydraulic model using ReFH2.2. The ReFH2.2 hydrograph shapes were derived at FEP01/FEP02/FEP03 and for the residual catchment and are all based on an 11.1 hour event storm duration, which is the critical storm duration at the downstream modelling extent (FEP03).

3.5 Inflows into the Boreham Brook hydraulic model

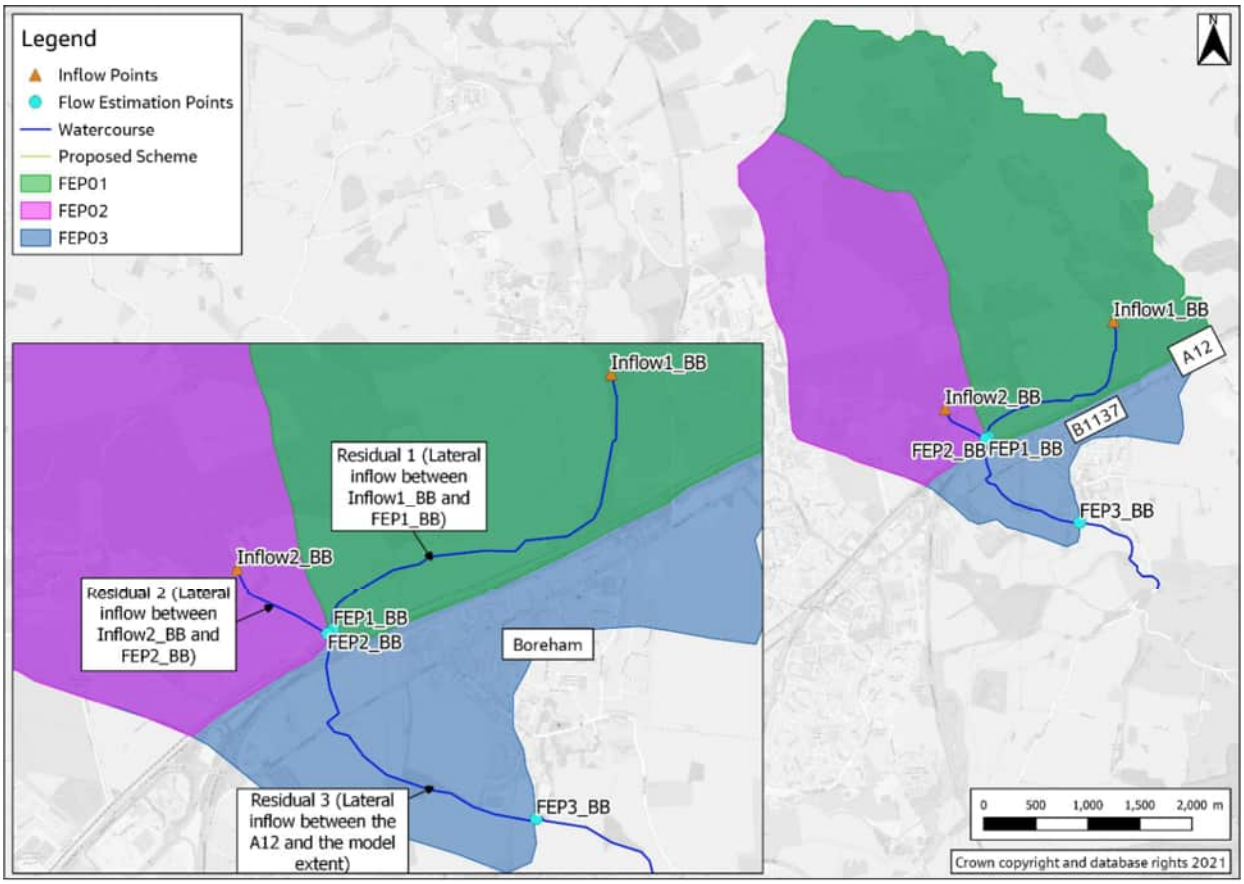
3.5.1 The Boreham Brook hydraulic model requires design hydrographs at five locations as described below:

- Inflow 1 – at the upstream extent of the Brick House Farm tributary;
- Inflow 2 – at the upstream extent of the Bulls Lodge tributary;
- Residual 1 – lateral inflow from the residual catchment between Inflow 1 and FEP01;
- Residual 2 – lateral inflow from the residual catchment between Inflow 2 and FEP02;

- Residual 3 – lateral inflow from the residual catchment between the A12 and the model extent.

3.5.2 A map showing the Boreham Brook hydraulic model extent and inflow locations is provided in Plate 3.1.

Plate 3.1 Model extent and inflow locations

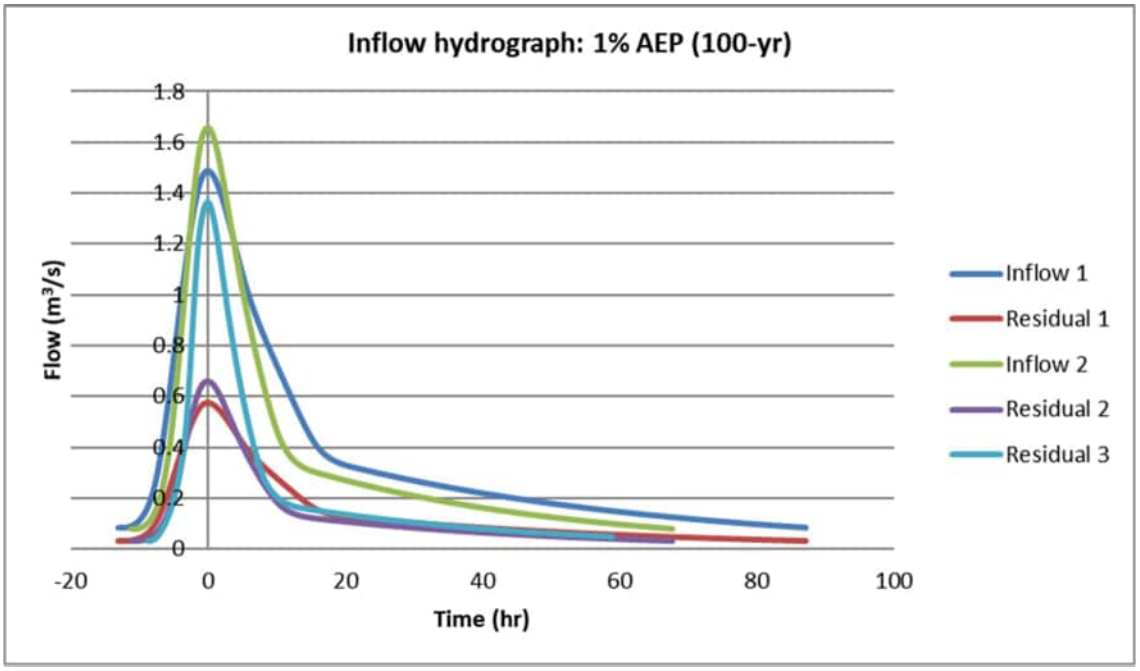


3.5.3 The inflow hydrographs were produced by scaling the derived hydrograph shapes (as described above) to the estimated peak flows given in Table 3.2. Plate 3.2 presents the inflow hydrographs used for the 1% AEP (100-year return period) event.

Table 3.2 Inflow peaks as a function of event rarity

Inflows	Peak Flows (m^3/s)										
AEP	**50%**	**20%**	**10%**	**5%**	**3.33%**	**2%**	**1.33%**	**1%**	**1% + 38% CC**	**0.5%**	**0.1%**
Inflow 1 (BBR01_3466)	0.54	0.74	0.88	1.03	1.13	1.26	1.38	1.49	2.05	1.81	2.83
Inflow 2 (BBR02_0531)	0.59	0.82	0.98	1.15	1.25	1.40	1.54	1.66	2.29	2.02	3.14
Residual 1 (Lateral_BB)	0.21	0.29	0.34	0.40	0.44	0.49	0.54	0.57	0.79	0.70	1.10
Residual 2 (Lateral_Trib)	0.23	0.33	0.39	0.46	0.50	0.56	0.61	0.66	0.91	0.81	1.25
Residual 3 (Lateral_AC)	0.46	0.66	0.80	0.94	1.03	1.16	1.27	1.37	1.88	1.65	2.66

Plate 3.2 1% AEP (100-year return period) inflow hydrographs



4 Baseline model build

4.1.1 Surveyed cross section data has been used to inform the in-channel geometry of the modelled watercourses. The locations of the surveyed cross-sections are shown in Plate 4.1.

4.1.2 To aid model performance on all modelled watercourses, interpolated cross sections were added between the surveyed cross sections. The spacing of the interpolated cross sections ranged from 30m to 90m.

4.1.3 The floodplain has been represented in the model using extended cross sections. Selected cross sections have been extended using LiDAR data.

4.1.4 Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using the photographs taken during the survey. Generally, gravel can be seen in the bed of Boreham Brook and the tributary, with long grass and heavy vegetation on the floodplain. The Boreham Brook channel itself is vegetated over at some locations. The Manning's 'n' coefficients used in the model are shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow 1959).

Plate 4.1 Boreham Brook baseline model schematisation

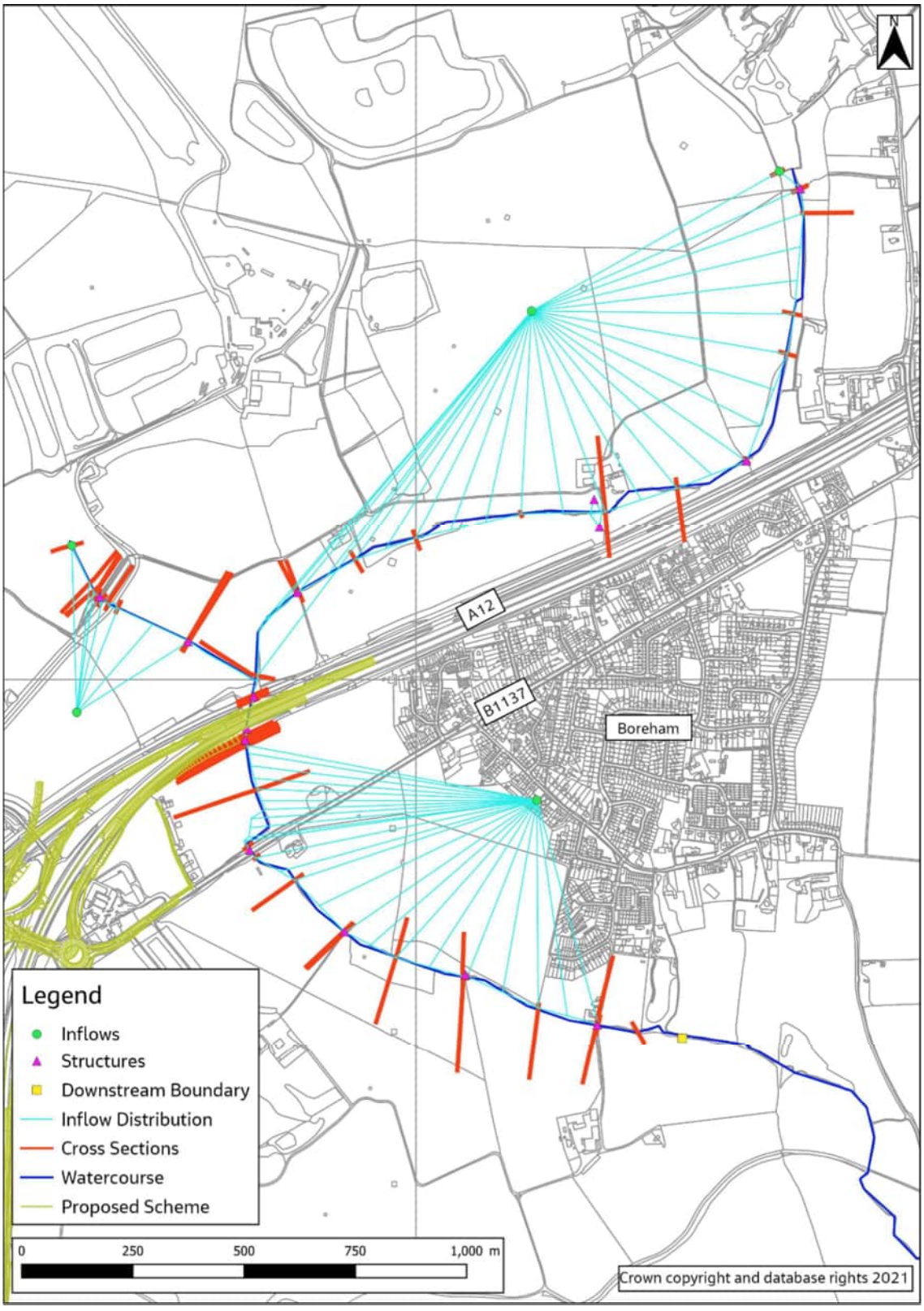


Table 4.1 Manning's 'n' coefficients – 1D domain

Flood Modellers Nodes	Watercourse	Bed Manning's 'n'	Bed Material	Floodplain Manning's 'n'	Floodplain Material
BBR01_3466 to BBR01_1429d	Boreham Brook	0.040	Gravel	0.055	Long grass
				0.06	Light brush and trees
BBR01_1301 to BBR01_0000				0.1	Heavy vegetation and trees
BBR01_1392d to BBR01_1380		0.055	Vegetation		
BBR01_1380		0.055	Vegetation		
BBR02_0531 to BBR02_0173d	Tributary	0.040	Gravel		

4.1.5 Fourteen hydraulic structures were included in the model. Their locations are shown in Plate 4.1. A structure schedule is presented in Annex B.

4.1.6 The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4.2. Inflow locations are shown in Plate 4.1.

Table 4.2 Boundary conditions – 1D domain

Type of Boundary	Flood Modellers Node	Description
QT Boundary	BBR01_3466	ReFH2.2 hydrograph. Applied at the upstream end of Boreham Brook
QT Boundary	Lateral_BB	ReFH2.2 hydrograph. Applied laterally between the upstream end of Boreham Brook and the tributary
QT Boundary	Lateral_AC	ReFH2.2 hydrograph. Applied laterally from downstream of the A12 and the downstream end of the model on Boreham Brook
QT Boundary	BBR02_0531	ReFH2.2 hydrograph. Applied at the upstream end of the Tributary
QT Boundary	Lateral_Trib	ReFH2.2 hydrograph. Applied laterally from the upstream to downstream on the tributary
Normal depth downstream boundary	BBR01_0000	Normal depth boundary condition applied to the downstream end of Boreham Brook. Sensitivity testing has shown that this boundary assumption does not affect model results in the area of interest (see Section 6.4 Sensitivity Analysis)

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 38% uplift is in accordance with guidance (Environment Agency 2021) for climate change uplifts for this area. For further details of the climate change uplift, see the Flood Risk Assessment (Environment Agency 2021).

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope (see Section 6.4).

Table 5.1 Modelled events

Scenario	AEP Event		
	5%	1%	1%+38%CC
Baseline	X	X	X
With Scheme	X	X	X
Roughness Sensitivity (1D)		X	
Hydrological Inflow Sensitivity		X	
Downstream Boundary Sensitivity (1D)		X	

6 Model proving

6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution, within a specified number of iterations, for each model time step. The convergence of the 1D model was checked, and as shown in Plate 6.1 and Plate 6.2 below for the baseline and with scheme scenarios, there are two incidences of non-convergence evident. However, this has been identified as due to mode switching for an orifice unit situated at the downstream end of the model and is therefore deemed of no consequence for the present study and the model performance is acceptable. This convergence plot is typical for all events modelled. The 1D model mass error has also been checked and is within the recommended tolerance of 1% for all simulations carried out.

Plate 6.1 1D model convergence for baseline scenario – 1% AEP + 38%CC event

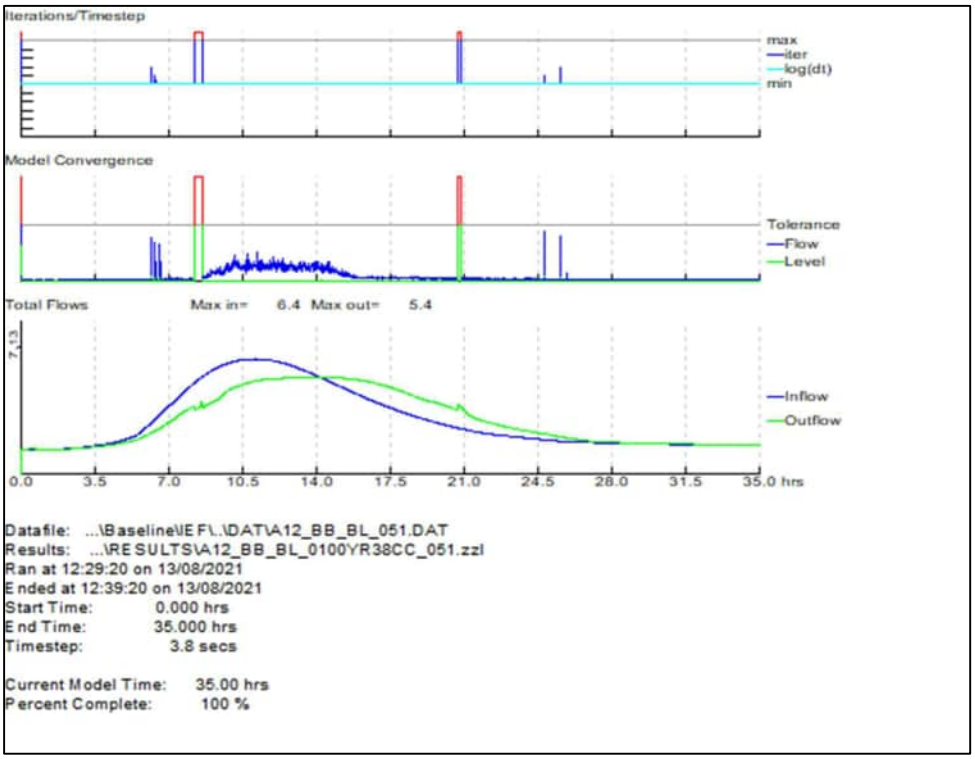
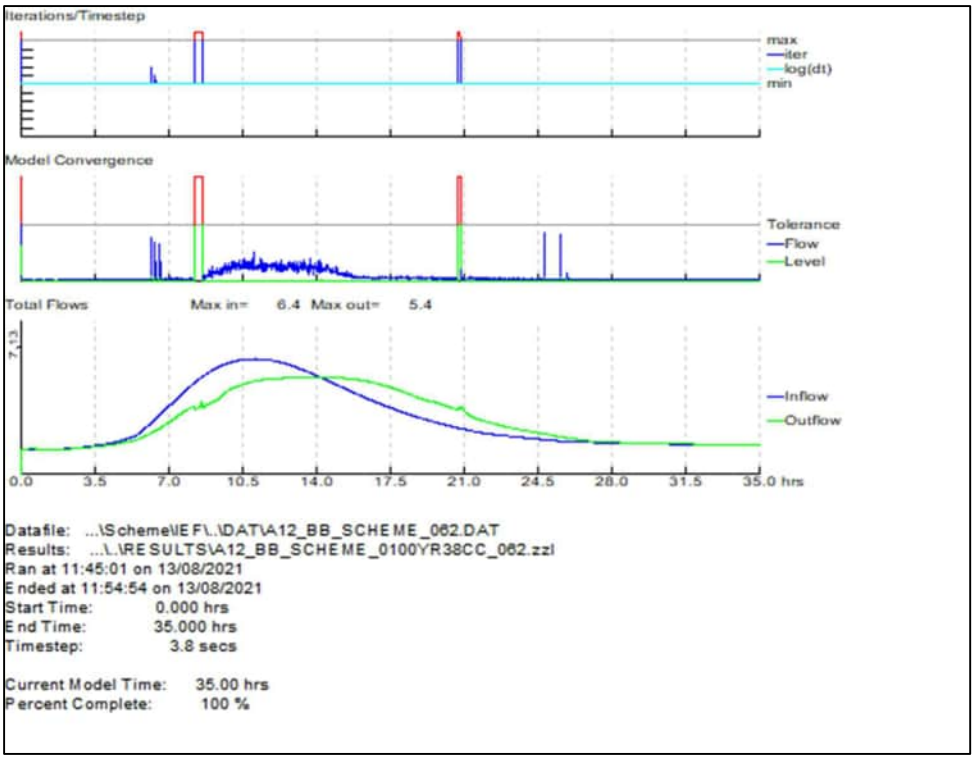


Plate 6.2 1D model convergence for with scheme scenario – 1% AEP + 38%CC event



6.3 Calibration and verification

Flow reconciliation

6.3.1 The modelled flows at the downstream end of the model domain were checked against the overall catchment hydrology analysis. A flow multiplier of 0.9 was then applied to ensure a good match with the design hydrology.

Calibration

6.3.2 There are no gauges on Boreham Brook or its tributary within the extent of the model. Therefore, it was not possible to calibrate the model. However, some data is available for a high-level verification of the model results.

Verification using historic data

6.3.3 The Environment Agency published historic flood maps for UK watercourses, which show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. However, a review of this data showed that there is no historic flood data for Boreham Brook.

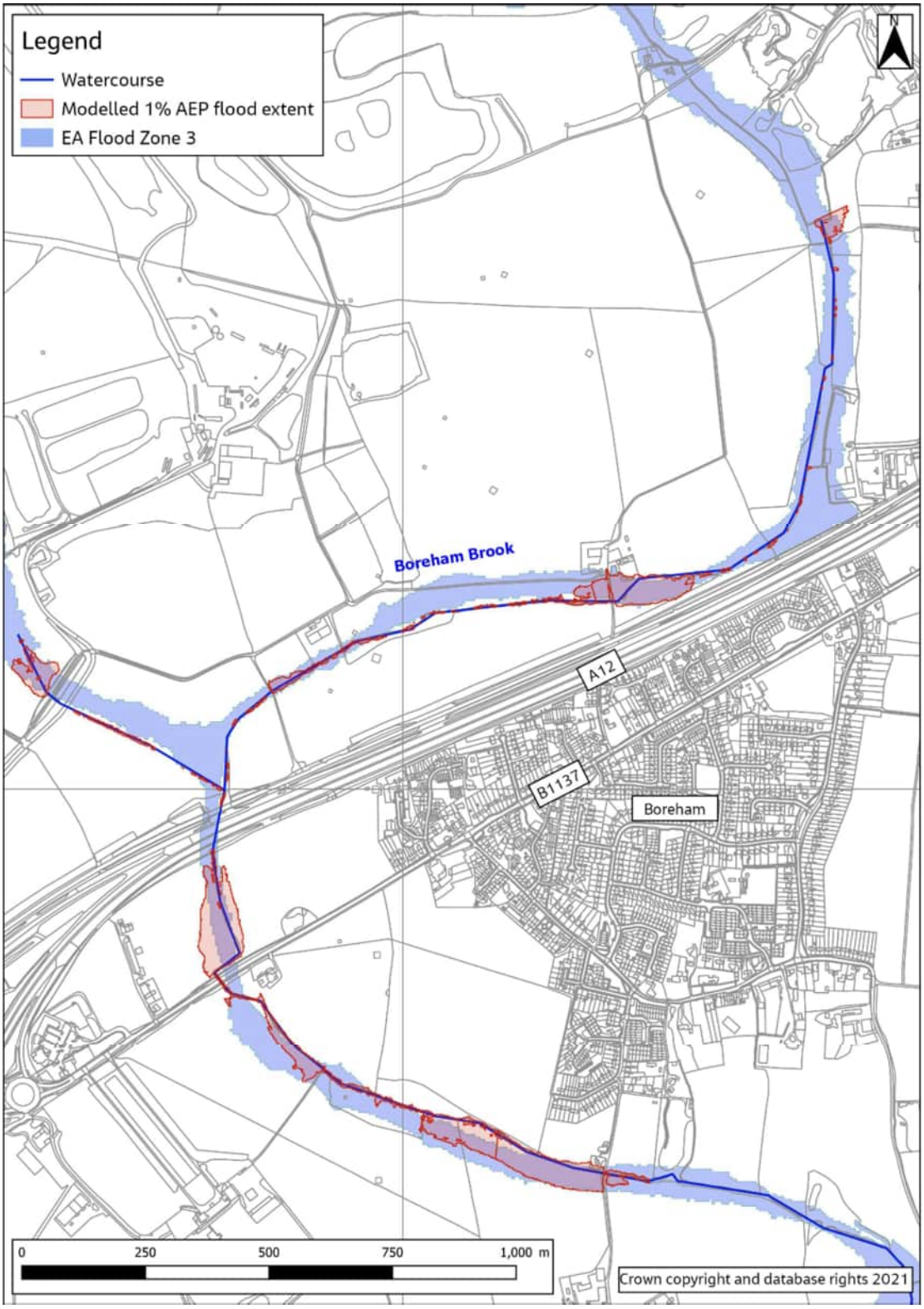
Verification using EA flood maps

6.3.4 Plate 6.3 shows the modelled 1% AEP existing situation flood extent and the Environment Agency published Flood Zone 3. The comparison shows that the Environment Agency flood map predicts a generally larger flood extent than the model results. This is expected as the EA map is generated based on the high-level J-Flow modelling system which does not explicitly account for the in-channel capacity of the watercourse, which the present study shows to carry the full flood flow for much of the modelled reach. It is presumed that the EA flood map modelling is based on a higher level and conservative assessment of design flow than the detailed analysis utilised in the present study.

Verification conclusion

6.3.5 In conclusion, there is limited data available for verification of the Boreham Brook model. The Environment Agency flood map shows a larger flood extent than the model. This is due to the present study being a more detailed assessment of flooding than the Environment Agency flood map.

Plate 6.3 Modelled 1% AEP event flood extent vs. Environment Agency flood zone 3



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness, hydrological inflows and downstream boundary conditions. These analyses were performed on the penultimate version of the baseline model. However, the changes made in the final version of the baseline model were small (the addition of a single cross-section to the baseline model to match the cross-section in the with-scheme). Therefore, the changes will not change the general outcomes of the sensitivity tests. All sensitivity tests were performed using the 1% AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Table 6.1 shows the impact of changing the model roughness on predicted peak water levels. The results show that the in-channel water levels are on average, moderately sensitive to changes in roughness with local spots along the tributary showing moderate sensitivity.

Table 6.1 Roughness sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	0.23	0.00	0.06
-20% Roughness	-0.32	0.00	-0.07

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Table 6.2 shows the impact of changing the model inflows on predicted peak water levels. The results show that the in-channel water levels are moderately sensitive to changes in flow with some specific model nodes at the downstream end of the model exhibiting greater response. They do not have an impact on the scheme area.

Table 6.2 Flow sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.35	0.00	0.16
-20% Flow	-0.42	0.00	-0.06

Downstream boundary condition sensitivity

6.4.4 The slope in the normal depth downstream boundary was adjusted by +20% and -20%. Table 6.3 shows the response at the downstream end of the model (Flood Modeller node BBR01_0000). The location at which there is no change in water level as a result of changing the downstream boundary has been identified. Distances from this location, in relation to the downstream end of the model (tailwater distance) and in relation to the proposed scheme are also shown. The results show that the effect of the downstream boundary does not reach the scheme location.

Table 6.3 Downstream boundary condition sensitivity results

Sensitivity	Water Level Difference at BBR01_0000	Tailwater Distance (m)	Distance to proposed scheme (m)
+20% Downstream boundary slope	0.05	424	877
-20% Downstream boundary slope	0.06	424	877

7 With scheme model build

7.1.1 The new layout of the A12 highway includes three lanes on both carriageways. To accommodate this, an embankment will be built that overlaps the extended part of two model cross sections (BBR01_1301 and BBR01_1292) as shown in Plate 7.1. The changes made to the cross-section profiles are shown in Plate 7.2 and Plate 7.3. No changes will be made to the A12 culvert.

Plate 7.1 Scheme layout at Boreham Brook crossing

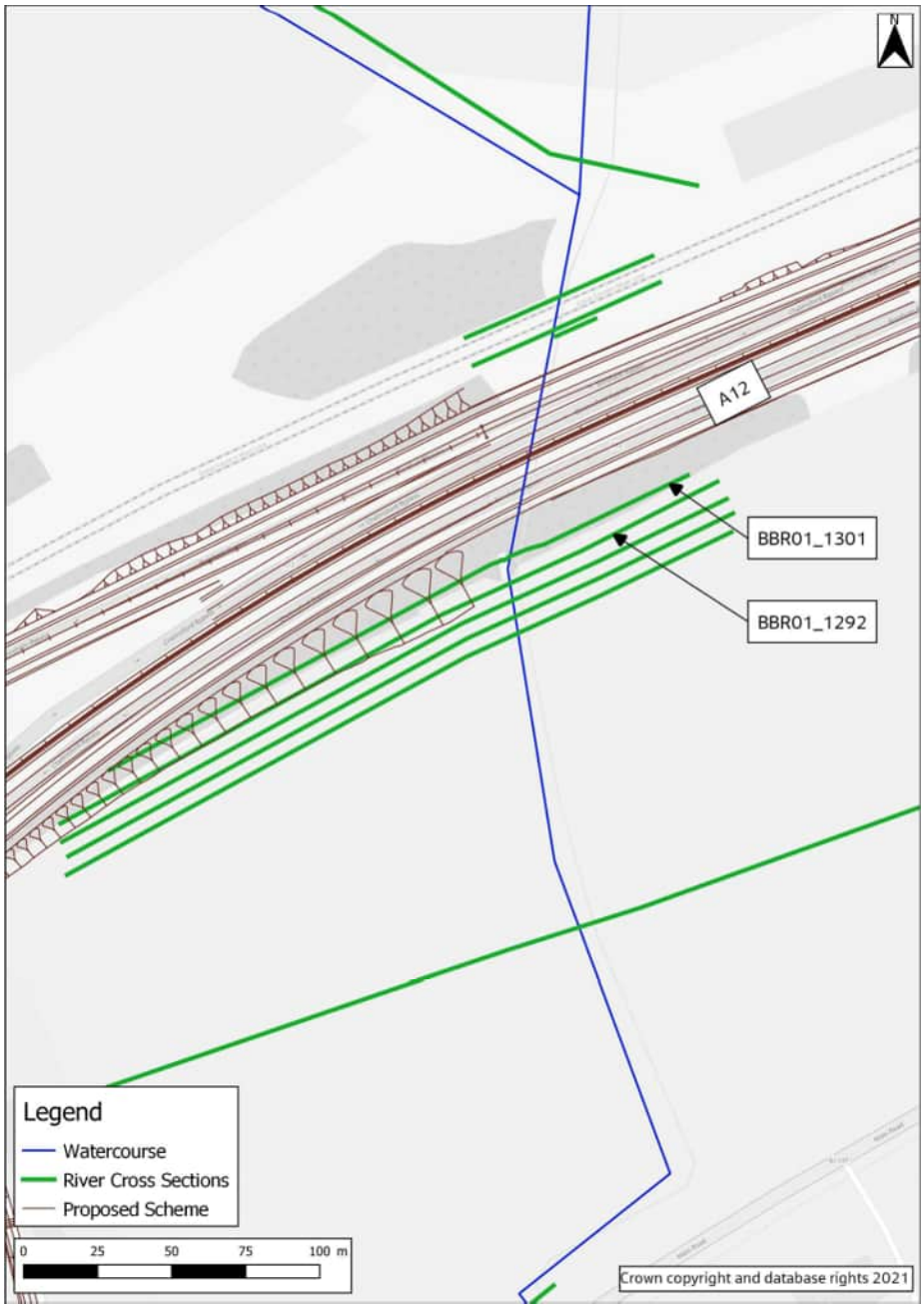
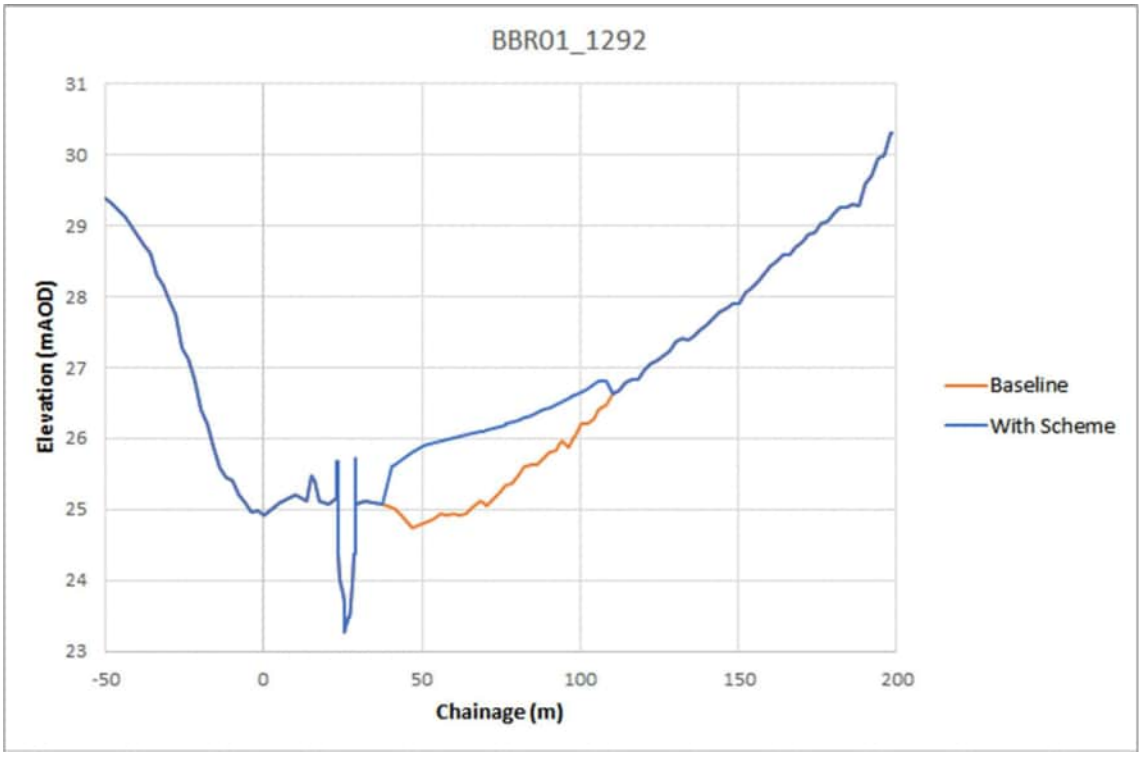


Plate 7.2 Cross section for baseline and with scheme at node BBR01_1301



Plate 7.3 Cross section for baseline and with scheme at node BBR01_1292



8 Model results

8.1 Baseline model results

8.1.1 In-channel maximum water levels have been inspected at key locations in relation to the proposed scheme. Table 8.1 shows in-channel maximum water levels for the 1% AEP +38%CC event at these locations. Water levels at these locations for all other events can be found in Annex C.

8.1.2 Plate 8.2 shows the maximum flood extent for the 1% AEP +38%CC event. The flood extents for all modelled events are shown in Annex D.

8.1.3 The results show that no properties are flooded within the model domain for any of the events simulated and that there is no flood risk to the existing A12 itself. There is flooding between the A12 and the B1137 for all events simulated as well as some inundation of small, low-lying areas adjacent to the watercourse.

8.2 With scheme model results

8.2.1 Table 8.1 compares the in-channel maximum water levels at key locations in the model between the baseline and with scheme scenarios for the 1% AEP +38%CC event simulated. It shows that changes to the water level due to the implementation of the scheme are negligible throughout the model. The A12 culvert is not surcharged in any of the modelled events, as shown in Plate 8.1. Water levels and differences at these locations for all other events can be found in Annex C.

8.2.2 Downstream from the A12 crossing, the proposed scheme elements are all seen to be outside of the modelled flood extent and modelled water levels there are not affected by the implementation of the scheme.

8.2.3 Plate 8.2 also shows a comparison of the flood extents for the baseline and with scheme scenarios. The flood extents are not affected by the implementation of the scheme and there is no visible change in extent at the mapping resolution of 1m. Plate 8.3 shows the water level difference for the 1%AEP +38%CC event. The impacts on maximum flood level difference between baseline and with scheme are categorised into levels of impact due to the scheme. It shows no significant impact to flooding due to the proposed scheme. The flood extents and water level difference maps for higher frequency events can be found in Annex D.

Table 8.1 In-channel water levels at key locations for all events

Nodes	Description	Water Level (mAOD)		
		1% AEP +38%CC		
		Baseline	With Scheme	Diff (m)
BBR01_3466	Upstream extent of Boreham Brook. 2km upstream of the scheme	35.48	35.48	0.00
BBR01_1380	Immediately upstream of the scheme	24.81	24.81	0.00
BBR01_1173	110m downstream of the scheme	24.64	24.64	0.00
BBR01_0000	Downstream end of Boreham Brook. 1.3km downstream of the scheme	17.66	17.66	0.00
BBR02_0531	Upstream extent of Boreham Brook tributary	28.54	28.54	0.00
BBR02_0173d	Confluence of tributary and Boreham Brook	25.40	25.40	0.00

Plate 8.1 With scheme culvert maximum water levels for all events modelled

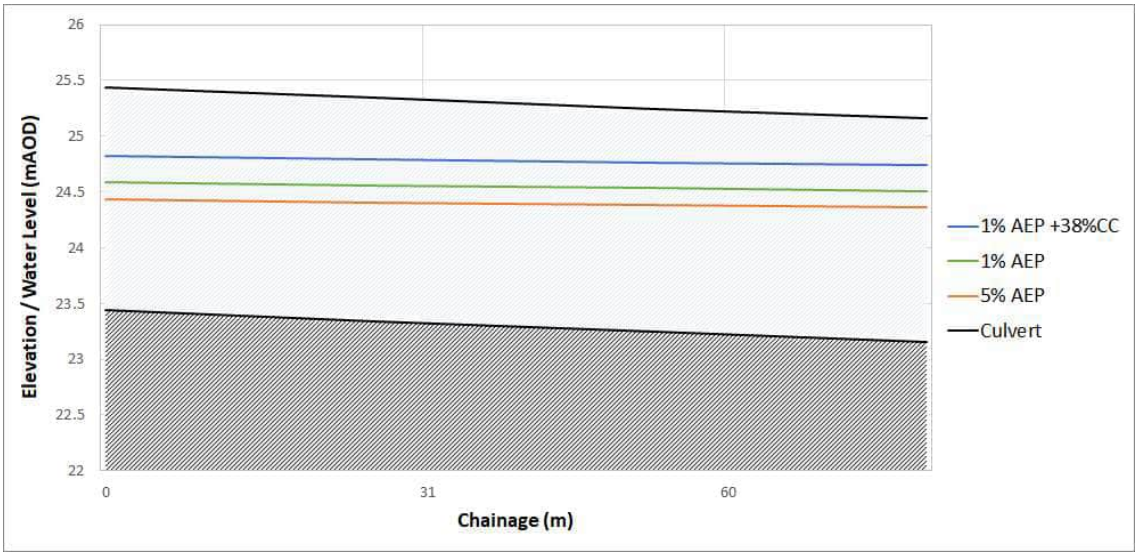


Plate 8.2 Flood extent comparison between baseline and with scheme scenarios for the 1% AEP +38%CC

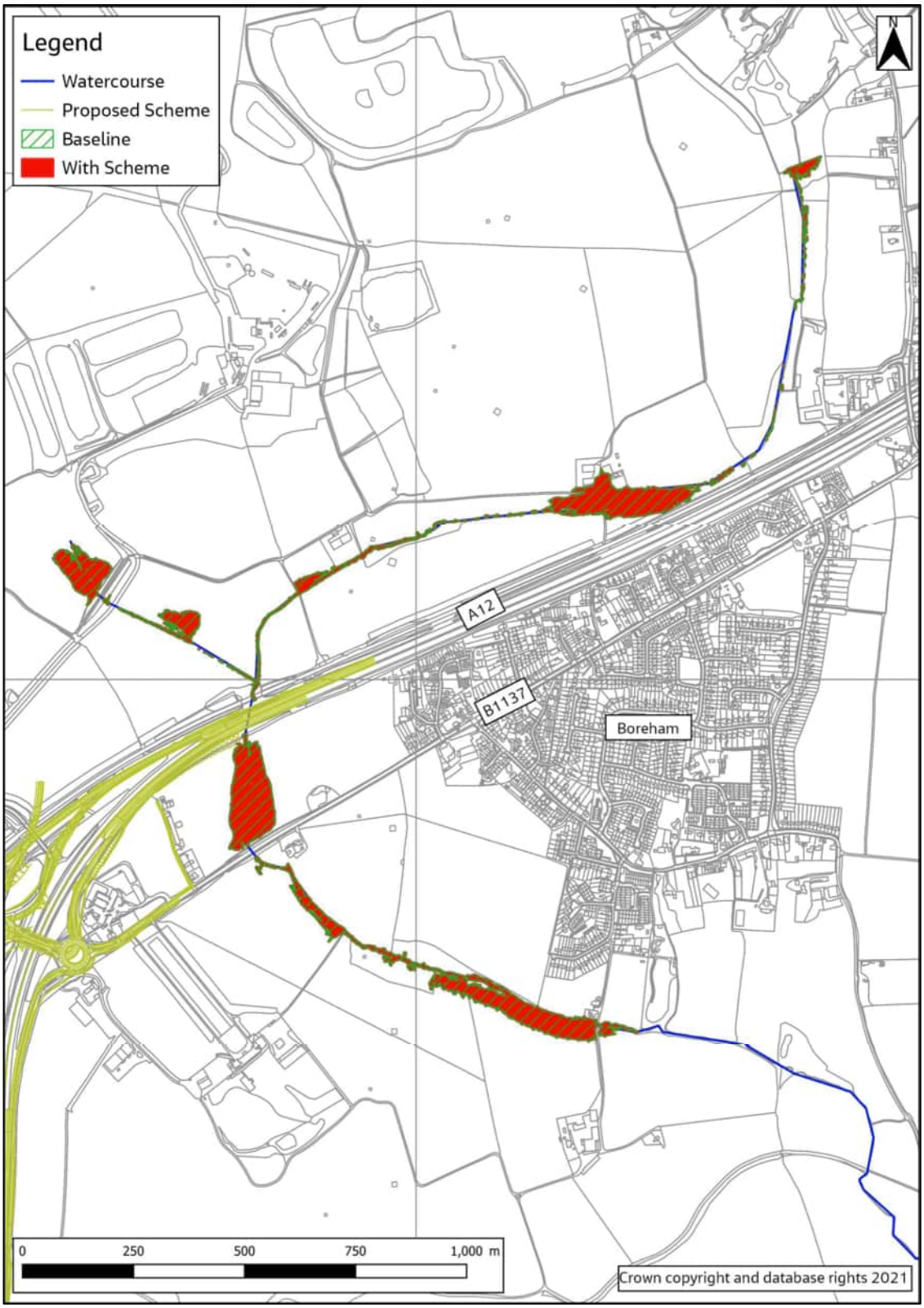
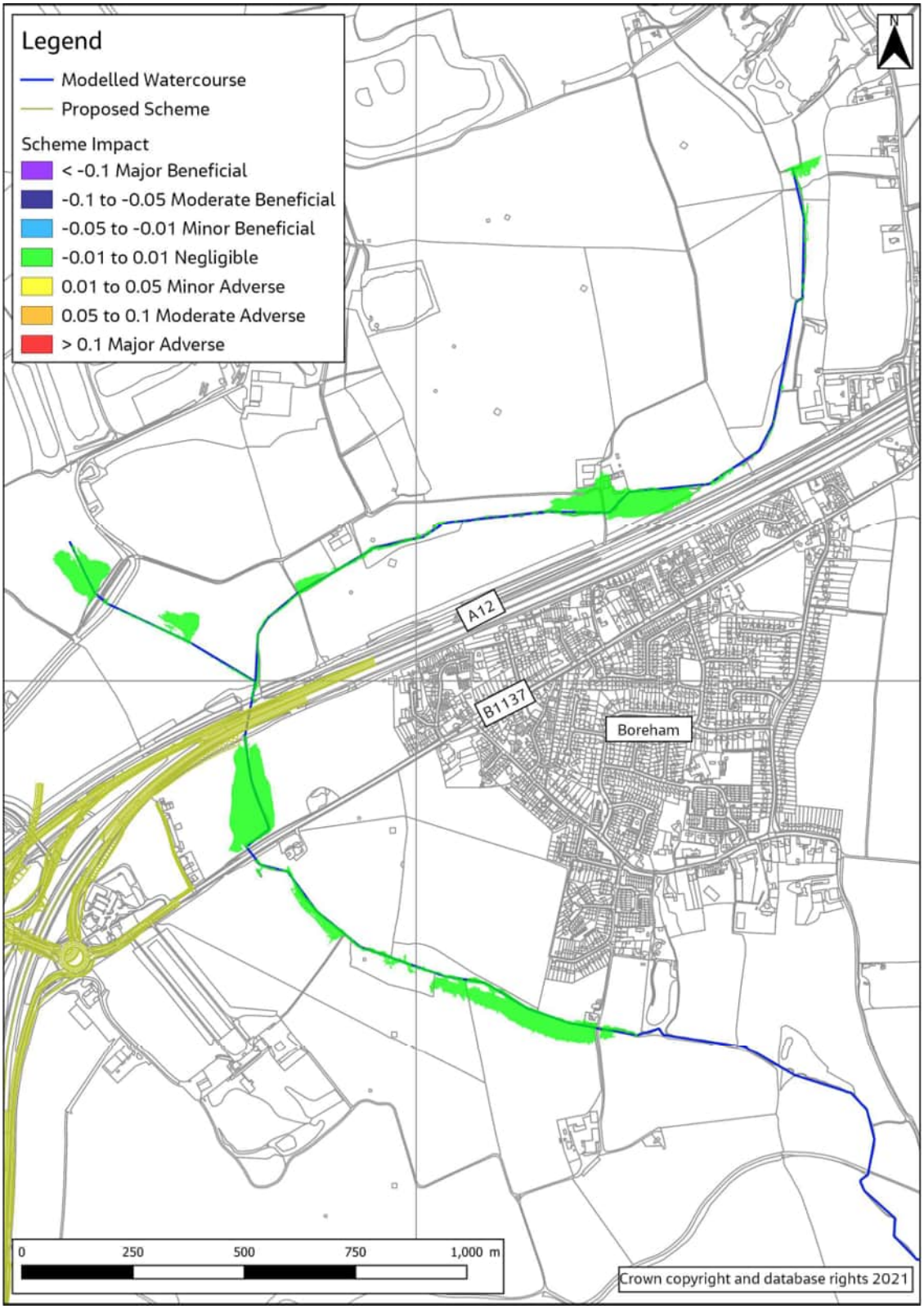


Plate 8.3 Water level difference (with scheme - baseline) for the 1% AEP +38%CC event



9 Model assumptions and limitations

9.1 Hydrology

9.1.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- As the Boreham Brook is a small ungauged catchment there is an uncertainty associated with flow estimation for small, ungauged catchments.
- Most of the sand and gravel pits within the catchment do not have a significant impact on the flood flows as they appear to be disconnected from the river. It is assumed that this is also true during flood events. If this were not true, then it is likely that the storage afforded by the gravel pits would be likely to attenuate the forward flood and lessen the design flood in the vicinity of the A12 crossing.
- ReFH2.2 is potentially overestimating design peak flows for the target catchment (based on results of analysis at a nearby gauge (River Ter @ Crabbs Bridge – 37003).

9.2 Hydraulic modelling

9.2.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological and topographic data included in the model. Therefore, the most appropriate available information has been used to construct the model.

9.2.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are considered to be generally conservative for modelled water levels at the proposed scheme and are therefore appropriate for the flood risk assessment.

9.2.3 The key sources of uncertainty and the limitations associated with the modelling undertaken for Boreham Brook and its tributary are as follows:

- Surveyed cross sections have been extended using LiDAR data.
- Channel roughness has been assigned using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the Flood Modeller software. The dimensions for structures have been based on detailed survey measurements.
- A normal depth boundary was used at the downstream end of the model. The sensitivity analysis has shown that changes to the downstream boundary only impact on water levels up to 424m

upstream of the downstream model extent, which is 877m downstream from the proposed scheme. Therefore, it was deemed appropriate to use a normal depth boundary.

- No calibration was carried out as the Boreham Brook catchment is ungauged.

10 Conclusion

10.1.1 This report has detailed the modelling carried out to assess the baseline and with scheme flood risk for Boreham Brook and its tributary with respect to the location of the proposed highway scheme.

10.1.2 The results of the baseline modelling have shown that there are no properties at risk of flooding and the A12 remains free from flooding up to the 1% AEP +38%CC event. This remains the case with the proposed scheme in place.

10.1.3 The presence of the scheme does not cause any significant difference in flood extent and water levels. Therefore, no mitigation has been proposed for this watercourse.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

References

Chow, Ven Te (1959). Open-Channel Hydraulics. McGraw-Hill.

Environment Agency (2021). Flood risk assessment: climate change allowance guidance.

Environment Agency (2017b). Flood Estimation Guidelines. Technical Guidance 197_08.

Environment Agency (2017c). Spatial Data Catalogue. Accessed October 2017. http://environment.data.gov.uk/ds/catalogue/#/catalogue.

Environment Agency (2020). Flood Estimation Guidelines. Technical guidance 197_08.

Jacobs (2022). Environmental Statement - Appendix 6.6 - Flood Risk Assessment. Ref: TR10060/APP/6.6.

Soil Map of England and Wales - Soil Survey of England and Wales (1983). Soil Maps of England and Wales Scale 1:250,000.

Annex A **Boreham Brook Flood Estimation Calculation Record**

See document HE551497-JAC-EWE_SCHW-RP-LE-0070.docx

Annex B **Structure Schedule**

Table B.1 Model Structure Schedule

Structure Name	Node Label(s)	XY Coordinates	Modelling Approach
Foot Bridge 01.037	BBR01_3406bu	575865, 211106	Represented as Arch Bridge + Spill unit. Data from detailed survey (Jacobs 2017). FM default coefficients applied for all parameters.
Foot Bridge 01.032	BBR01_2759bu	575742, 210492	Represented as Arch Bridge + Spill unit. Data from detailed survey. Spill has weir coefficient of 1.5 for concrete slab. FM default coefficients applied for all other parameters.
Farm Bridge 01.029	BBR01_2404bu	575587, 210432	Represented as Arch Bridge + Spill unit. Data from detailed survey. FM default coefficients applied for all parameters.
Farm Bridge 01.024	BBR1_1671ou1 BBR1_1671ou2	574736, 210195	Bridge represented as twin orifices for stability. Data from detailed survey. Spill unit has coefficient of 0.9 for gravel/earth and vegetation. FM default coefficients applied for all other parameters.
Railway Bridge	BBR01_1392bu	574632, 209964	Represented as Arch Bridge with four openings. Data from detailed survey. No direct measurements of fourth opening so dimensions assumed from drawing. No spill required due to high headwall. FM default coefficients applied for all parameters.
A12 Road Bridge	BBR01_1380ci BBR01_1380cu	574636, 209952	Box culvert represented with rectangular conduit and inlet/outlet units.

	BBR01_1301cd		Data from detailed survey, but width set to 4 m instead of 5 m to account for skew. Spill not required. Roughness value of 0.05 applied to bed for natural materials. Mid-value of roughness taken for walls and soffit (0.02), representing aged condition concrete. Headloss amended to 0 as the culvert width is the same as channel. FM default coefficients applied for all other parameters.
Farm Bridge 01.017	BBR01_1292bu	574615, 209867	Represented as Arch Bridge with Spill unit. Data from detailed survey. FM default coefficients applied for all parameters. Bridge relocated downstream for the “with scheme” scenario.
B1137 Main Road	BBR01_1012ci BBR01_1012cu BBR01_0981cd	574621, 209624	Culvert represented using circular conduit and inlet/outlet units. Spill not required. Culvert outlet replaced with outfall to improve stability. Manning’s n value of 0.012 applied for finished concrete. FM default coefficients applied for all other parameters.
Farm Bridge 01.010	BBR1_0714ci1 BBR1_0710cu1 BBR1_0710cd1 BBR01_0714ou	574838, 209434	Represented using circular conduit and inlet/outlet units. Flood relief culvert modelled with orifice. Data from detailed survey. Weir coefficient of 1.5 for concrete slab. Manning’s n value of 0.012 applied for finished concrete. FM default coefficients applied for all other parameters.
Vehicle Crossing	BBR1_0424ou1 BBR1_0424ou2	575105, 209332	Represented using twin Orifice units. Data from detailed survey.

			Spill has weir coefficient of 0.9 due to thick vegetation. FM default coefficients applied to all other parameters.
Road Bridge	BBR01_0099ou	575405, 209224	Represented with Orifice unit. Spill not required. Data from detailed survey. Weir coefficient of 0.9 due to presence of thick vegetation and gravel/earth. FM default coefficients applied for all other parameters.
Bridleway Bridge	BBR02_040ou	574284, 210190	Represented with orifice unit. Spill not required. Data from detailed survey. FM default coefficients applied for all parameters.
Quarry Access Road Bridge	BBR02_0373ci BBR02_0373cu BBR02_0349cd	574308, 210170	Represented using circular conduit and inlet/outlet units. Spill unit has coefficient of 0.9 for gravel/earth and vegetation. Data from detailed survey. Manning's n value of 0.012 applied for finished concrete.
Farm Bridge 02.003	BBR02_0176bu	574486, 210087	Represented using Arch Bridge unit. Data from detailed survey. FM default coefficients applied for all parameters.

Annex C **Maximum Water Level Tables and Long Sections**

Table C.1 Maximum Water Level of Baseline and With Scheme Models for all Events at Key Nodes

Nodes	Description	Water Level (mAOD)								
		5% AEP			1% AEP			1% AEP +38%CC		
		Baseline	With Scheme	Diff (m)	Baseline	With Scheme	Diff (m)	Baseline	With Scheme	Diff (m)
BBR01_3466	Upstream extent of Boreham Brook. 2km upstream of the scheme	35.36	35.36	0.00	35.41	35.41	0.00	35.48	35.48	0.00
BBR01_1380	Immediately upstream of the scheme	24.42	24.42	0.00	24.58	24.58	0.00	24.81	24.81	0.00
BBR01_1173	110m downstream of the scheme	24.09	24.09	0.00	24.30	24.30	0.00	24.64	24.64	0.00
BBR01_0000	Downstream end of Boreham Brook. 1.3km downstream of the scheme	17.37	17.37	0.00	17.53	17.53	0.00	17.66	17.66	0.00
BBR02_0531	Upstream extent of Boreham Brook tributary	28.06	28.06	0.00	28.26	28.26	0.00	28.54	28.54	0.00
BBR02_0173d	Confluence of tributary and Boreham Brook	25.16	25.16	0.00	25.29	25.29	0.00	25.40	25.40	0.00

Plate C.1 Long profile for baseline and with scheme scenarios showing peak water levels for the 5% AEP event

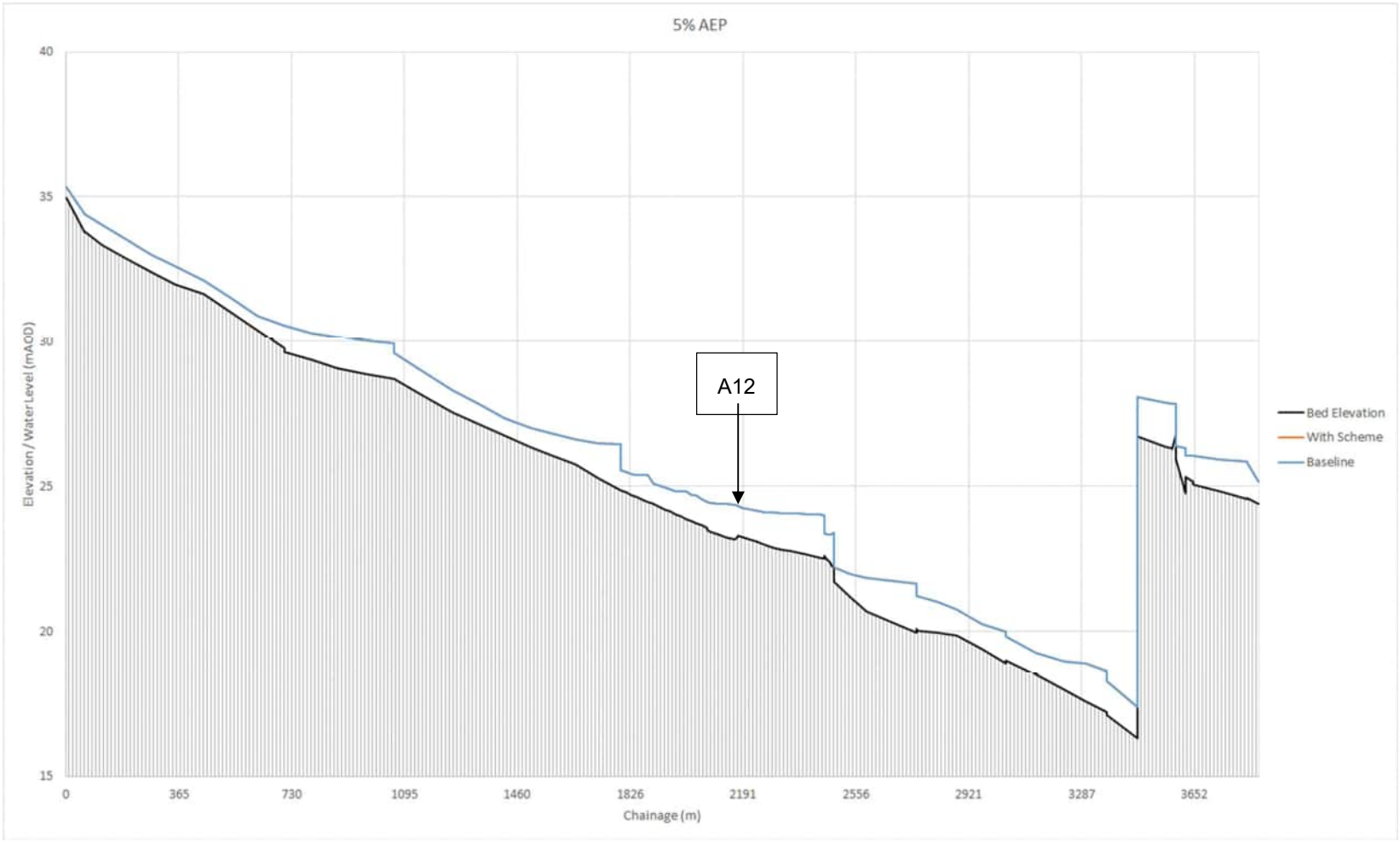


Plate C.2 Long profile for baseline and with scheme scenarios showing peak water levels for the 1% AEP event

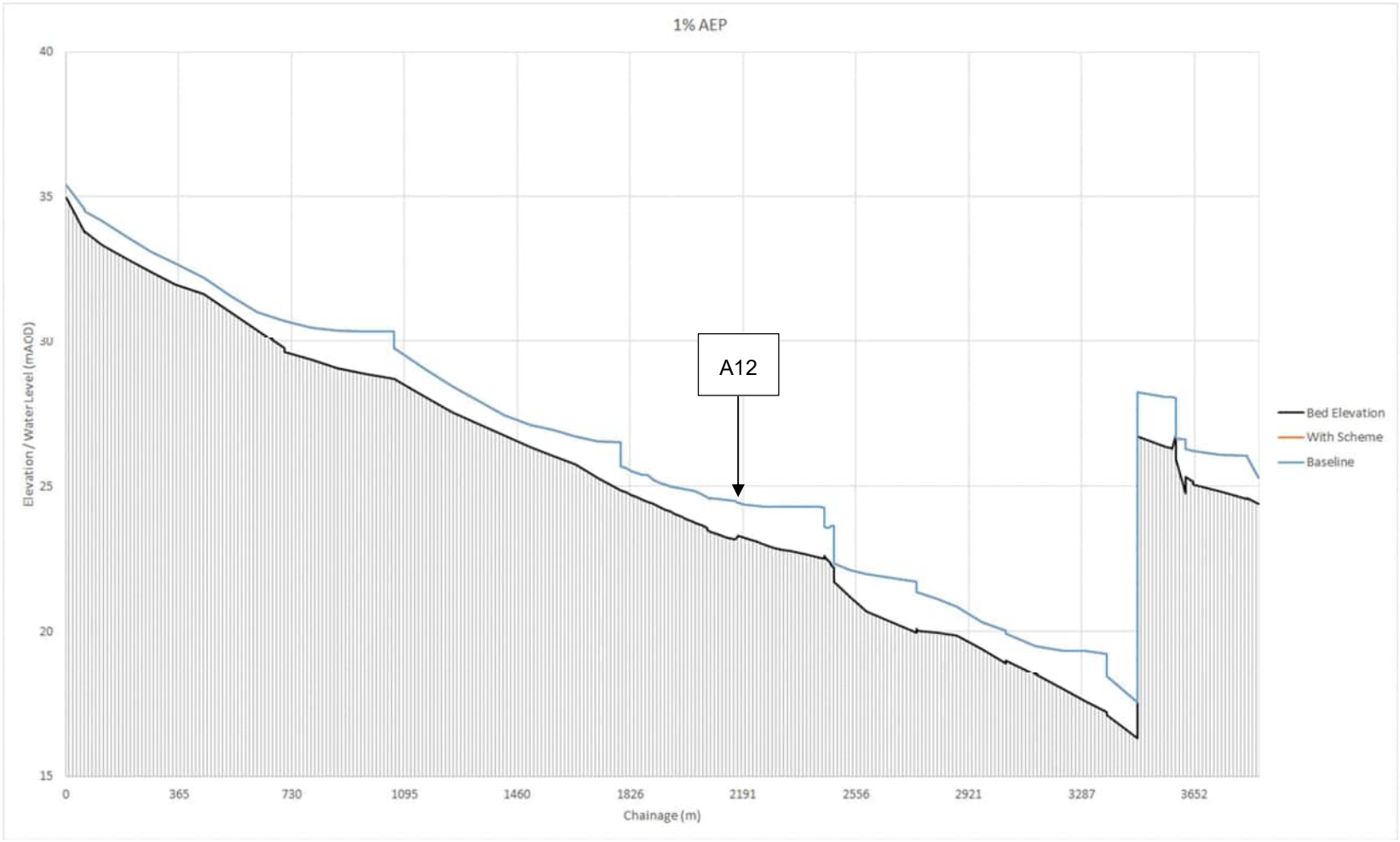
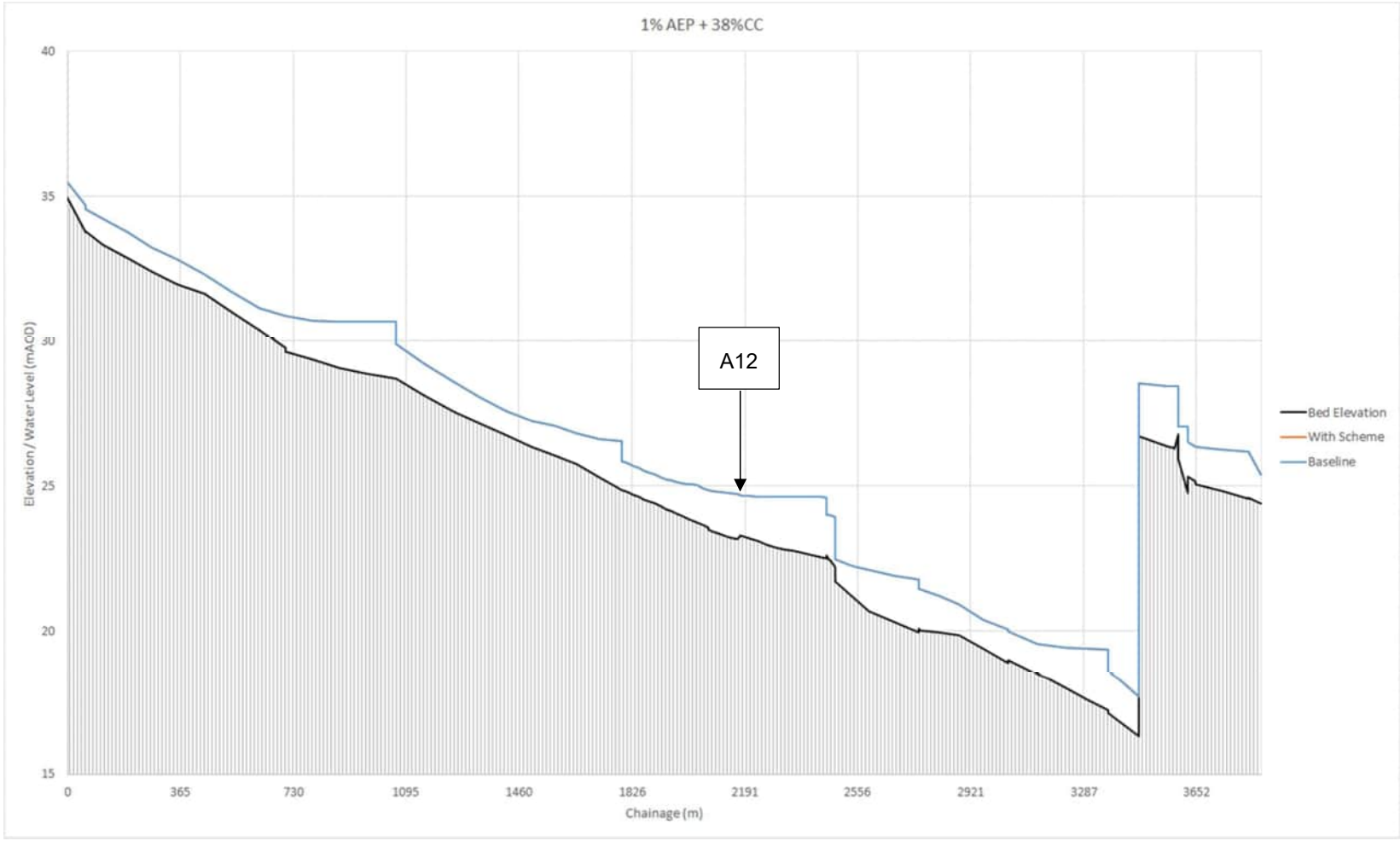


Plate C.3 Long profile for baseline and with scheme scenarios showing peak water levels for the 1% AEP +38%CC event



Annex D **Flood Mapping**

Plate D.1 5% AEP maximum flood extent for baseline and with scheme scenarios

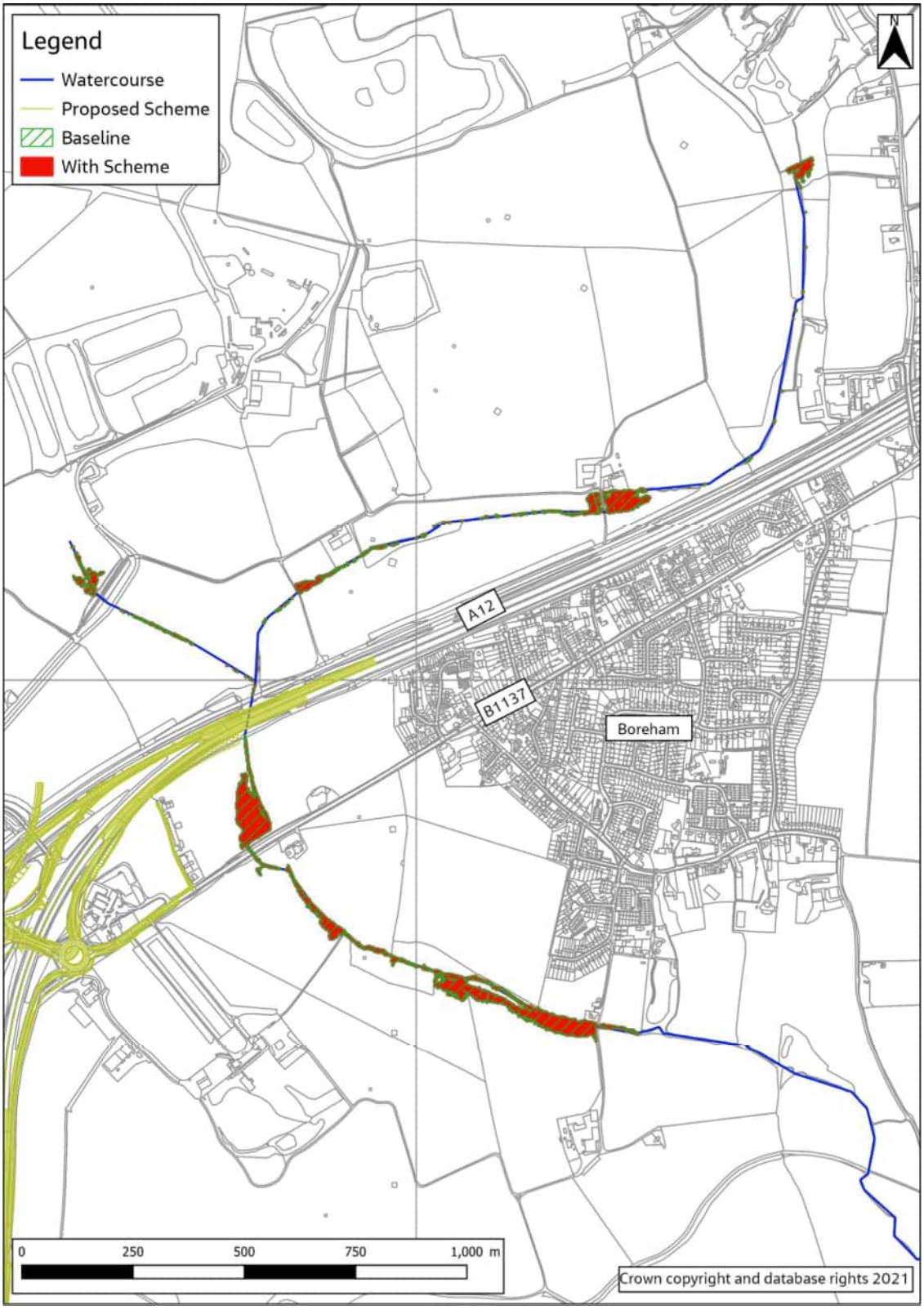


Plate D.2 1% AEP maximum flood extent for baseline and with scheme scenarios

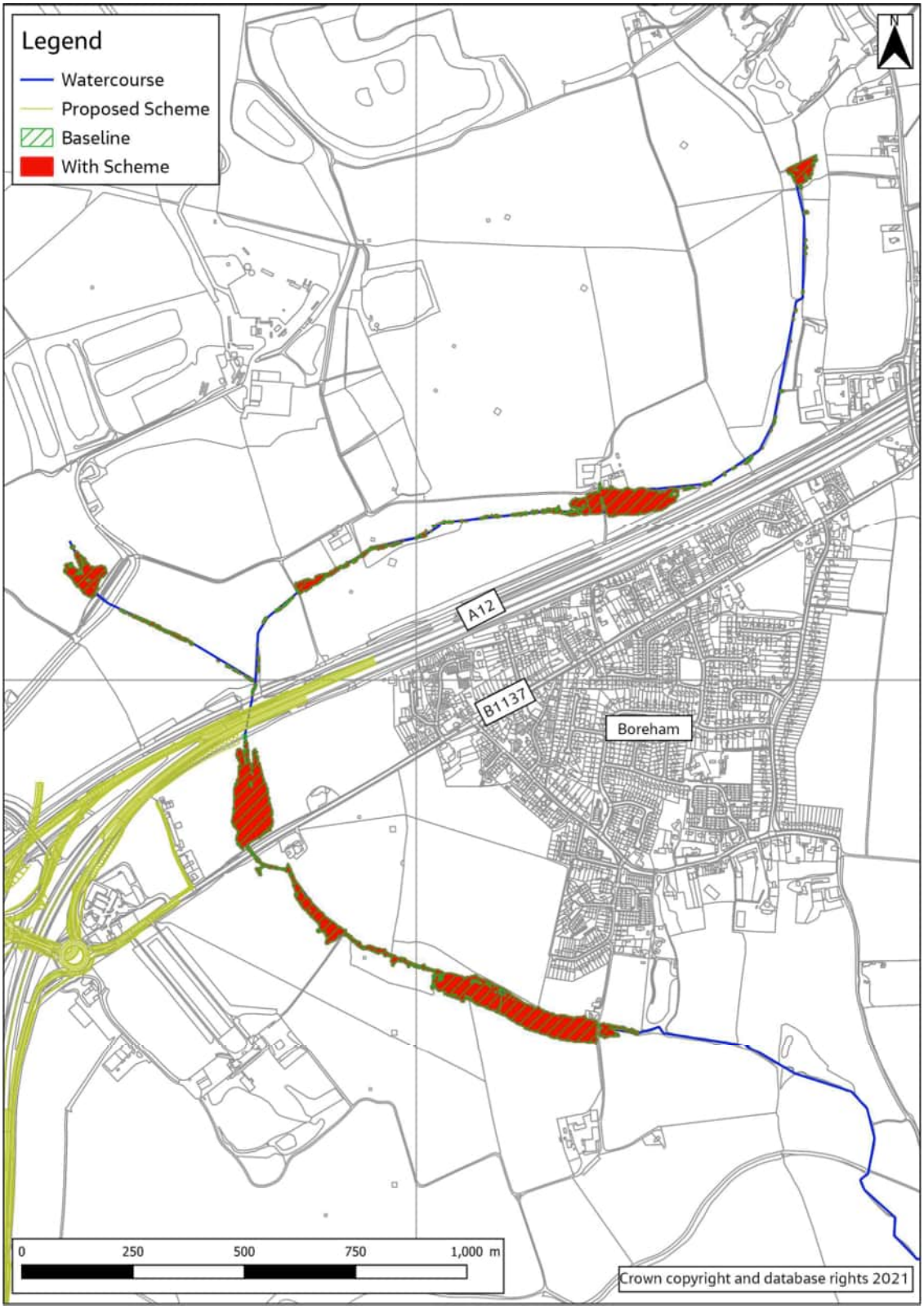


Plate D.3 1% AEP +38%CC maximum flood extent for baseline and with scheme scenarios

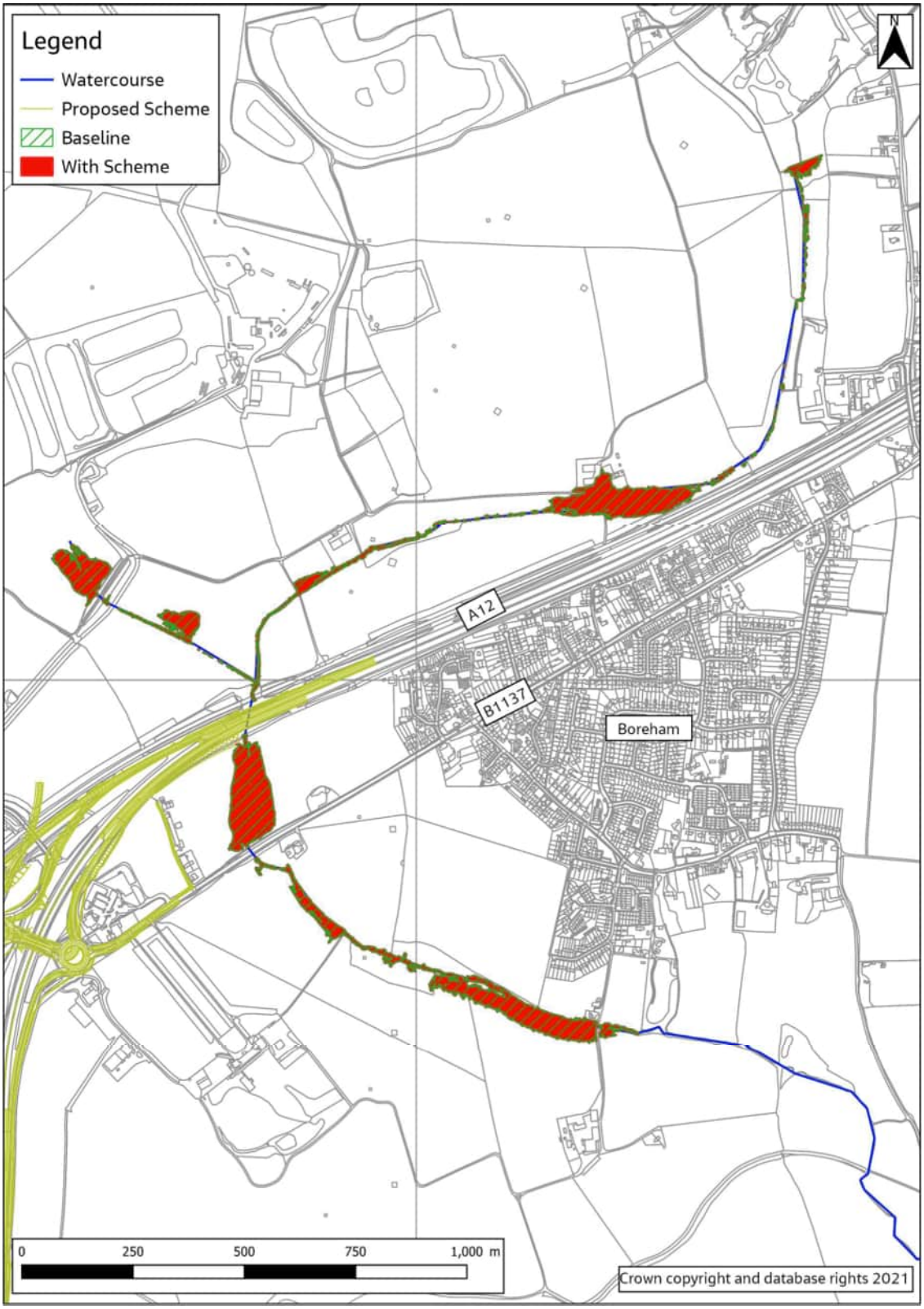


Plate D.4 Water level difference (with scheme - baseline) for the 5% AEP event

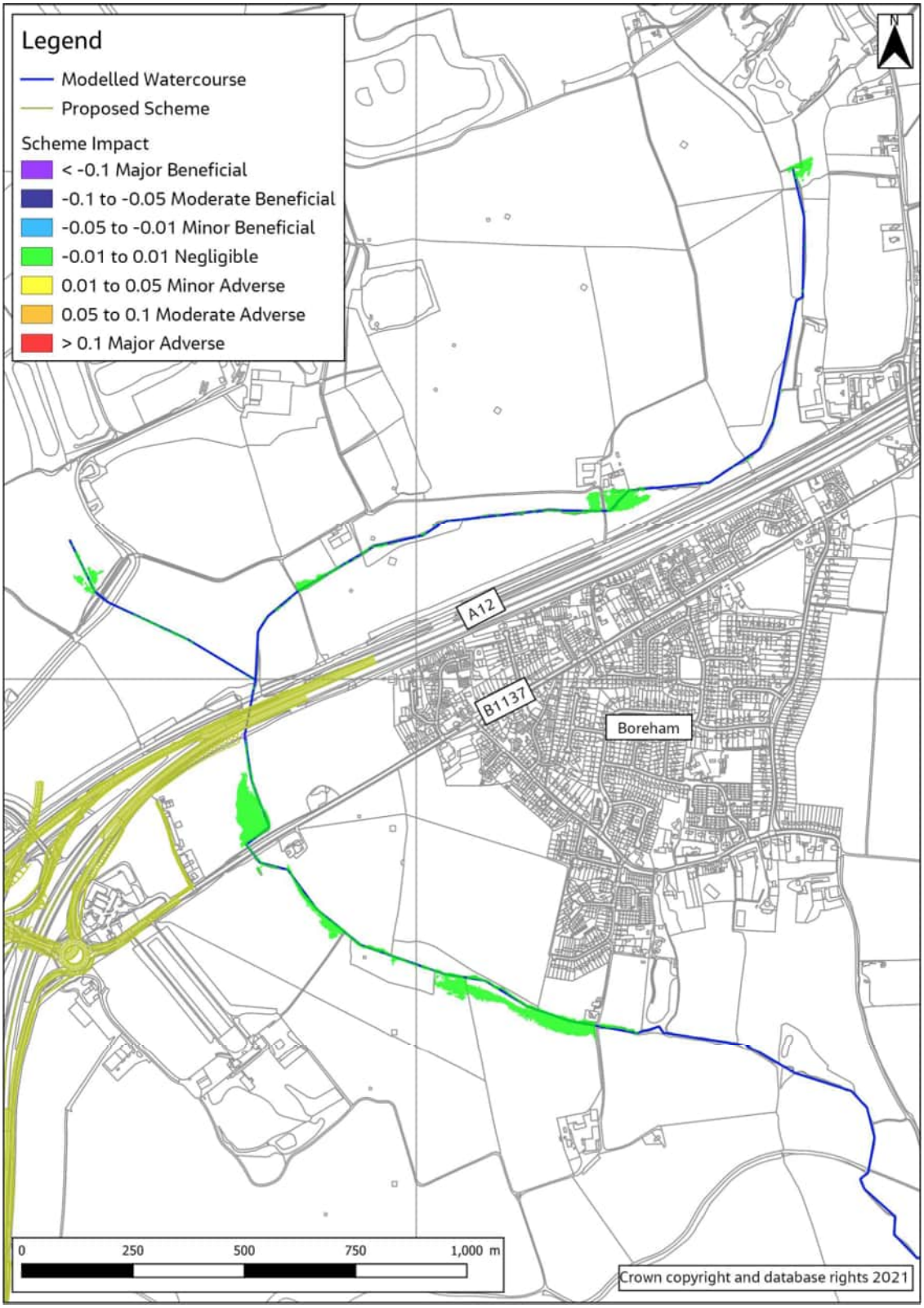


Plate D.5 Water level difference (with scheme - baseline) for the 1% AEP event

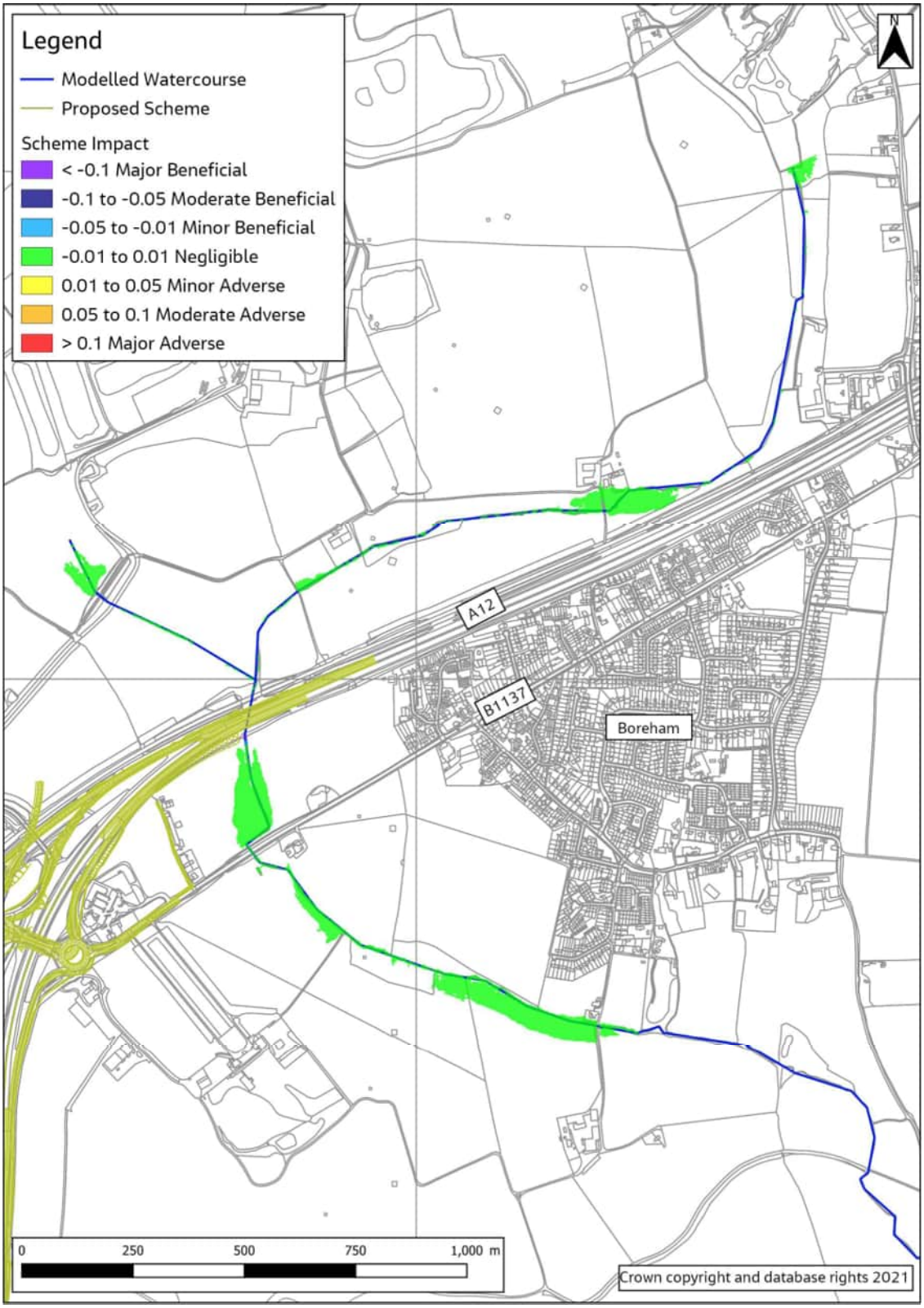
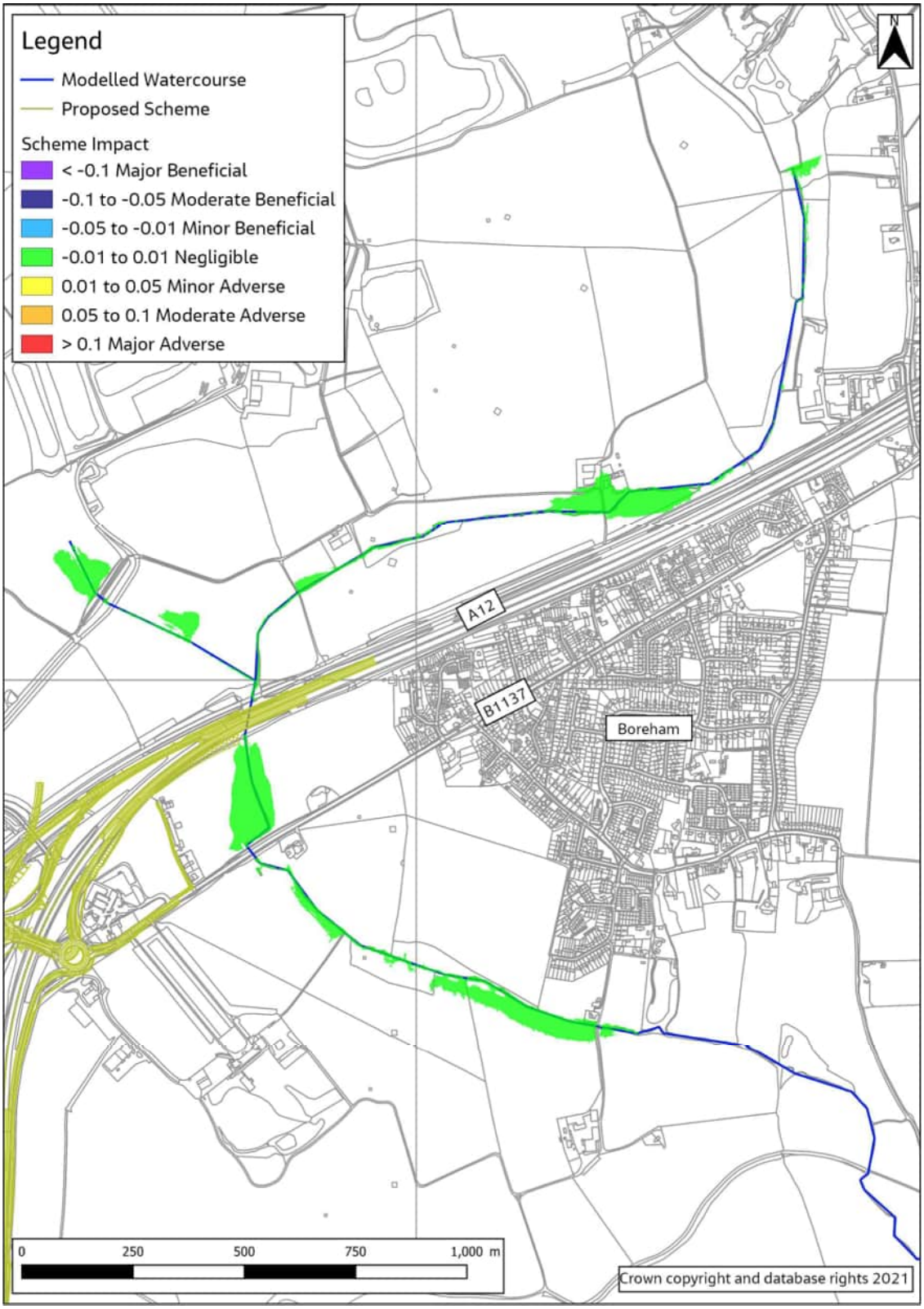


Plate D.6 Water level difference (with scheme - baseline) for the 1% AEP +38%CC event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT BOREHAM BROOK HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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1 Executive Summary

This document has been prepared to support the A12 Boreham Brook with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This record sheet is to be used where flood estimates are needed at a single location.

3 Method Statement

3.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Peak flow or hydrograph? • Range of return periods • Approx. time available	The A12 is proposed to be widened and therefore design peak flows and hydrographs are required for input into the Boreham Brook numerical hydraulic model that is to be developed for assessing flood risk in the vicinity of the scheme. Design peak flows and hydrographs are required for the following AEP events: 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 1% plus climate change adjustment (plus CC), 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 100 plus CC, 200 and 1000-year design return periods). For the climate change uplift the following scenario will be tested: • 1% AEP event plus 38% uplift (based on most recent Environment Agency (EA) 2021 guidance[1] - Combined Essex Management Catchment peak river flow allowance, higher limit, 2080s scenario)

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The Boreham Brook catchment is a lowland catchment in the south east of England. The catchment is essentially rural with a small urban area around Boreham. There are small pockets of woodland within the catchment and a disused airfield. The geology of the catchment has been assessed as being predominantly London Clay overlain with superficial deposits of mainly bolder clay and some sands and gravels. The Boreham Brook catchment has numerous sand and gravel pits and small waterbodies assumed to be associated with the sand and gravel pits. The soils within the catchment are predominantly of the chalky till variety with glaciofluvial drift around the watercourse. The soils are noted as having the potential for slow permeability with some areas of clayey subsoils.

3.3 Source of flood peak data

Was the NRFA/HiFlows UK dataset used? If so, which version?	Yes, WINFAP-FEH v5 data files were used which were downloaded on the 06/07/17. These contain the most up to date NRFA/HiFlows UK dataset at the time of assessment.

[1] Environment Agency (2021). Flood Risk Assessments: Climate Change Allowances.

Record any changes made.	

3.4 Gauging stations (flow or level)

3.4.1 The river is ungauged the following table presents the nearby gauges considered as possible donor catchments for Boreham Brook.

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
River Ter	Crabbs Bridge	037003	37003	TL786107	77.8	Level	1964 to present*
River Brain	Guithavon Valley	037009	37009	TL817147	60.7	Level	1962 to present
River Chelmer	Rushes Loch	037002	37002	TL794090	533.9	Not stated in NRFA	1932 to 2010
River Chelmer	Springfield	037008	37008	TL713071	190.3	Level	1965 to present
Sandon Brook	Sandon Bridge	037013	37013	TL755054	75.1	Level	1963 to present
Blackwater	Appleford Bridge	037010	37010	TL844158	247.3	Level	1962 to present

** In terms of acceptable flood monitoring the record starts in 1964, though a record from 1932-1964 are held from a former, less accurate Hatfield Peverel station.*

3.5 Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Crabbs Bridge	1964-2016	No	Yes	Yes	No	Trapezoidal flume with throat tapping. Theoretically rated. Modular limit 0.9m, no level yet recorded above 1.6m. There is increased scatter in gaugings beyond QMED leading to some uncertainty in the accuracy of the high flow rating. One peak flow rating has been applied to this engineered structure across period of record. The Environment Agency (EA) supplied for the study the 15-minute flow (1979 to 2017) and AMAX data (1964 to 2016). (It was noted that the 2015 AMAX value did not match the 15-minute flow data, and the 2016 water year was incomplete. The 2015 / 2016 AMAX values were therefore not used in this analysis. There was also a difference observed in the supplied EA AMAX series to the values recorded in the Peak Flow dataset on the NRFA website for water year 1963. The EA supplied value was favoured. Water year 1980 was further excluded from QMED assessment as it has been rejected in WINFAP. The EA AMAX QMED = 5.69m^3/s whereas to the equivalent Peak Flow QMED = 5.42m^3/s. The EA AMAX series was accepted as the most up to date interpretation of the record and was used in this study.
Guithavon Valley	1962-2016	No	Yes	Yes	No	Runoff increased by effluent returns and reduced by industrial/agricultural abstraction. Drowning noted to occur at very low levels but noted to have minimum effect. A previous study by CH2M (2017) indicates that the rating curve for this gauge is low in relation to other local gauges and suggests further work is required to improve the confidence in the rating.
Rushes Loch	1932-2010	No	No	No	No	Station with complex hydrometric history with rural catchment on London Clay with significant urban development. Weir provides head for lock (navigable river), discharge through lock not measured. Lock thought to influence flow pattern at low flows during busy periods, e.g. in summer of 2006. Weir repaired in 1982 because of serious leakage. No accurate measure of low or high flow.

Station name	**Start and end of data in HiFlows-UK**	**Update for this study?**	**Suitable for QMED?**	**Suitable for pooling?**	**Data quality check needed?**	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Springfield	1965-2016	No	Yes	Yes	No	'Essex' profile located in Chelmsford gauging a predominantly rural catchment with mixed geology and some artificial influences.
Sandon Bridge	1963-2016	No	Yes	Yes	No	Insensitive 'Essex' profile weir gauging a rural catchment with reservoirs affecting the regime. Naturalised flows (1963-1976) account for reservoir compensation water (0.011 cumecs) and storm overflows. Minor additions from industrial effluent.
Appleford Bridge	1962-2016	No	Yes	Yes	No	Station with poor quality record gauging a groundwater-influenced rural catchment with major artificial influences. Groundwater -influenced regime. Intermittently affected, since 1971, by Ely / Ouse Transfer Scheme pumping, significantly affected in the late 1980s and in the 1990s. Abstractions from both chalk and gravel aquifers for private water supply. Previous studies by CH2M (2017) indicates that the rating curve for this gauge may be unreliable and further spot gauging's particularly at higher flows was recommended to improve the rating.
Give link/reference to any further data quality checks carried out	No further data quality checks were undertaken.					

3.6 Rating equations

Station name	**Type of rating** e.g. theoretical, empirical; degree of extrapolation	**Rating review needed?**	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Crabbs Bridge	Theoretical	No, though a high-level check on the data provided was made.	The rating is based upon a theoretical understanding developed by the gauging authority (the EA). Flow is modular beyond 0.9m (circa 3.2m^3/s) - 0.95m (circa 3.8m^3/s) and the ratings do not include bypass or non-modularity for which the EA noted they make adjustments for in the data they provided.
Guithavon Valley	N/A	No	Detailed rating review outside the scope of this study. A rating review was carried out for Guithavon Valley by CH2M (2017) and Hydro-Logic (2013-2014). The CH2M study noted the rating curve for Guithavon Valley was low and it recommended that further spot gaugings should be carried out particularly at higher flows and the rating reviewed.
Rushes Loch	N/A	No	Rating review outside the scope of this study.
Springfield	N/A	No	Rating review outside the scope of this study.
Sandon Bridge	N/A	No	Rating review outside the scope of this study.
Appleford Bridge	N/A	No	Rating review outside the scope of this study. The rating curve for Appleford Bridge was assessed by CH2M (2017) and Hydro-Logic (2013-2014). The CH2M study recommended that further spot gauging's should be carried out at higher flows and the rating reviewed as there were discrepancies noted in the consistency of flow estimates between gauges on the River Blackwater.
Give link/reference to any rating reviews carried out			

3.7 Other data available and how it has been obtained

Type of data	**Data relevant to this study?**	**Data available ?**	**Source of data and licence reference if from EA**	**Date obtained**	**Details**
Check flow gaugings (if planned to review ratings)	No detailed rating reviews are proposed to be carried out for this study.				

Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies (e.g. CFMPs, Strategies)					
Other data or information (e.g. groundwater, tides)					

3.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes - FEH Statistical method considered appropriate for this relatively small and essentially rural catchment. As it is considered sensible to compare the peak flows in small catchments derived from two independent methods, Version 2 of ReFH2, available while the hydrological analysis was being undertaken, has been applied to compare the resulting peak flows with those from the FEH Statistical method. ReFH2.2 has also been used in the process of deriving design hydrographs since the watercourse has no gauge from which to derive a typical hydrograph shape.
Outline the conceptual model, addressing questions such as: • What is likely to cause flooding at the site (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might the site flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Flooding at the site may be caused by peak flows / flood volume. Flooding has been indicated by the EA Flood Maps for Development along the watercourse upstream and downstream of Boreham and in the vicinity of the sand and gravel pits on the Brick House Farm tributary. There are numerous waterbodies within the catchment, which are generally associated with the sand and gravel pits. Most of these waterbodies are offline and their presence is not anticipated to have a major impact on the fluvial downstream flooding i.e. any flood risk is likely to be associated with processes across the catchment.

	No debris dam issues are known about for this catchment.
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, use permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	There are numerous waterbodies which appear to be associated with the sand and gravel pits within the Boreham Brook catchment. The majority of these waterbodies have been assessed as being offline and therefore are not considered to unduly impact design peak flow estimates. The FEH Web Service records a FARL of 0.962 for the Boreham Brook catchment to the model extend. Some attenuation due to reservoirs / waterbodies is therefore likely to be occurring but the impact of such attenuation is not considered to have a major influence on the design peak flow estimates. The geology of the catchment has been noted by the FEH Web Service as having rocks with essentially no groundwater and therefore the catchment is not considered highly permeable. The catchment BFIHOST (0.529) and SPRHOST (41.77) values are in the middle of the normal range and do not suggest the catchment to be particularly permeable or impermeable. The Boreham Brook catchment is classified by FEH as 'essentially rural' with only limited urban areas predominantly in the lower reaches of the catchment around Boreham. The FEH catchment descriptors FPEXT value to the model extent is 0.0887. This suggests the Boreham Brook catchment has some but limited floodplain storage within the catchment.
Initial choice of method(s) and reasons	The FEH Statistical method and ReFH2.2 will be undertaken for the Boreham Brook catchment. Design peak flows will then be adopted following a comparison of the peak flows from the two methods. The design hydrograph shape will be based on the output from ReFH2.2 which will be scaled to agree with the finalised peak flow.
Software to be used (with version numbers if relevant) e.g. FEH Web Service (date accessed) WINFAPv4	FEH Web Service – accessed 29th June 2017 WINFAPv4 WINFAP-FEH v5 data files – downloaded 6th July 2017 ReFH2.2 / FEH13 rainfall data

WINFAP-FEH v5 data files (previously HiFlows) ReFH2.2	

3.9 Site details

Watercourse	Site	Easting	Northing	AREA on FEH Web service(km^2)	Revised AREA if altered (km^2)
Boreham Brook	Boreham	575500	209200	14.79	14.73

3.10 Catchment descriptors (incorporating any changes made)

FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT 2000	FPEXT
0.962	0.31	0.529	4.35	17.3	566	41.77	0.0207	0.0887

3.11 Checking catchment descriptors

Record how the catchment boundary was checked and describe any changes (refer to maps if needed)	The Boreham Brook catchment was slightly amended for this assessment. Small alterations were made to the FEH Web Service catchment to take into account contour data (as provided in the Ordinance Survey 1:25,000 basemaps).
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	The soils within the catchment were assessed using the Soil Map of England and Wales at a scale of 1:250,000 (published 1983). Soils were noted to be of predominantly chalky till variety with glaciofluvial drift around the watercourse. Based on this assessment the FEH SPRHOST and BFIHOST values were not amended.
Source of URBEXT	$URBEXT_{2000}$ has been extracted for the Boreham Brook catchment from the FEH Web Service.
Method for updating of URBEXT	$URBEXT_{2000}$ amended within WINFAPv4 using urban adjustment procedures.

4 Statistical Method

4.1 Overview of estimation of QMED

Used	Condition	Approach followed
	N >=30	Estimate QMED using annual maxima
	14=< N =<29	Estimate QMED from annual maxima & optionally adjust for climatic variation
	2=< N= <13	Estimate QMED from POT data & adjust for climatic variation
	N <2 & suitable donor site with 20 years or more of record	Ignore record at subject site; transfer QMED from donor site
	N <2 & suitable donor with 10 to 19 years of record & 12 month overlap between records	Estimate QMED using procedure based on flood peak regression
	N <2 & suitable donor with 10 to 19 years of record but no 12 month overlap	Ignore record at subject site; transfer QMED from donor site
	N <2 & no long-record site nearby	Estimate QMED from very short POT record
	N <2 & no long-record site nearby	Treat site as ungauged catchment
	N <2 & no long-record site nearby	Defer analysis until longer flow record available
	N <2 & no long-record site nearby	(Abstract flood event information and apply the UH rainfall-runoff model as an alternative, to the pooling group procedure. Particularly recommended when site is urbanised)
X	Ungauged catchment	Estimate QMED from catchment descriptors

X	Ungauged catchment	Estimate QMED by data transfer from donor catchment
	Ungauged catchment	Estimate QMED by data transfer from analogue catchment
	Ungauged catchment	Estimate QMED from channel dimensions
	Ungauged catchment	Compare to regional pattern of mapped QMED adjustment factors

Comment on potential donor sites

Mention:

- Number of potential donor sites available
- Distances from subject site
- Similarity in terms of AREA, URBEXT, FARL and other catchment descriptors
- Quality of flood peak data

Include a map if necessary. Note that donor catchments should usually be rural.

River Ter @ Crabbs Bridge (gauge 37003) is located immediately adjacent to the Boreham Brook. The donor site has a larger catchment area (approximately 78km^2) but is essentially rural and has similar catchment characteristics such as SAAR, BFIHOST and SPRHOST to the target site. The donor gauge is a HiFlows gauge which is suitable for pooling and QMED and has been selected as one of the ReFH data set gauges. All of the candidate gauges identified as potential donors were assessed for suitability but were considered less favorable than the River Ter @ Crabbs Bridge gauge, and hence not taken forward.

The Boreham Brook consists of two watercourses, which converge just north of the A12. The north eastern watercourse is referred to as the Brick House tributary and the western watercourse is referred to as the Bulls Lodge tributary. For providing inflows to the hydraulic model, the design flow is required at three locations; the Brick House tributary (FEP01), the Bulls Lodge tributary (FEP02) and at the downstream extent of the model (FEP03). Design peak flows are also required for the area of land south of the A12 know as the residual catchment. The residual catchment is the intervening catchment between the confluence of the Brick House Tributary (FEP01), the Bulls Lodge tributary (FEP02) and the model extent (FEP03). A figure of the Boreham Brook catchment showing the FEP's and residual catchment is provided in Annex A.1.

The CDs and QMED at the three flow estimation points (FEP01, FEP02 and FEP03) and for the residual catchment are presented below.

NAME	Boreham Brook @ Boreham (FEP03 - Target site for pooling)	River Ter @ Crabbs Bridge (Donor)	Brick House Tributary (FEP01)	Bulls Lodge Tributary (FEP02)	Residual catchment ***
Gauge No.		37003			
AREA	14.73	77.6*	8.18	4.87	1.68
SAAR	566	570	565	568	565
FARL	0.962	0.994	0.938	1.000	1.000
SPRHOST	41.77	41.76	39.14	45.92	42.53

BFIHOST	0.529	0.461	0.567	0.470	0.515
Years	-	50	-	-	-
QMEDcd's	1.14	6.51	0.56	0.60	0.27
QMED data	-	5.690**	-	-	-
Ratio(R) $QMED_{data}$/ $QMED_{cds}$	-	0.874	-	-	-

**catchment area revised.*

***additional AMAX data added (2014) and one AMAX value revised (1963) based on data provided by the EA.*

****The catchment descriptors for the residual catchment were derived using FEH methodologies*

4.2 Search for donor sites for QMED

Method	Location	Initial estimate of QMED (m^3/s)	Data transfer				Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	
CD	FEP01	0.56	37003	5.99	0.44	0.943	0.53
CD	FEP02	0.60	37003	6.60	0.43	0.945	0.57
CD	FEP03	1.14	37003	6.32	0.43	0.944	1.08

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added below.

When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why.

The data transfer procedure is the revised one from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

If more than one donor has been used, give the weights used in the averaging.

4.3 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
37003	Suitable gauge with similar key catchment descriptors. The gauge is in close proximity to the target site and has been assessed as suitable for QMED/Pooling. The gauge also has 50 years of data.	AM	No	5.69*	6.51 (QMED urban)	0.874

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
37013	This gauge is also located in close proximity to the target catchment. The gauge has some different catchment characteristics in relation to, BFIHOST and FARL. Due to a large reservoir being present within the catchment (which is likely to impact flow) and due to the differences in permeability of the catchment to the target site this gauge has been rejected for QMED adjustment.					
37010	This gauge has been rejected as it has a much larger catchment than the target site and is noted as being groundwater influenced. The watercourse further receives transfers from the Ouse which may impact QMED. This gauge has also been assessed as having potential problems with its rating by a previous CH2M (2017) study.					

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
37009	This catchment was rejected as a potential donor as the catchment contains urban areas such as Witham and Braintree. It is also a bit further away from the target site.					
37002	Not suitable for QMED/pooling so this gauge has been discounted. Also the watercourse drains a significantly larger catchment.					
37008	This gauge is on a watercourse draining a much larger catchment. The watercourse has a catchment area over x12 the target catchment. Urban area in the lower catchment (Chelmsford). This gauge was therefore rejected as a potential donor.					

**QMED from data has been amended as additional AMAX value added for 2014 and the 1963 AMAX value has been amended based on data received from the Environment Agency.*

4.4 QMED (m^3/s) estimation from catchment descriptors

Location	QMEDcd(rural)	QMEDcd(urban)	68% Confidence Interval*	95% Confidence Interval*
FEP01	0.54	0.56	0.38 / 0.77	0.26 / 1.10
FEP02	0.60	0.60	0.42 / 0.86	0.29 / 1.23
FEP03	1.11	1.14	0.77 / 1.58	0.54 / 2.27

4.4.1 Final estimate of Qmed from FEH Statistical method

Gauge 37003 has been selected as a suitable donor for QMED adjustment for all the Flow Estimation Points (FEPs) on the Boreham Brook. This gauge has been selected due to the close proximity and the similarity of its hydrological characteristics to the target sites.

The following are the QMED estimates from FEH statistical method:

QMED (FEP01) = 0.53m^3/s

QMED (FEP02) = 0.57m^3/s

QMED (FEP03) = 1.08m^3/s

4.5 Derivation of pooling group

Name of saved .feh group file: Boreham Brook Final
Target return period (years) for 5T rule *: T = 100 years

*(Updated recommendations advise that a standard 500 years is used regardless of target return period i.e.T=100yr)

Target site: Boreham Brook at FEP03 (at the extent of the hydraulic model).

For initial and final pooling group sites refer to Annex A.3.

Total number of sites	17
Total number of years	547
Total number of initial highly discordant sites	0
Discordant sites	N/A
Total number of short records (< 7 years) removed	1
List sites removed with reasons	49005 – short record
	26802 / 26803 / 44008 / 33054 – SPRHOST values less than 20. Suggesting these to be

	permeable catchments with substantial groundwater components, whilst the target catchment has a SPRHOST of 41.8% which suggests a moderately impermeable catchment
Number of pooled years after sites removed	435

Is subject site included in pooled group: ~~yes~~ / no

If no state reason why: Target site ungauged

Test statistics on validity of pooling group for flood frequency analysis

Heterogeneity test H2 value = 2.515

Status			
	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable	X	2 < H2 < 4
	Review essential		H2 > 4

Goodness-of-fit test	Z values			Value
	GL	acceptable	/ ~~not acceptable~~	0.582
	GEV	acceptable	/ ~~not acceptable~~	-1.519
	PT3	~~acceptable~~	/ not acceptable	-2.277

other

(Note: in the FEH the GL is the generally favoured distribution for use)

ACTION is construction of flood frequency curve valid?

YES: / NO: review the pooling group further

Comment ?	No; need to add sites for 5T Rule

4.6 Revision of Pooling Group

Revision No. 1

Station Number	Reason for changes in pooling group
53017	Added for 5T Rule
39033	Added for 5T Rule – Note - highly permeable – BFIHOST 0.766*

*see Annex A.4 for further details

Number of sites 14 Years 530

Heterogeneity test H2 value = 3.363

Status			
	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable	/	2 < H2 < 4
	Review essential		H2 > 4

Note: FEH Vol.3, chapter 16.3.2: “The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review.”

					Value
Goodness-of-fit test	Z values	GL	acceptable	/ ~~not acceptable~~	0.222
		GEV	~~acceptable~~	/ not acceptable	-1.750
		PT3	~~acceptable~~	/ not acceptable	-2.753
	other				

ACTION Is construction of flood frequency curve valid?

~~No~~ Yes

Comment?	Pooled group has been reviewed, therefore proceed to construction of FFC.

Derivation of flood growth curves

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 100-year return period**
P	GL – L-median GEV – L-median		2.803 2.672
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

Refer to Annex A.2 to A4 for pooling group details.

4.7 Flood estimates from the statistical method

GL	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	30	50	75	100	200	1000
Growth Factor	1.000	1.404	1.689	1.989	2.176	2.428	2.642	2.803	3.223	4.406
FEP03	1.08	1.51	1.82	2.15	2.35	2.62	2.85	3.02	3.48	4.75
FEP01*	0.53	0.74	0.89	1.05	1.14	1.28	1.39	1.47	1.69	2.32
FEP02*	0.57	0.79	0.95	1.12	1.23	1.37	1.49	1.58	1.82	2.49
Residual* catchment	0.25	0.35	0.43	0.50	0.55	0.61	0.67	0.71	0.81	1.11

**Given the catchment descriptors at FEP01 and FEP02 were similar to FEP03 (the model extent) the FEH Statistical growth curve has been assessed as appropriate to use to derive design peak flow estimates at these locations. The growth curve has also been assessed as appropriate to use for the residual catchment. Derived design peak flows for FEP01, FEP02 and the residual catchment are therefore included in the table above.*

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	30	50	75	100	200	1000
Growth Factor	1.000	1.447	1.744	2.028	2.192	2.396	2.558	2.672	2.948	3.586
FEP03	1.08	1.56	1.88	2.19	2.36	2.58	2.76	2.88	3.18	3.87
FEP01*	0.53	0.76	0.92	1.07	1.15	1.26	1.35	1.41	1.55	1.89
FEP02*	0.57	0.82	0.99	1.15	1.24	1.35	1.45	1.51	1.67	2.03
Residual catchment*	0.25	0.37	0.44	0.51	0.55	0.61	0.65	0.68	0.75	0.91

**Given the catchment descriptors at FEP01 and FEP02 were similar to FEP03 (the model extent) the FEH Statistical growth curve has been assessed as appropriate to use to derive design peak flow estimates at these locations. The growth curve has also been assessed as appropriate to use for the residual catchment. Derived design peak flows for FEP01, FEP02 and the residual catchment are therefore included in the table above.*

A sensitivity test was also carried out with gauging station 39033 removed as SPR just above threshold (SPRHOST: 22.35) and BFIHOST indicates highly permeable catchment. See Annex A.4 for further details. Removing this gauge from the pooling group was not assessed as having a significant impact to the design peak flow estimates.

Refer to Annex A.5 for plots of the Boreham Brook @ Boreham (FEP03) FEH Statistical growth curve.

5 Revitalised Flood Hydrograph (ReFH2.2) Model

5.1 Parameters for ReFH2.2 model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible so it is not essential to enter them in the table.

Location	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
Boreham Brook @ model extent (FEP03)	CD – ReFH2.2	6.83	449.09	49.6	1.27
Boreham Brook @ Bricks House Tributary (FEP01)	CD – ReFH2.2	6.41	495.68	49.61	1.37
Boreham Brook @ Bulls Lodge Tributary (FEP02)	CD – ReFH2.2	4.38	385.28	39.51	1.11
Residual Catchment	CD – ReFH2.2	3.16	433.05	37.69	1.23
Brief description of any flood event analysis carried out (further details should be given below or in a project report)	No flood event analysis carried out.				

5.2 Design events for calculating peak flow using ReFH2.2 method

Location	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
Boreham Brook @ model extent (FEP03)	Rural	Winter	11.1hrs	Catchment area
Boreham Brook @ Bricks House Tributary (FEP01)	Rural	Winter	11.1hrs	Catchment area
Boreham Brook @ Bulls Lodge	Rural	Winter	6.5hrs	Catchment area

Location	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
Tributary (FEP02)				
Residual catchment	Urban	Summer	4.5hrs	Catchment area

5.3 Peak flood estimates from the ReFH2.2 method (based upon the sub-catchment specific critical storm durations)

Using ReFH2.2 Duration	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	30	50	75	100	200	1000
Model Extent (FEP03)	1.92	2.63	3.12	3.63	3.96	4.42	4.87	5.27	6.61	10.91
Brick House Tributary (FEP01)	0.96	1.32	1.56	1.82	1.98	2.22	2.45	2.65	3.32	5.54
Bulls Lodge Tributary (FEP02)	1.08	1.50	1.78	2.09	2.28	2.55	2.82	3.05	3.82	6.29
Residual Catchment	0.66	0.96	1.17	1.38	1.51	1.70	1.88	2.02	2.49	4.20

6 Discussion and Summary of Results

6.1 Comparison of results from different methods

This table compares peak flows from the two methods for two key return periods.

	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	FEH Statistical	ReFH2.2	Ratio	FEH Statistical	ReFH2.2	Ratio
FEP03	1.08	1.92	**1.78**	3.02	5.27	**1.74**
FEP01	0.53	0.96	**1.81**	1.47	2.65	**1.80**
FEP02	0.57	1.08	**1.89**	1.58	3.05	**1.93**
Residual catchment	0.25	0.66	**2.64**	0.71	2.02	**2.85**

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	ReFH2.2 produces much higher peak flows than the FEH Statistical method for the Boreham Brook catchment. Application of ReFH2.2 on the donor River Ter @ Crabbs Bridge gauge (see companion analysis for the River Ter) suggests that the ReFH2.2 method overestimated the commoner return period flows. However, the ratio of ReFH2.2 to statistical shown above is greater than the equivalent for the River Ter. It was therefore considered prudent to give some weight to the ReFH2.2 estimates and the average of the FEH Statistical and ReFH2.2 flows was used to provide the final flows. Hydrograph shapes are derived from the application of ReFH2.2. For use in the hydraulic modelling the final design peak flows estimated at FEP01 and FEP02 will be divided into two inflows each (one as point inflow to be applied at the top of the model and the other as a lateral flow to be applied between top of the model and the flow estimation point), proportionate to the contributing catchment area. For the residual catchment south of the confluence of the two tributaries (i.e., south of FEP01 / FEP02), the final design peak flows have been derived as the average of the peak flows from the FEH and the ReFH2.2 methods and applied as lateral inflows into the hydraulic model. The final design peak flows at FEP03 are suitable to use as a target flow to reconcile the modelled flow at the model downstream extent obtained using the inflows at FEP01, FEP02 and the residual catchment. It should be noted that the Boreham Brook hydrological analysis was originally undertaken in 2017 but the project was subsequently put on hold. The Boreham Brook 2017 hydrology adopts a conservative approach by adopting the average of the FEH Statistical and ReFH2.2 peak flow estimates. A review of the Boreham Brook 2017 study undertaken in February 2021 using the latest WINFAP software and Version 9 of the peak flow dataset showed there was only a slight increase in the growth factor. For the 1% AEP (100-year return period)

	event it results in approximately 4% increase in peak flow estimates. The final peak flow estimates are derived from an average of the FEH Statistical and ReFH2.2 peak flow estimates so the overall increase in 1% AEP (100-year return period) event peak flow would be in the range of only 2%. It was therefore assessed that updates to the hydrology would be likely to only result in minor adjustments and therefore the 2017 hydrology was assessed as appropriate to take forward for the purposes of the A12 scheme.

6.3 Assumptions, limitations and uncertainty

<table>
<tr><td>List the main assumptions made (specific to this study)</td><td>The main assumptions used in this study are as follows:
Most of the sand and gravel pits within the catchment do not have a significant impact on the flood flows as they appear to be disconnected from the river. It is assumed that this is also true during flood events. If this were not true then it is likely that the storage afforded by the gravel pits would be likely to attenuate the forward flood and lessen the design flood in the vicinity of the A12 crossing;
ReFH2.2 is likely to be overestimating design peak flows for the target catchment (based on results of analysis at a nearby gauge (River Ter @ Crabbs Bridge – 37003)).
The 2017 Hydrology is appropriate to use for this assessment.</td></tr>
<tr><td>Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed</td><td>Despite the offline nature of most of the gravel pits the catchment is given a FARL of 0.962. This suggests that some water body attenuation should be expected. The influence of waterbodies is approximately taken into account in the FEH pooling group estimates but not in the ReFH2.2 method. This may in part explain the higher estimates from the ReFH2.2 method.</td></tr>
<tr><td>Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).</td><td>QMED has been derived at FEP03 from catchment descriptors and adjusted using the River Ter @ Crabbs Bridge $QMED_{obs}/QMED_{cds}$ adjustment ratio using the revised donor procedure of Kjeldsen et al. 2014.
<table>
<tr><th>Boreham Brook (FEP03)</th><th>Lower limit (m^3/s)</th><th>Upper Limit (m^3/s)</th></tr>
<tr><td>68%</td><td>0.75</td><td>1.55</td></tr>
<tr><td>95%</td><td>0.53</td><td>2.21</td></tr>
</table></td></tr>
<tr><td>Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.</td><td>The design peak flow estimates have been derived specifically for the Boreham Brook for the A12 model. Before the flows are used in future studies there should be an assessment of their suitability for the study in</td></tr>
</table>

	question. Flow derivation methods change in time and this should be taken into consideration.
Give any other comments on the study, for example suggestions for additional work.	Additional work could be undertaken to analyse the impact of the sand and gravel pits within the catchment on design peak flows. Further analysis could also be undertaken regarding the large differences in flow estimates from FEH Statistical and ReFH 2.2 methods.

6.4 Checks

What do the results imply regarding the return periods of floods during the period of record?	There are no gauging stations on the Boreham Brook. Analysis of return periods of previous floods was therefore not undertaken for this study.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	The 1% AEP (100-year return period) growth factor for the average of the FEH Statistical and ReFH2.2 method at FEP03 is 2.76. The growth factor is within the expected range.
If 1000-year flows have been derived, what is the ratio for 1000-year flow over 100-year flow?	The ratio of the 0.1% AEP (1000-year return period) flow to the 1% AEP (100-year return period) flow at FEP03 is 1.89.
What specific runoff (l/s/ha) does the design flow equate to?	The corresponding 1% AEP (100-year) and 50% (2-year) specific runoffs are 2.81l/s/ha and 1.02l/s/ha respectively.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	At the time of assessment, no other studies on the Boreham Brook were available for comparison. A study on the River Ter which is a neighboring watercourse to the Boreham Brook (and used in this assessment as a donor for QMED adjustment) derived a 1% AEP (100-year return period) flow which equated to 1.80l/s/ha. The difference in the runoff could be attributed to slightly greater urbanisation in the Boreham Brook catchment / slightly less attenuation of flows in the floodplain but such differences are believed to be marginal. Using the average of the ReFH2.2 and the statistical may have resulted in the higher specific rates and it may be that the flows estimated for Boreham Brook are conservatively large. However, there is no gauge on the Boreham Brook to give firm guidance as to whether the ReFH2.2 is over predicting on this catchment and it was thought prudent to give some weight to the ReFH2.2 estimates.
Are the results compatible with the longer-term flood history?	Information on the longer-term flood history was not available at the time of assessment.

Describe any other checks on the results	No further checks on the results carried out.

6.5 Final results

Flood peak (m^3/s) for the following return periods (in years) – taken as the average of FEH Statistical and ReFH2.2 methods											
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**100+38 % CC Uplift**	**200**	**1000**
FEP03	1.50	2.07	2.47	2.89	3.15	3.52	3.86	4.14	5.72	5.04	7.83
FEP01	0.74	1.03	1.23	1.43	1.56	1.75	1.92	2.06	2.84	2.51	3.93
FEP02	0.82	1.14	1.37	1.60	1.75	1.96	2.16	2.32	3.20	2.82	4.39
Residual catchment	0.46	0.66	0.80	0.94	1.03	1.16	1.27	1.37	1.88	1.65	2.66

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	Design peak flows and flood hydrographs have been derived for the target locations for input into the Boreham Brook hydraulic model. The 50% AEP (2-year return period) and 1% AEP (100-year return period) hydrographs are presented in Annex A.6. The full range of flood hydrographs are saved at the following location: A12 Hydrological Calculation Sheet_Stage III_Boreham Brook_Rev2 Checked It should be noted that the peak flows for the individual critical storm duration was used in the hydraulic model inflow but the hydrograph shape is based on the 11.1hr duration storm event, which is equivalent to the storm duration at the downstream modelling extent (FEP03). Plus that the river model peak flows at FEP03 needed to be reconciled with the flows given in the table above.

Acronyms

Abbreviation	Term
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent

Abbreviation	Term
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

References

Environment Agency (2017). Flood Estimation Guidelines. Technical Guidance 197_08.

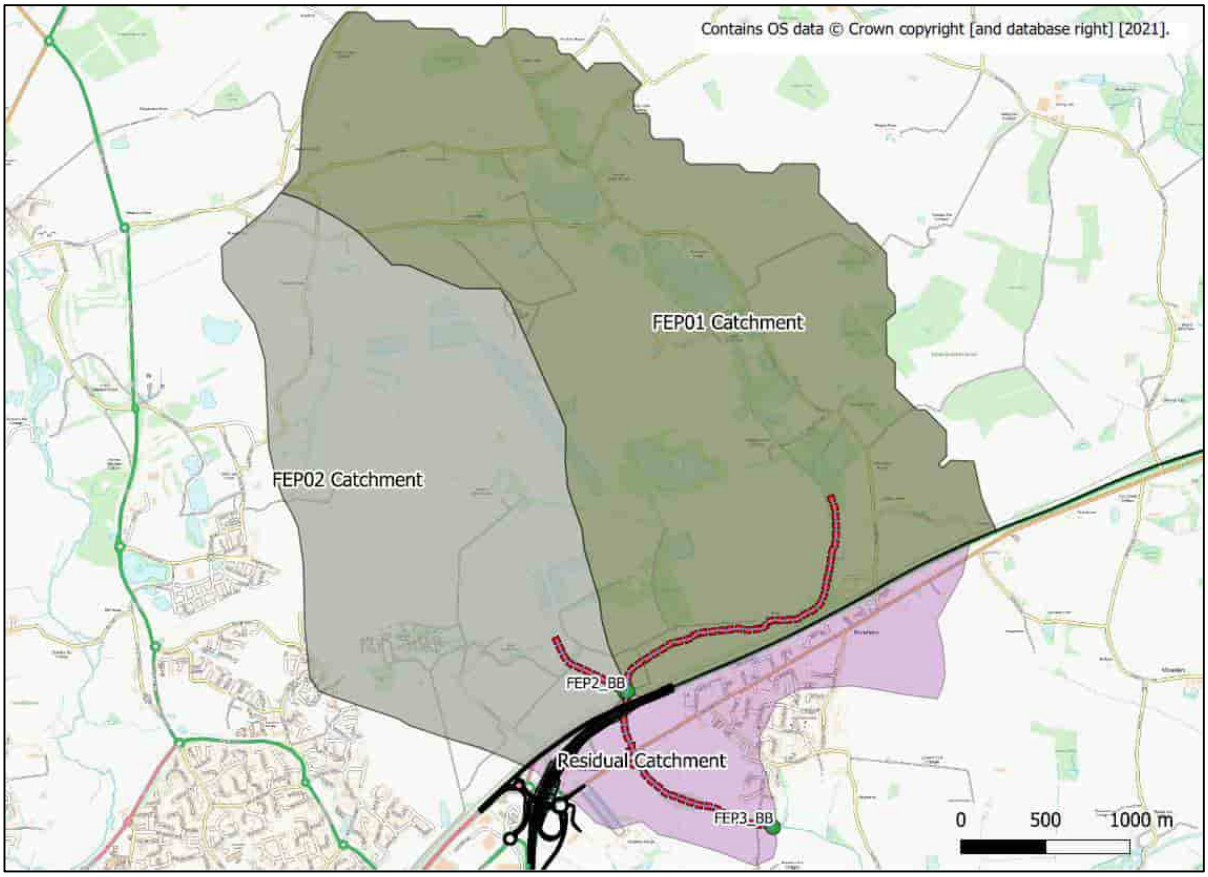
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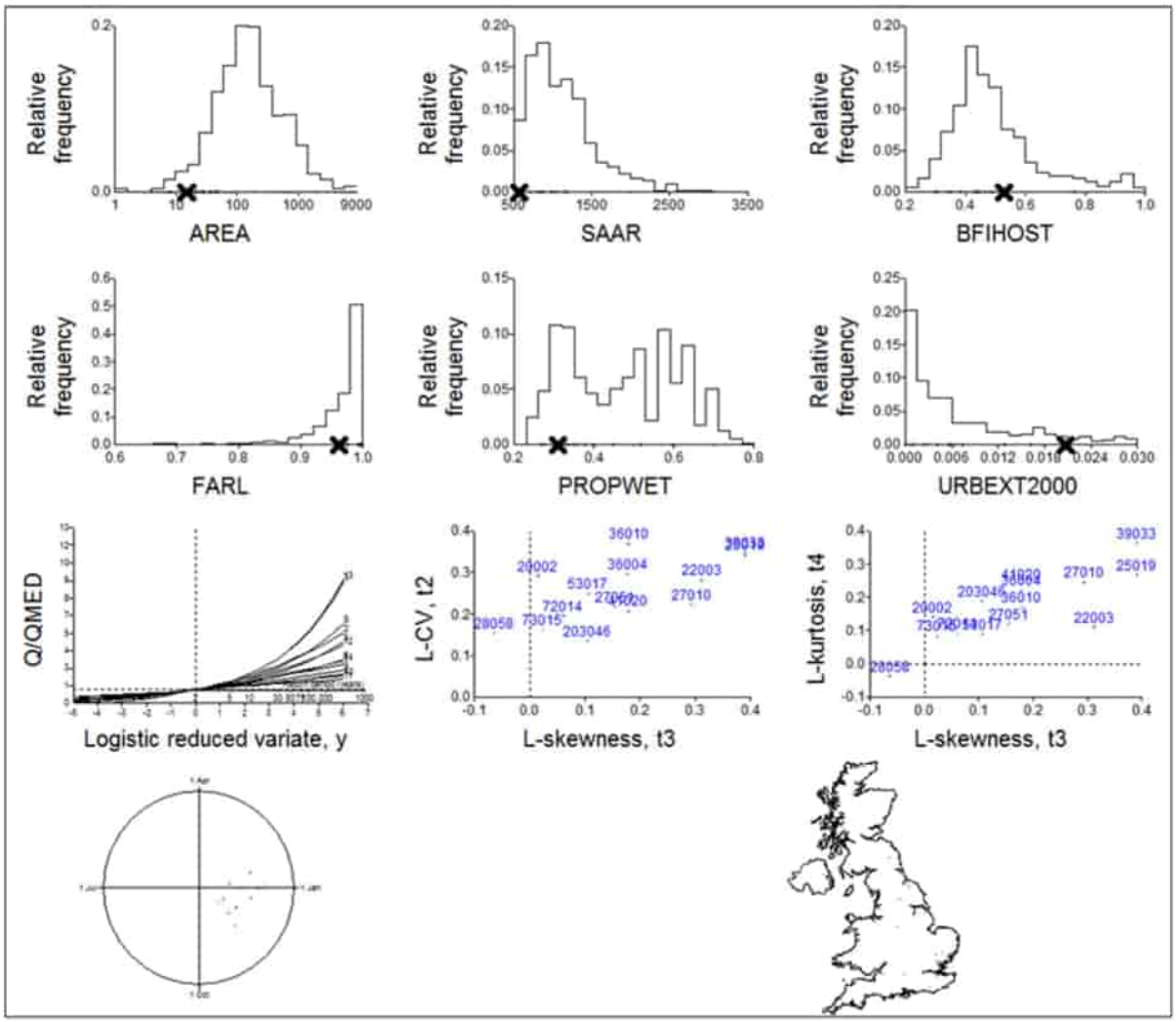
Soil Survey of England and Wales (1983). Soil Maps of England and Wales Scale 1:250,000

Annex A

A.1 Location of Catchments



A.2 Pooling Group Details - Graphs



A.3 Pooling Group Details – Table

A.3.1 Initial Pooling Group Details: (show default selection) at FEP03

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
26802 (Gypsey Race @ Kirby Grindalythe)	0.895	16	0.112	0.274	0.274	0.503
20002 (West Peffer Burn @ Luffness)	0.917	41	3.299	0.292	0.015	1.925
36010 (Bumpstead Brook @ Broad Green)	1.035	48	7.545	0.37	0.178	1.33
25019 (Leven @ Easby)	1.092	37	4.989	0.342	0.39	1.102
203046 (Rathmore Burn @ Rathmore Bridge)	1.344	33	10.77	0.136	0.104	1.176
49005 (Bollingey Stream @ Bolingey Cocks Bridge)	1.4	5	5.777	0.288	0.256	0.714
27051 (Crimple @ Burn Bridge)	1.438	43	4.514	0.219	0.154	0.189
27010 (Hodge Beck @ Bransdale Weir)	1.446	41	9.42	0.224	0.293	1.041
26803 (Water Forlornes @ Driffield)	1.467	16	0.418	0.31	0.145	1.387
44008 (South Winterbourne @ Winterbourne Steepleton)	1.471	36	0.434	0.418	0.344	1.515
41020 (Bevern Stream @ Clappers Bridge)	1.517	46	13.78	0.208	0.178	0.765
22003 (Usway Burn @ Shillmoor)	1.623	13	16.17	0.282	0.311	1.452
72014 (Conder @ Galgate)	1.701	48	17.595	0.196	0.06	0.318
73015 (Keer @ High Keer Weir)	1.702	25	12.135	0.162	0.022	0.627
28058 (Henmore Brook @ Ashbourne)	1.734	12	9.006	0.155	-0.064	1.943
33054 (Babingley @ Castle Rising)	1.742	39	1.129	0.209	0.069	0.363
36004 (Chad Brook @ Long Melford)	1.756	48	5.293	0.296	0.177	0.649
Total		547				
Weighted means				0.26	0.169	

A.3.2 Initial Pooling Group Catchment Descriptors: (show default selection) at FEP03

Station	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
26802 (Gypsey Race @ Kirby Grindalythe)	0.895	15.85	757	0.03	1	0
20002 (West Peffer Burn @ Luffness)	0.917	25.51	616	0.127	0.995	0.002
36010 (Bumpstead Brook @ Broad Green)	1.035	27.58	588	0.045	0.999	0.007
25019 (Leven @ Easby)	1.092	15.09	830	0.019	1	0.004
203046 (Rathmore Burn @ Rathmore Bridge)	1.344	22.5	1043	0.072	1	0
49005 (Bollingey Stream @ Bolingey Cocks Bridge)	1.4	16.08	1044	0.023	0.991	0.006
27051 (Crimple @ Burn Bridge)	1.438	8.17	855	0.013	1	0.006
27010 (Hodge Beck @ Bransdale Weir)	1.446	18.82	987	0.009	1	0.001
26803 (Water Forlornes @ Driffield)	1.467	32.42	721	0.016	1	0.007
44008 (South Winterbourne @ Winterbourne Steepleton)	1.471	20.18	1012	0.015	1	0.004
41020 (Bevern Stream @ Clappers Bridge)	1.517	35.48	886	0.076	0.993	0.013
22003 (Usway Burn @ Shillmoor)	1.623	21.88	1056	0.006	1	0
72014 (Conder @ Galgate)	1.701	28.99	1183	0.082	0.975	0.006
73015 (Keer @ High Keer Weir)	1.702	30.04	1158	0.074	0.976	0.003
28058 (Henmore Brook @ Ashbourne)	1.734	38.52	895	0.03	0.977	0.021
33054 (Babingley @ Castle Rising)	1.742	48.53	686	0.118	0.944	0.005
36004 (Chad Brook @ Long Melford)	1.756	50.33	589	0.065	1	0.006

A.3.3 Final Pooling Group Details at FEP03

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
20002 (West Peffer Burn @ Luffness)	0.917	41	3.299	0.292	0.015	1.663
36010 (Bumpstead Brook @ Broad Green)	1.035	48	7.545	0.37	0.178	1.473
25019 (Leven @ Easby)	1.092	37	4.989	0.342	0.39	1.026
203046 (Rathmore Burn @ Rathmore Bridge)	1.344	33	10.77	0.136	0.104	1.42
27051 (Crimple @ Burn Bridge)	1.438	43	4.514	0.219	0.154	0.203
27010 (Hodge Beck @ Bransdale Weir)	1.446	41	9.42	0.224	0.293	0.817
41020 (Bevern Stream @ Clappers Bridge)	1.517	46	13.78	0.208	0.178	0.735
22003 (Usway Burn @ Shillmoor)	1.623	13	16.17	0.282	0.311	2.23
72014 (Conder @ Galgate)	1.701	48	17.595	0.196	0.06	0.242
73015 (Keer @ High Keer Weir)	1.702	25	12.135	0.162	0.022	0.53
28058 (Henmore Brook @ Ashbourne)	1.734	12	9.006	0.155	-0.064	1.472
36004 (Chad Brook @ Long Melford)	1.756	48	5.293	0.296	0.177	0.43
39033 (Winterbourne Stream @ Bagnor)	1.767	53	0.399	0.348	0.389	1.456
53017 (Boyd @ Bitton)	1.839	42	13.446	0.25	0.106	0.302
Total		530				
Weighted means				0.252	0.17	

A.3.4 Final Pooling Group Catchment Descriptors at FEP03

Station	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
20002 (West Peffer Burn @ Luffness)	0.917	25.51	616	0.127	0.995	0.002
36010 (Bumpstead Brook @ Broad Green)	1.035	27.58	588	0.045	0.999	0.007
25019 (Leven @ Easby)	1.092	15.09	830	0.019	1	0.004
203046 (Rathmore Burn @ Rathmore Bridge)	1.344	22.5	1043	0.072	1	0
27051 (Crimple @ Burn Bridge)	1.438	8.17	855	0.013	1	0.006
27010 (Hodge Beck @ Bransdale Weir)	1.446	18.82	987	0.009	1	0.001
41020 (Bevern Stream @ Clappers Bridge)	1.517	35.48	886	0.076	0.993	0.013
22003 (Usway Burn @ Shillmoor)	1.623	21.88	1056	0.006	1	0
72014 (Conder @ Galgate)	1.701	28.99	1183	0.082	0.975	0.006
73015 (Keer @ High Keer Weir)	1.702	30.04	1158	0.074	0.976	0.003
28058 (Henmore Brook @ Ashbourne)	1.734	38.52	895	0.03	0.977	0.021
36004 (Chad Brook @ Long Melford)	1.756	50.33	589	0.065	1	0.006
39033 (Winterbourne Stream @ Bagnor)	1.767	45.31	717	0.033	1	0.001
53017 (Boyd @ Bitton)	1.839	47.58	807	0.05	0.998	0.016

A.4 Sensitivity Test on Growth Curve to the Removal of 39033

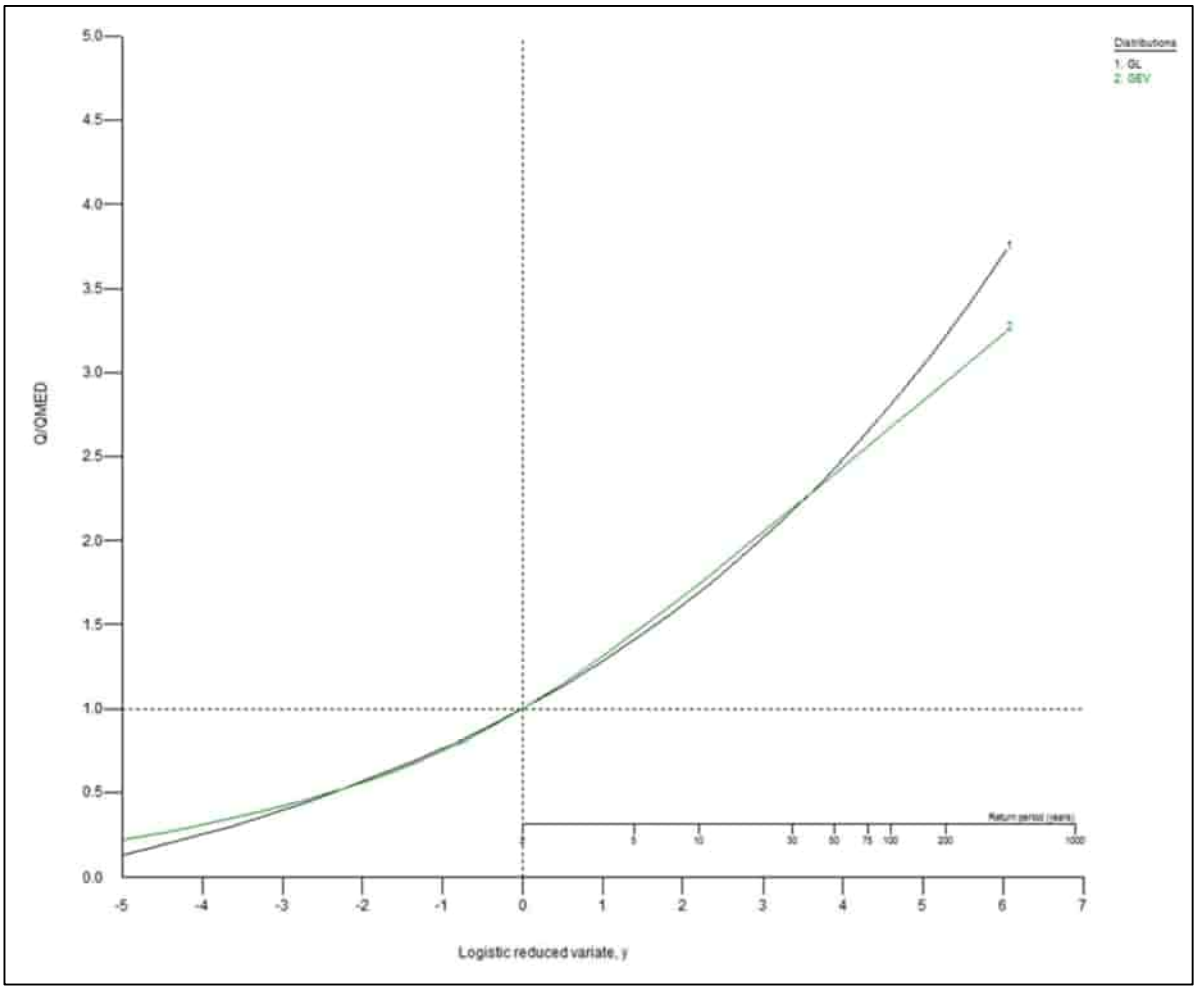
A.4.1 39033 was removed as a test as SPRHOST = 22.35 and BFIHOST = 0.766 suggesting it is a permeable catchment.

A.4.2 Station 24007 and 47022 added as a substitute to meet the 5T Rule. Station 47022 has a FARL of 0.942 which was accepted given the target site has a FARL of 0.96.

Return Period	Fittings	Peak Flow (m^3/s)
2	1.000	1.08
5	1.387	1.50
10	1.654	1.78
30	2.102	2.27
50	2.329	2.51
75	2.521	2.72
100	2.664	2.87
200	3.034	3.27
1000	4.053	4.37

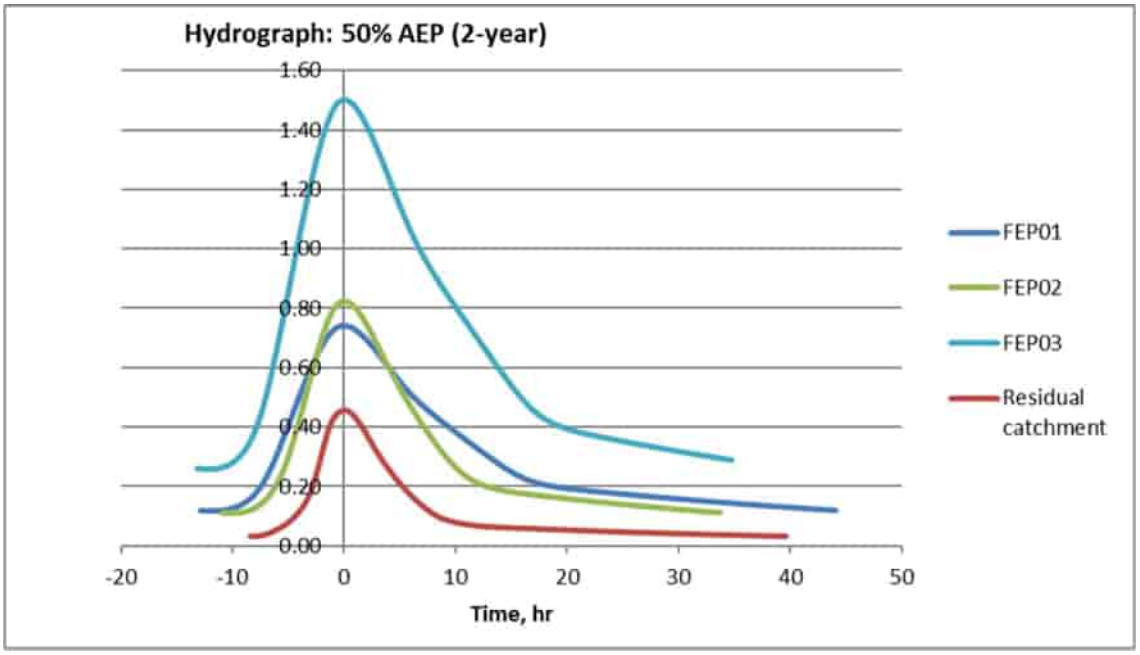
A.4.3 Adding station 24007 and 47022 instead of 39033 makes minimal difference to the pooling group with slightly lower flows being estimated. Based on this assessment it was decided to leave the pooling group with 39033 being one of the lower gauges in the group.

A.5 Downstream modelling extent (FEP03) Growth Curve

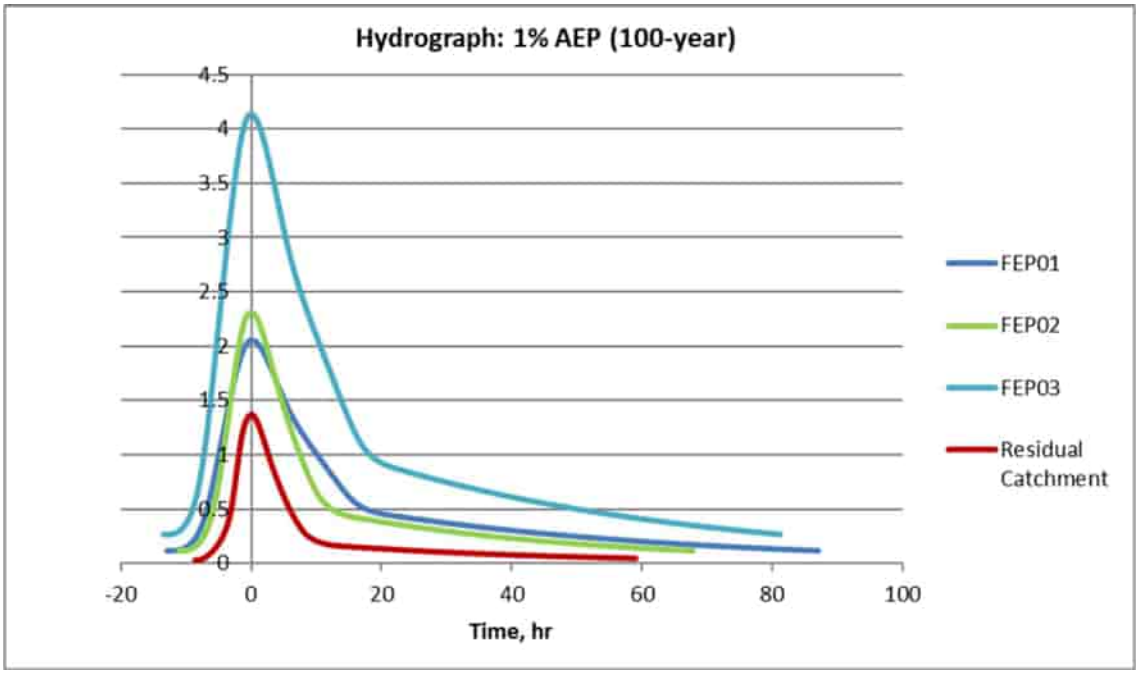


A.6 50% AEP (2-year RP) and 1% AEP (100-year RP) Hydrographs

A.6.1 Boreham Brook 50% AEP (2-year return period) event Hydrograph



A.6.2 Boreham Brook 1% AEP (100-year return period) Hydrograph



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVER TER MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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0 Executive Summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the A12 highway crossing at River Ter. The modelling and hydrology have been carried out in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1D-2D linked type using Flood Modeller software (Jacobs 2022) for the 1D domain and TUFLOW Classic (WBM 2022) for the 2D domain and was built from scratch.

0.1.2 Three events were simulated: the 5%, 1% and 1% AEP with 65% climate change. Flow reconciliation was carried out to match the design hydrology. No data was available for calibration, but the model was verified using Environment Agency (EA) flood maps. The model was then used to simulate baseline three design events. Sensitivity analysis to roughness, inflows and the downstream boundaries was carried out to further validate the results.

0.1.3 The with-scheme scenario involved the upgrade of the highway to three lanes per carriageway, without any adjustment of the external bridge structure or any change to the highway embankment at this location.

0.1.4 The modelled flood extents do not intersect with any proposed scheme elements, therefore there is no requirement to carry out any modelling for the proposed scheme scenario.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme is looking at improving the A12 between junction 19 (Boreham) and junction 25 (Marks Tay). The proposed scheme includes widening the A12 to three lanes throughout the length plus associated works to junctions and side roads.

1.1.2 For key watercourse crossings, a Flood Risk Assessment (FRA) is required to meet relevant local and national planning legislation and inform the design and planning process. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic models with associated catchment hydrology. Water levels for the baseline and with scheme scenarios both upstream and downstream of the proposed scheme and the associated flood extents were determined for a range of storm flood events at each watercourse crossing.

1.1.3 The key watercourse crossings for which hydraulic modelling was carried out to support the FRA are:

- Boreham Brook
- River Ter (this report)
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Five minor watercourses were also modelled for the proposed scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for the River Ter, to assess the baseline situation and the with scheme scenario. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 It is noted that for the A12 crossing of the River Ter, the highway will be upgraded to three lanes per carriageway without any adjustment of the external bridge structure or any change to the highway embankment at this location. Consequently, no with scheme modelling was undertaken and hence no discussion of it appears in this report.

1.1.6 This report should be read in conjunction with the Flood Risk Assessment report (Jacobs 2022).

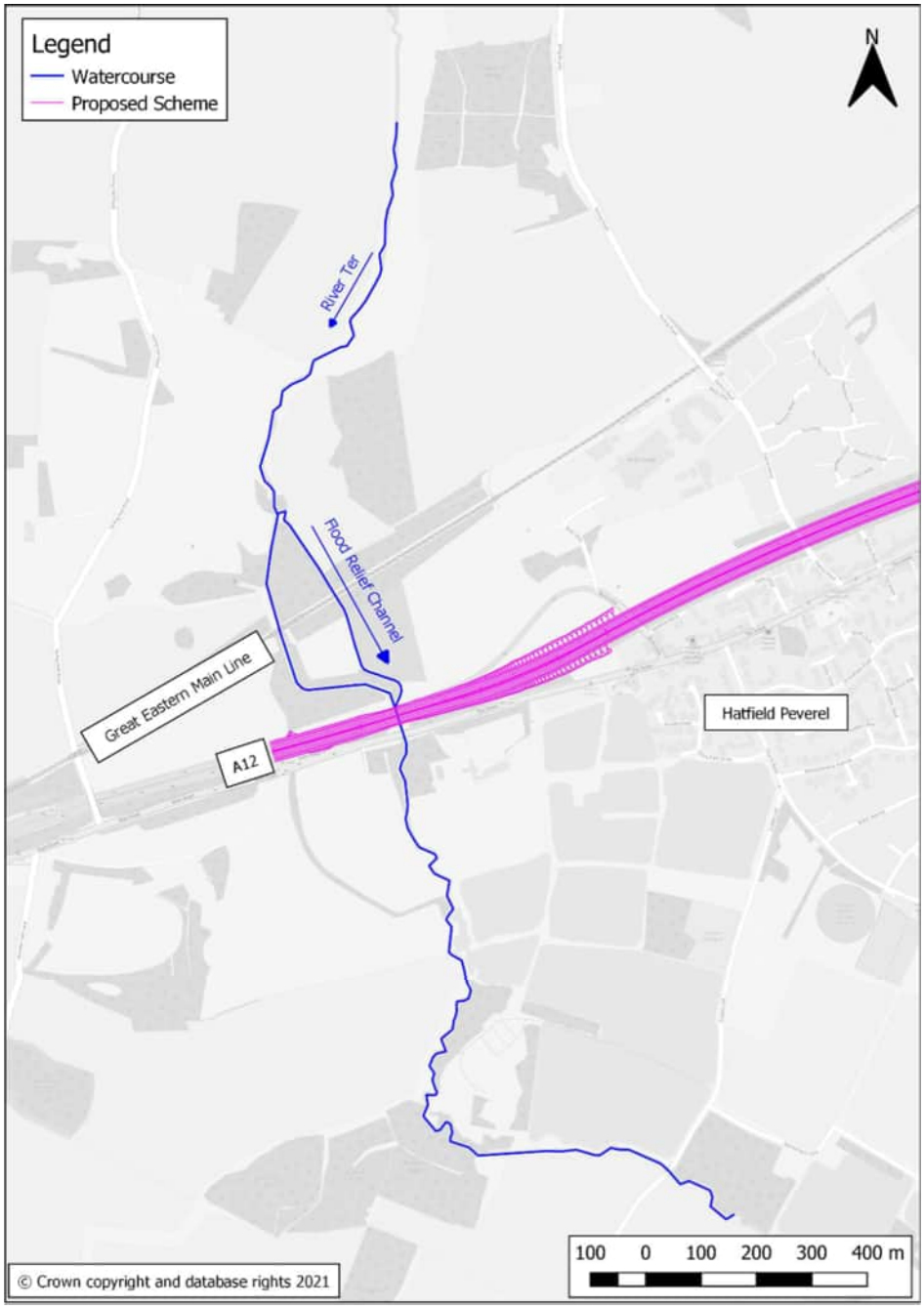
1.2 Methodology

1.2.1 The hydraulic model uses a linked One-dimensional/Two-dimensional (1D/2D) schematisation, where the river channel is represented as a 1D component and is linked to the floodplain, which is represented by a 2D domain. The 1D component was constructed using the river modelling package Flood Modeller Pro version 4.5 (Jacobs, 2020), and the 2D component was constructed using TUFLOW (version 2018-03-AE-iSP-w64).

1.3 Study Area

1.3.1 The River Ter flows to the west of Hatfield Peverel in Essex, flowing from north to south under the A12 (Plate 1.1). To the north of the A12, there is a 450m long flood relief channel which runs parallel to the River Ter. The model includes 1.3km of River Ter upstream of the A12 and 1.3km downstream.

Plate 1.1 River Ter study area



2 Input Data

2.1.1 The data used to construct the hydraulic model for the River Ter Model is summarised in Table 2.1.

Table 2.1 Data Used to Build the Hydraulic Model

Data	Description	Source
LiDAR	2m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) data. Used to represent the topography for the whole model extent	Environment Agency 2017
OS maps	Mastermap data	Highways England
Channel survey	In-channel cross sections and hydraulic structures	Jacobs August 2017
Hydrological analysis	Hydrological analysis carried out for River Ter	Jacobs 2017

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of the River Ter for the 5% and 1% Annual Exceedance Probability (AEP) events (equivalent to the 20 and 100-year return periods). The 1% AEP event is also required including an allowance for climate change (plus CC). For full details of the flow derivation methods and background hydrology refer to Appendix A.

3.2 Hydrological Considerations

3.2.1 The River Ter catchment (77.6km^2) is a lowland catchment located in the south east of England. The catchment is classified as essentially rural with small urban areas. The River Ter has a number of waterbodies within its catchment; however, these waterbodies are generally offline and are therefore likely to provide only minor attenuation of flows, as suggested by a FARL value close to 1. The geology of the catchment has been assessed as being predominantly London Clay overlain with superficial deposits of mainly boulder clay and some sand and gravel. A small area in the upper catchment has been classified as being of generally high permeability superficial deposits with the remainder of the catchment being classified as being of mixed permeability superficial deposits. The catchment to the River Ter @ Crabbs Bridge (37003) gauge (which is in the model extent) has a BFIHOST of 0.46 and a SPRHOST of 41.8.

3.3 Peak Flow Derivation

3.3.1 The Environment Agency River Ter gauging station at Crabbs Bridge (37003) has a flood record from 1964 to present. At the time of assessment (2017) its flood record was classified in the Peak Flows database as suitable for both QMED estimation and pooling group analyses. Enhanced single-site (ESS) statistical analysis was used on the gauged data to derive peak flows for up to and including the 1% AEP (100-year) event.

3.3.2 For consideration of rarer events the Environment Agency (EA) guidance was followed which indicates that these flows can be estimated by multiplying the FEH statistical 1% AEP (100-year) flow by the ratio of the ReFH2.2 (ReFH2.2 was the available ReFH software at the time of the original 2017 hydrology assessment) derived T-year peak flow, QT (for T>100-years) and the ReFH2.2 derived 1% AEP (100-year) flow, Q100.

3.3.3 At other flow estimation points the AEP flows were derived using the following approach:

- QMEDs were derived using catchment descriptors (CDs); and adjusted using the River Ter @ Crabbs Bridge gauge as the donor. These estimates were extended to rarer events up to and including the 1% AEP event by applying the same flood growth curve as that estimated by the ESS analysis at the gauge.

- The ratio of the ReFH2.2 estimates of the QT/Q100 was again used to extend the flood frequency curve from the 1% AEP event to rarer events.

3.3.4 The peak flow estimates at three locations (FEP01, FEP02 and FEP03 [the location of the Crabbs Bridge gauge]) along the River Ter are presented in Table 3 1: River Ter peak flows as a function of event rarity at the three flow estimation points.

Table 3.1 River Ter peak flows as a function of event rarity at the three flow estimation points.

Inflow	Peak Flow (m^3/s)										
	50% AEP (2-year)	20% AEP (5-year)	10% AEP (10-year)	5% AEP (20-year)	3.33% AEP (30-year)	2% AEP (50-year)	1.33% AEP (75-year)	1% AEP (100-year)	1% AEP +65% CC Uplift (100-year) **	0.5% AEP (200-year)	0.1% AEP (1000-year)
FEP01	5.4	7.5	8.9	10.2	10.9	11.9	12.7	13.2	21.8	16.3	26.3
FEP02	5.6	7.9	9.3	10.6	11.4	12.4	13.2	13.8	22.7	17.0	27.4
FEP03*	5.7	8.0	9.4	10.8	11.6	12.6	13.4	14.0	23.1	17.2	27.8

**The AEP flows at FEP03 are used to reconcile the flood model flows*

***For details regarding the climate change uplift refer to Section 3.5*

Event return period presented for reference in brackets.

3.4 Inflow Hydrograph Shape

3.4.1 A typical flood hydrograph shape was obtained from an analysis of the gauged data. 15-minute flow data was available from 1979 to 2017. The five largest flood events from this period were compared and the hydrograph shape from the 21st October 2001 flood event was selected as being typical. This was used to derive the hydrograph shape of all the inflows of the River Ter sub-catchments feeding into the hydraulic model of the reach.

3.5 Climate Change

3.5.1 A climate change allowance for peak river flows was obtained from the latest government guidance at the time of assessment (Environment Agency, 2017b). This produced an uplift allowance of 65% (the Anglian region upper limit, 2080s scenario) which was applied to the 1% AEP (100-year) design event giving a peak flow of 23.1m^3/s at the River Ter at Crabbs Bridge gauge. It should be noted that adopting the latest Environment Agency climate change guidance uplift (EA, 2021) would result in a lower climate change uplift of 38% (Combined Essex Management Catchment peak river flow allowance, higher 2080s scenario). Given the proposed scheme is outwith the 1% AEP (100-year) event plus 65% uplift flood extent no revision has been made to the climate change uplift.

3.6 Inflows into the River Ter hydraulic model

3.6.1 The River Ter hydraulic model requires design hydrographs at five inflow locations; one inflow at the top of the model and four lateral (residual) inflows as described below:

- Inflow 1 – at FEP01 (the upstream extent of the hydraulic model);
- Residual 1a – lateral inflow from the residual catchment between FEP01 and the railway;
- Residual 1b – lateral inflow from the residual catchment between the railway and the A12;
- Residual 2 – lateral inflow from the residual catchment between the A12 and the River Ter @ Crabbs Bridge gauging station (37003);
- Residual 3 – lateral inflow from the residual catchment between gauging station 37003 and the downstream model extent.

3.6.2 A map showing the River Ter hydraulic model extent and inflow locations is provided in Plate 3.1.

3.6.3 The inflow hydrographs were produced by scaling the typical hydrograph shape (as described above) to the estimated peak flows given in Table 3.2. For example: Plate 3.2 presents the inflow hydrographs for Inflow 1 used for the 5% and 1% AEP (20-year and 100-year) events.

Plate 3.1 Model Extent and Inflow Locations.

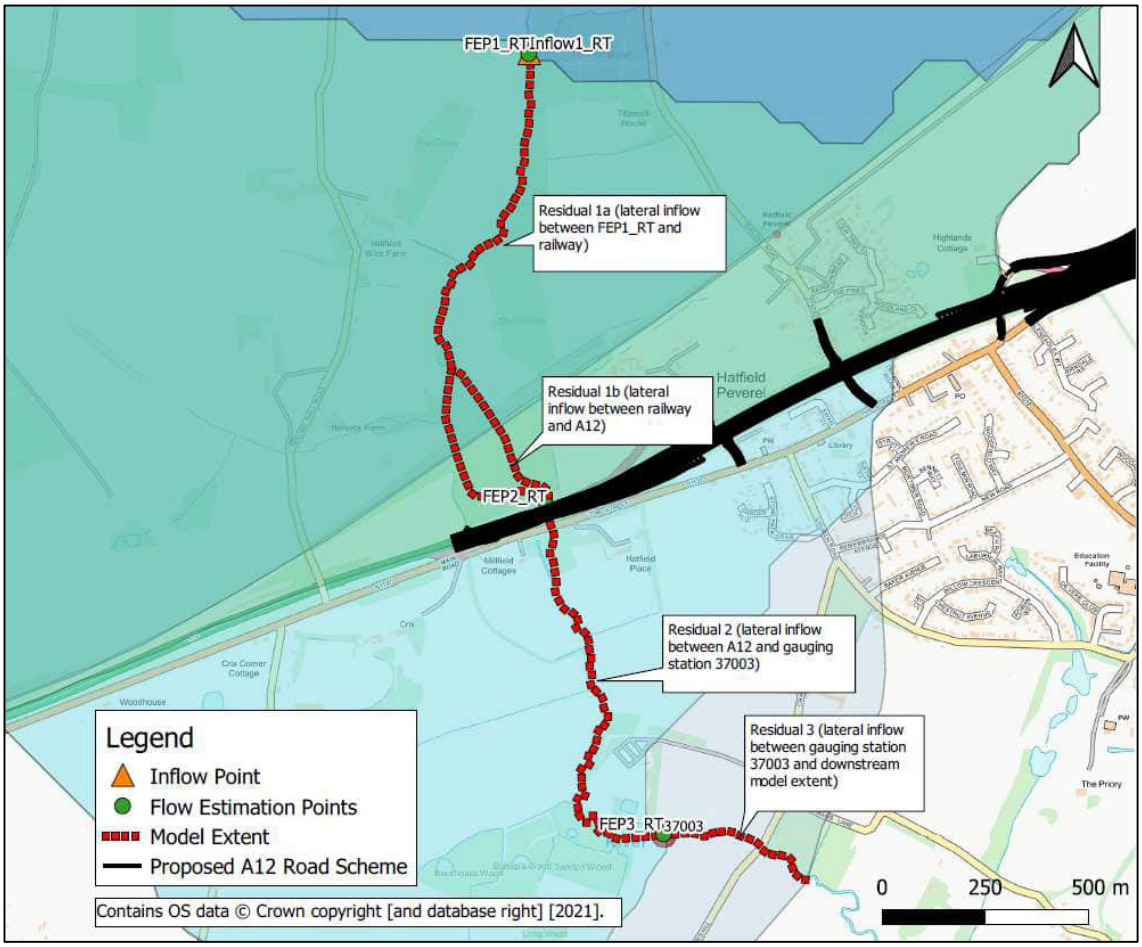


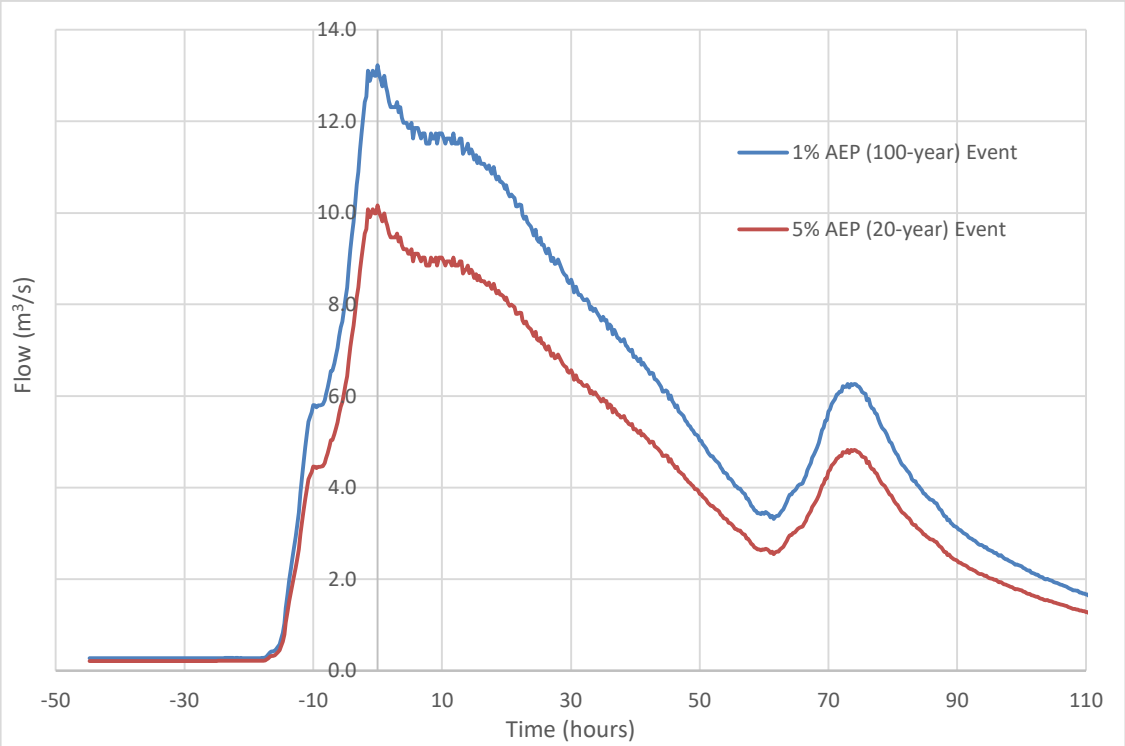
Table 3.2 Inflow peaks as a function of event rarity

Inflow	Peak Flow (m^3/s)										
	50% AEP (2-year)	20% AEP (5 year)	10% AEP (10-year)	5% AEP (20-year)	3.33% AEP (30-year)	2% AEP (50-year)	1.33% AEP (75-year)	1% AEP (100-year)	1% AEP + 65% CC Uplift (100-year) *	0.5% AEP (200-year)	0.1% AEP (1000-year)
Inflow 1	5.38	7.53	8.87	10.17	10.93	11.89	12.66	13.22	21.82	16.29	26.33
Residual 1a	0.34	0.48	0.57	0.65	0.70	0.76	0.81	0.84	1.39	1.04	1.68
Residual 1b	0.06	0.09	0.11	0.12	0.13	0.14	0.15	0.16	0.26	0.19	0.31
Residual 2	0.17	0.24	0.29	0.33	0.35	0.38	0.41	0.43	0.70	0.53	0.85
Residual 3	0.08	0.11	0.13	0.15	0.16	0.18	0.19	0.20	0.33	0.25	0.40

Event return period presented for reference in brackets.

**For details regarding the climate change uplift refer to Section 3.5*

Plate 3.2 5% and 1% AEP (20-year and 100-year) event inflow hydrographs (Inflow 1).



3.7 Summary

3.7.1 Design peak flow estimates for the River Ter were originally derived in 2017 but the project was subsequently put on hold. When the project was restarted in 2020 it was decided that the River Ter hydrological analysis was appropriate to take forward for the River Ter due to the watercourse being unlikely to be significantly impacted by the proposed scheme and due to any changes in the design hydrology likely to be minimal. The River Ter design peak flow estimates were derived using the FEH methodology applying the available hydrometric data at the River Ter at Crabbs Bridge gauge, FEH catchment descriptors and adopting the climate change allowance from the Environment Agency guidance at the time of assessment (EA, 2017). The adopted hydrograph shape was selected from analysis of gauged data at the River Ter at Crabbs Bridge gauge using the top five flood events on record at the time of assessment. The adopted hydrograph shape was from the 21st October 2001 flood event and was selected as being a typical shape. The inflow hydrographs were produced by scaling the selected hydrograph shape to the final design peak flow estimates for the River Ter and applied within the hydraulic model.

4 Baseline Modelling

4.1 Watercourse Schematisation - 1D Domain

4.1.1 Surveyed cross section data has been used to inform the in-channel geometry of the modelled watercourses. The locations of the surveyed cross-sections are shown in Plate 4.1.

4.1.2 To aid model performance on all modelled watercourses, interpolated cross sections were added between the surveyed cross sections. The spacing of the interpolated cross sections ranged from 70m to 100m.

4.1.3 Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using site photographs taken during the survey. Generally, gravel can be seen in the bed of River Ter with heavy vegetation and trees on the banks. There are some sections of concrete lined channels. The bed of the upstream end of the flood relief channel is heavily vegetated. The Manning's 'n' coefficients used in the model are shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow 1959).

4.1.4 Thirteen hydraulic structures were included in the model. Their approximate locations are shown in Plate 4.1. A structure schedule is presented in Annex B.

Plate 4.1 Model Extent and Inflow Locations.

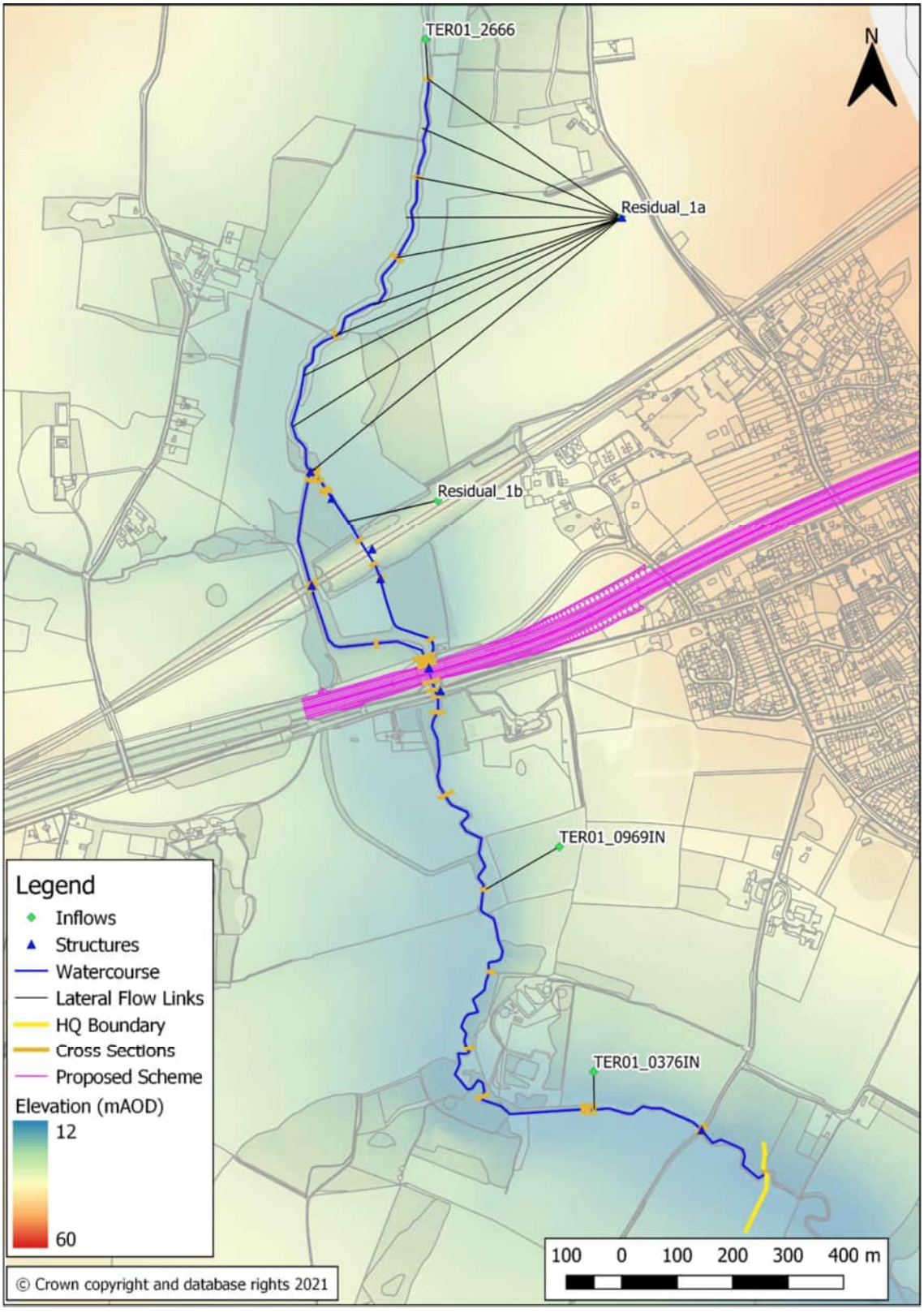


Table 4.1 Manning's 'n' Coefficients – 1D Domain

Flood Modellers Nodes	Watercourse	Bed Manning's 'n'	Bed Material	Floodplain Manning's 'n'	Floodplain Material
TER01_2666 to TER01_2136 TER01_1881 to TER01_1406 TER01_1348u TER01_1306 to TER01_0376 TER01_0356 to TER01_0000	River Ter	0.04	Gravel	0.1	Heavy vegetation/trees
TER01_1887u TER01_1405 to TER01_1363 TER01_1335d TER01_0371 to TER01_0366		0.02	Concrete		
TER02_0218d TER02_0191u to TER02_0191d	Flood relief channel	0.06	Vegetation		
TER02_0187 to TER02_0047		0.04	Gravel		
TER02_0017 to TER02_0000		0.04 0.02	Gravel Concrete		

4.1.5 The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4.2. Inflow locations are shown in Plate 4.1.

Table 4.2 Boundary Conditions – 1D Domain

Type of Boundary	Flood Modellers Node	Description
QT Boundary	TER01_2666	FEH hydrograph. Applied at the upstream end of the River Ter
QT Boundary	Residual_1a	FEH hydrograph. Applied laterally between the upstream end of the River Ter and the flood relief channel

Type of Boundary	Flood Modellers Node	Description
QT Boundary	Residual_1b	FEH hydrograph. Applied at the upstream end of the flood relief channel
QT Boundary	TER01_0969IN	FEH hydrograph. Applied on the River Ter downstream of the A12
QT Boundary	TER01_0376IN	FEH hydrograph. Applied on the River Ter downstream of Crabbs Bridge gauge
Normal depth downstream boundary	TER01_0000	Normal depth boundary condition applied to the downstream end of the River Ter. Sensitivity testing has shown that this boundary assumption does not affect model results in the area of interest (see Section 6.4)

4.2 Floodplain Schematisation - 2D domain

4.2.1 The 2D domain covers an area of 0.5 km^2.

4.2.2 The topography of the floodplain is represented using a 4m resolution square grid. The levels for the grid cells are based on a Digital Terrain Model (DTM) derived from 2m resolution LiDAR data.

4.2.3 A review of the floodplain using available aerial and OS mapping has shown that there are no existing structures within the

4.2.4 floodplain that require modelling. Therefore, apart from the breaklines to represent the River Ter banktops, no other modifications were made to the DTM.

4.2.5 Hydraulic roughness coefficients are applied over each grid cell of the 2D domain, as shown in Table 4.3, depending on land use taken from OS Mastermap data. Roughness values adopted were taken from standard guidance (Chow, 1959).

Table 4.3 Manning's 'n' Coefficients – 2D Domain

Land use	Manning's n
Buildings	1.000
General surface, multi surface	0.050
General surface, steps	0.025
General surface, natural	0.055
General surface, manmade	0.035
General surface, unknown	0.025

Glasshouses	1.000
Inland water	0.020
Pond/lake	0.035
Coniferous trees	0.100
Coniferous trees (Scattered)	0.075
Coppice or osiers	0.110
Marsh reeds or saltmarsh	0.055
Scrub	0.070
Steps, manmade	0.025
Path, manmade	0.025
Rail, manmade	0.050
Road or Track	0.025
Roadside	0.025
Structure/path	0.025
Structure/path	0.025
Landform/embankment/cliff	0.050

4.2.6 No inflow has been applied directly in the 2D domain. Any flow across the 2D domain is as a result of the 1D channel being overtopped, simulating out of bank conditions. Table 4.4 describes the downstream boundary condition used in the 2D domain. Its location is shown in Plate 4.1.

Table 4.4 Boundary Conditions – 2D Domain

Type of Boundary	Flood Modeler Node	Description
Stage-Discharge	HQ Boundary	Free flow normal depth boundary applied at the downstream extent of the model.

4.2.7 The link between the 1D and the 2D domains was defined along the banktop of the River Ter. To ensure that the correct level is used for the 1D/2D link, banktop lines have been digitised, using levels from the LiDAR data.

5 Modelled Events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model.

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope.

Table 5.1 Modelled Events

	AEP Event		
Scenario	**5%AEP**	**1%AEP**	**1%AEP+65% CC**
Baseline	X	X	X
Roughness Sensitivity (1D and 2D)		X	
Hydrological Inflow Sensitivity		X	
Downstream Boundary Sensitivity (1D and 2D)		X	

6 Model Proving

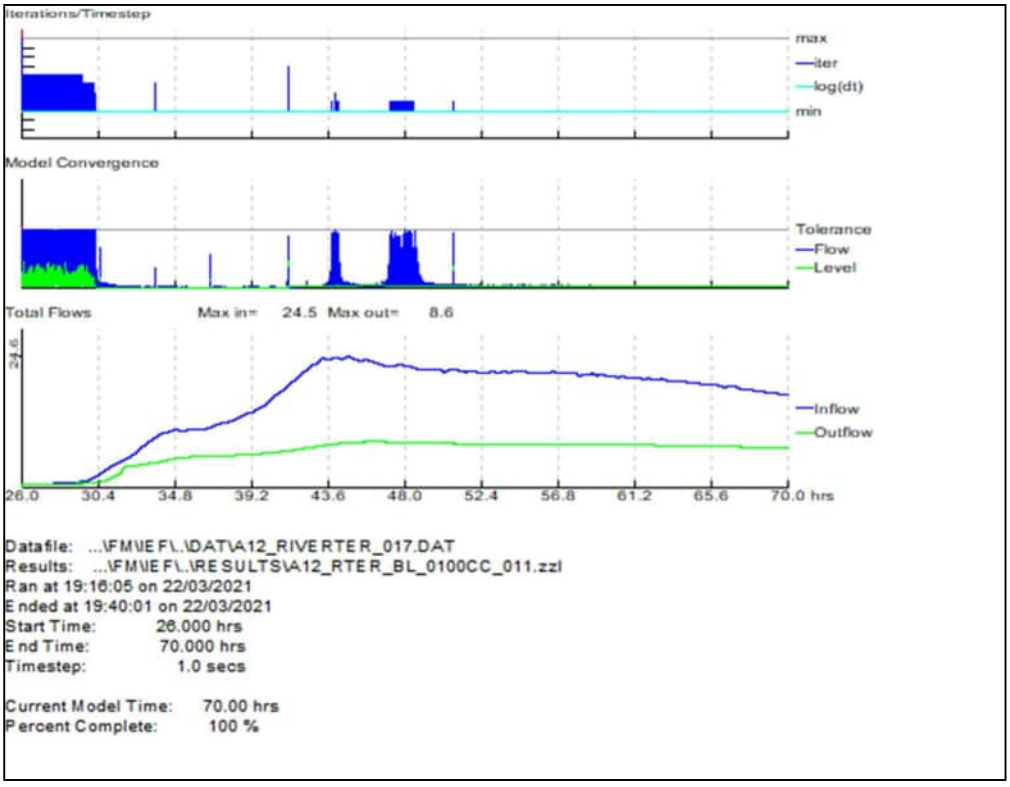
6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model Performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution within a specified number of iterations, for each model time step. The convergence of the 1D model was checked and seen to be within tolerance for the entire simulation. Plate 6.1 shows a typical convergence plot for the events modelled.

Plate 6.1 1D Model Convergence – 1% AEP +65% CC Event

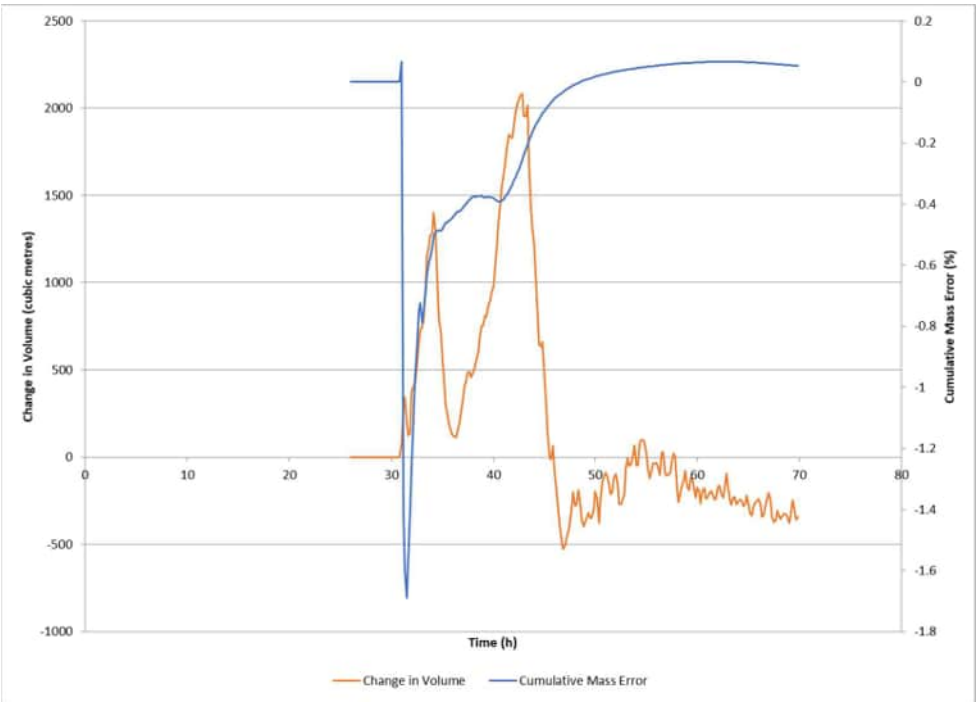


6.2.2 The cumulative mass error reports output from the TUFLOW 2D model have been checked for all simulated events. The accepted tolerance range recommended by the software manual is +/- 1% mass balance error. Plate 6.1 shows that for the 1% AEP flood event the cumulative mass error is well within this tolerance range for most of the duration of the run. The spike of the mass error at the beginning of the simulation is

expected and relates to the onset of water flow from 1D to 2D. Mass error rapidly decreases to acceptable values of around -0.4%.

6.2.3 Smooth variation of the change in volume (dV) through the model simulation is another indicator of good convergence of the 2D model (Plate 6.2). These 2D mass error and dV diagnostics are typical for all events simulated.

Plate 6.2 Cumulative Mass Error and Change in Volume – Baseline Run – 1% AEP +65%CC



6.3 Calibration and Verification

Calibration

6.3.1 There is a flume gauge on the River Ter at Crabbs Bridge. However due to the very large freeboard between the baseline flood extent and the highway scheme it was not required to utilise this gauge for any calibration of the hydraulic model.

Verification using Historic Data

6.3.2 The Environment Agency Publish historic flood maps for UK watercourses, which show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. However, a review of this data showed that there is no historic flood data for River Ter.

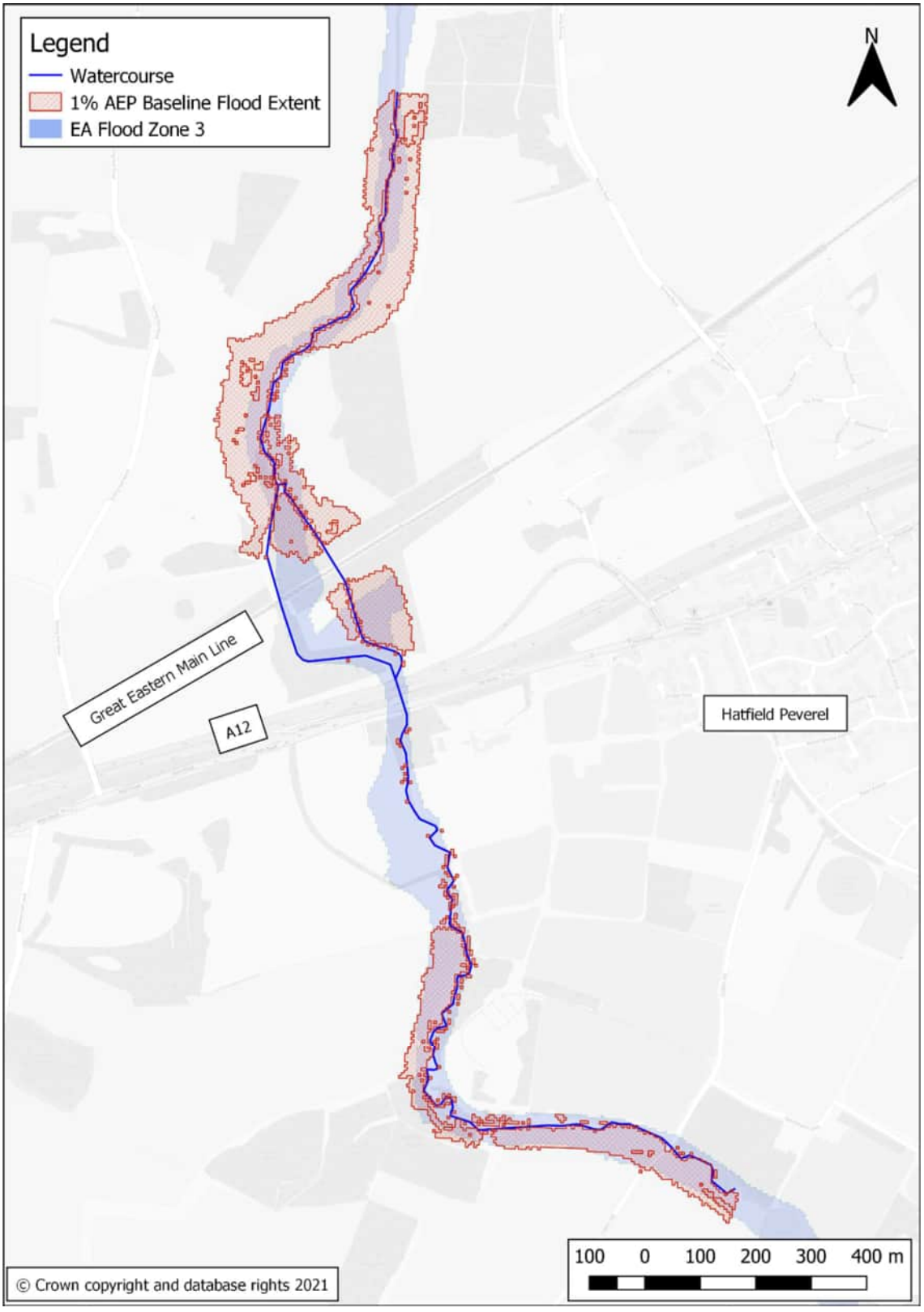
Verification using EA Flood Maps

6.3.3 Plate 6.3 shows the modelled 1% AEP baseline scenario flood extent and the Environment Agency published Flood Zone 3. The comparison shows that the Environment Agency flood map predicts a slightly larger flood extent than the present model results downstream of the railway crossing whereas upstream, the model prediction is larger. A review of the topography in the study area shows that the Environment Agency flood map features flooding on low ground adjacent to the watercourse. The present modelling shows that these low areas would not flood as they are separated from the watercourse by higher ground. The inconsistency is also expected as the EA map is generated based on the high-level J-Flow modelling system which does not explicitly account for the in-channel capacity of the watercourse, which the present study shows to carry the full flood flow for much of the modelled reach. It is presumed that the EA flood map modelling is based on a higher level and conservative assessment of design flow than the detailed analysis utilised in the present study.

Verification Conclusion

6.3.4 In conclusion, there is limited data available for verification of the River Ter model. The Environment Agency flood map is generally larger downstream of the railway and smaller upstream of it. This is due to the present study being a more detailed assessment of flooding than the Environment Agency flood map.

Plate 6.3 Modelled 1% AEP Event Flood Extent vs. Environment Agency Flood Zone 3



6.4 Sensitivity Analysis

Roughness Sensitivity

6.4.1 In-channel and floodplain roughness coefficients (Manning's 'n') were adjusted by +20% and -20%. Table 6.1 shows the impact of changing the model roughness on predicted peak water levels. The results show that the in-channel water levels are moderately sensitive to changes in roughness.

Table 6.1 Roughness Sensitivity Results

Sensitivity	Roughness Sensitivity Results		
	Max	Min	Average
+20% Roughness	0.25	0	0.08
-20% Roughness	-0.18	0.04	-0.07

Hydrological Inflow Sensitivity

6.4.2 The flows into the model were adjusted by +20% and -20%. Table 6.2 shows the impact of changing the model inflows on predicted peak water levels. The results show that the in-channel water levels are moderately sensitive to changes in flow.

Table 6.2 Flow Sensitivity Results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.37	0	0.12
-20% Flow	-0.24	0	-0.12

Downstream Boundary Condition Sensitivity

6.4.3 The slope in the normal depth downstream boundary was adjusted in both the 1D and the 2D domains for two sensitivity tests. For the first test, downstream boundary slope was changed from the baseline value of 0.002 to 0.001 and for the second to 0.0007. Table 6.3 shows the response at the downstream end of the model (Flood Modeller node TER01_0000). The location at which there is no change in water level as a result of changing the downstream boundary has been identified. Distances from this location, in relation to the downstream end of the model (tailwater distance) and in relation to the proposed scheme are also shown. The results show that the effect of the downstream boundary does not reach the scheme location.

Table 6.3 Downstream boundary condition sensitivity results

Sensitivity	Water Level Difference (m)		
	Water Level Difference at TER01_0000	Tailwater Distance	Distance to proposed scheme
Downstream boundary slope = 0.001	0.14	366 m	1000 m
Downstream boundary slope = 0.0007	0.32	366 m	1000 m

7 Model Results

7.1.1 In-channel maximum water levels have been inspected at key locations in relation to the proposed scheme. Table 7.1 shows in-channel maximum water levels for all modelled events. Plate 7.1 shows the maximum flood extent for all modelled events.

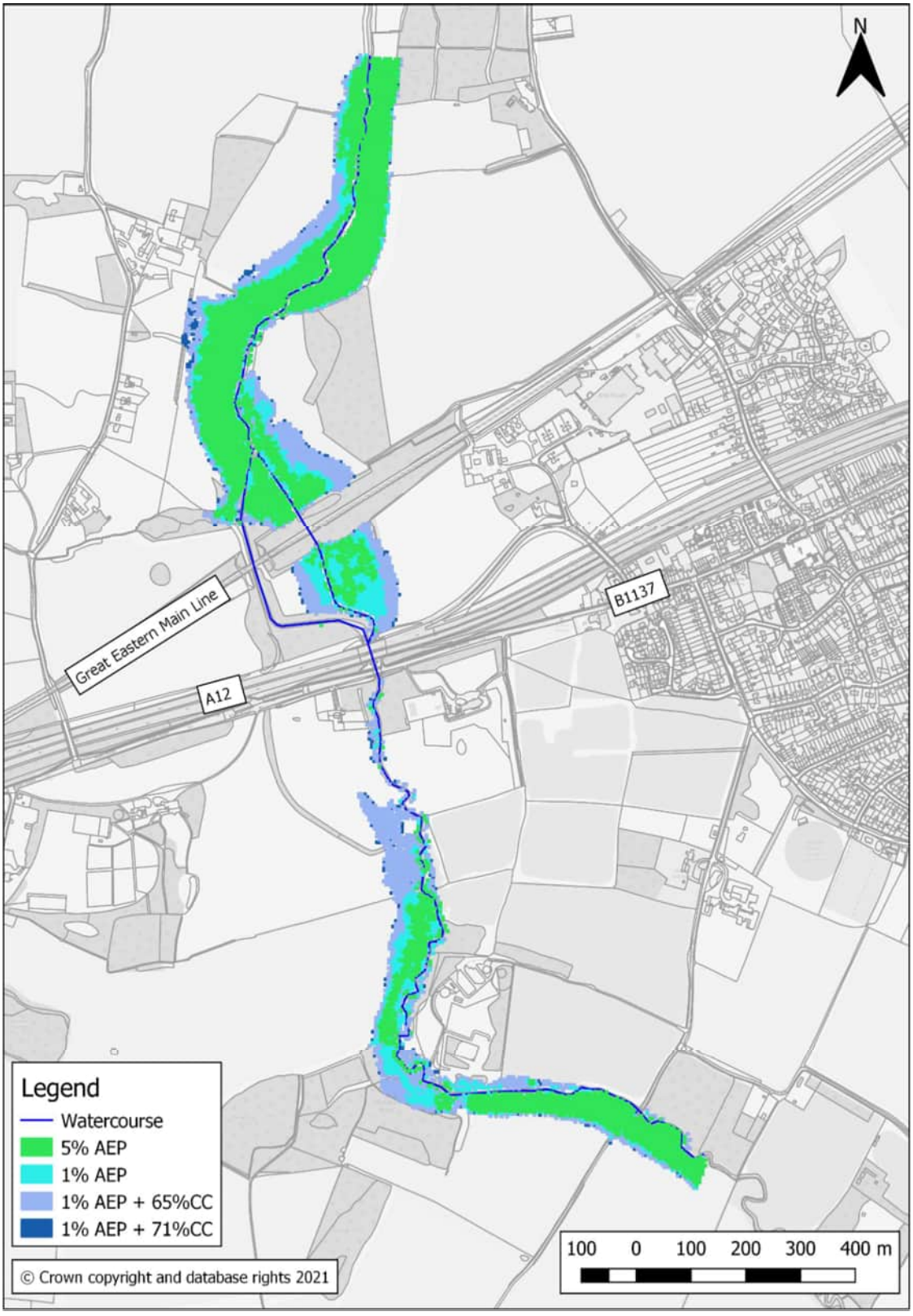
7.1.2 The results show that for the entire modelled extent, water is contained within the river valley and no properties are flooded. There is some water ponding upstream of the Great Eastern Main Line railway and upstream of the A12 Crossing.

7.1.3 The highway crossing at the River Ter has a freeboard of 3.75m between the 1%AEP +65% CC maximum water level and the bridge soffit. It is confirmed that the modelled baseline flood extent for the River Ter does not interact with any elements of the proposed highway scheme. The crossing will be upgraded to three lanes per carriageway without any adjustment of the external Bridge structure or any change to the highway embankment at this location.

Table 7.1 In-channel maximum water level at key locations

Model Node	Description	Baseline Water Level (m AOD)		
		5% AEP	1% AEP	1% AEP + 65%CC
TER01_2666	Upstream extent of the River Ter. 1.3km upstream of the scheme	21.03	21.10	21.26
TER01_1394u	Immediately upstream of the A12 crossing	18.07	18.31	18.99
TER01_1335d	Immediately downstream of the A12 crossing	17.70	17.88	18.31
TER01_0000	Downstream end of the River Ter. 1.3km downstream of the scheme	15.14	15.22	15.41

Plate 7.1 Flood Extents for all Modelled Events



8 Model Assumptions and Limitations

8.1 Hydrology

8.1.1 The main assumptions used in this study are:

- the AMAX data available from the Environment Agency is fit for purpose;
- the rating equation and gauging station are of suitable quality;
- With AMAX 50-years, the FEH Statistical method (Enhanced Single Site Analysis) estimated return period peak flows are more appropriate than those estimated using the ReFH2.2 method for up to the 1% AEP (100-year) event;
- The most appropriate means of extending the flood frequency curve beyond the 1% AEP (100-year) event is to estimate by multiplying the 1% AEP FEH Statistical peak by the ratio of ReFH2.2 T-year event peak and ReFH2.2 1% AEP (100-year) event peak.
- A climate change uplift of plus 65% is satisfactory for the purposes of this assessment (the Anglian region upper limit, 2080s scenario). The climate change uplift was applied to the 1% AEP (100-year) event based on the EA guidance (EA, 2017b) at the time of assessment. It should be noted that adopting the latest Environment Agency climate change guidance uplift (EA, 2021) would result in a lower climate change uplift of plus 38%. As the proposed scheme is outwith the 1% AEP (100-year) event plus 65% uplift flood extent it was assessed as not necessary to alter the climate change uplift.

8.1.2 It is noted that the ratio of the ReFH2.2 estimates for return periods greater than the 1% AEP (100-year) event to extrapolate the flood frequency curve beyond the 1% AEP (100-year) event does result in quite a significantly steepening of the flood frequency curve. This may introduce a layer of conservatism into these rarest of events.

8.2 Modelling

8.2.1 The accuracy and validity of the hydraulic model results is heavily dependent on the accuracy of the hydrological and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

8.2.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are considered to be generally conservative for modelled water levels at the proposed scheme and are therefore appropriate for the flood risk assessment.

8.2.3 The key sources of uncertainty and the limitations associated with the modelling undertaken for River Ter and its flood relief channel are as follows:

- Channel roughness has been assigned using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the Flood Modeller software. The dimensions for structures have been based on detailed survey measurements.
- A normal depth boundary was used at the downstream end of the model. The sensitivity analysis has shown that changes to the downstream boundary do not impact upon modelled levels at the area of interest (A12); therefore, it was deemed appropriate to use a normal depth boundary.
- The LIDAR data is assumed to appropriately represent the floodplain.
- A 4m grid has been used. This is deemed an enough level of detail to represent floodplain topography and flooding mechanisms predicted by the model in the vicinity of the scheme.
- No calibration was carried out. That activity would add no value to the flood risk analysis of the A12 highway scheme, due to the initial finding from the model results that the scheme elements are not affected by the River Ter fluvial system by a considerable margin.

9 Conclusion

9.1.1 This report has detailed the modelling carried out to assess the baseline flood risk for River Ter and its flood relief channel with reference to the location of the proposed highway scheme.

9.1.2 The results have shown that there are no properties at risk of flooding, and the A12 remains free from flooding for up to the 1% AEP + 65%CC event.

9.1.3 The modelled flood extents do not intersect with any proposed scheme elements, therefore there is no requirement to carry out any modelling for the proposed scheme scenario.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

References

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Environment Agency (2017c). Spatial Data Catalogue. [online] Available at: http://environment.data.gov.uk/ds/catalogue/#/catalogue [Accessed October 2017].

Environment Agency (2020a). Flood Estimation Guidelines. Technical guidance 197_08.

Environment Agency (updated 2021). Flood Risk Assessments: Climate Change Allowances guidance.

Jacobs (2020). River Ter, A12 Flood Risk Assessment Report.

Annex A

A.1 River Ter Hydrology Annex

A.1.1 HE551497-JAC-EWE-SCHW-RP-LE-0072

Annex B

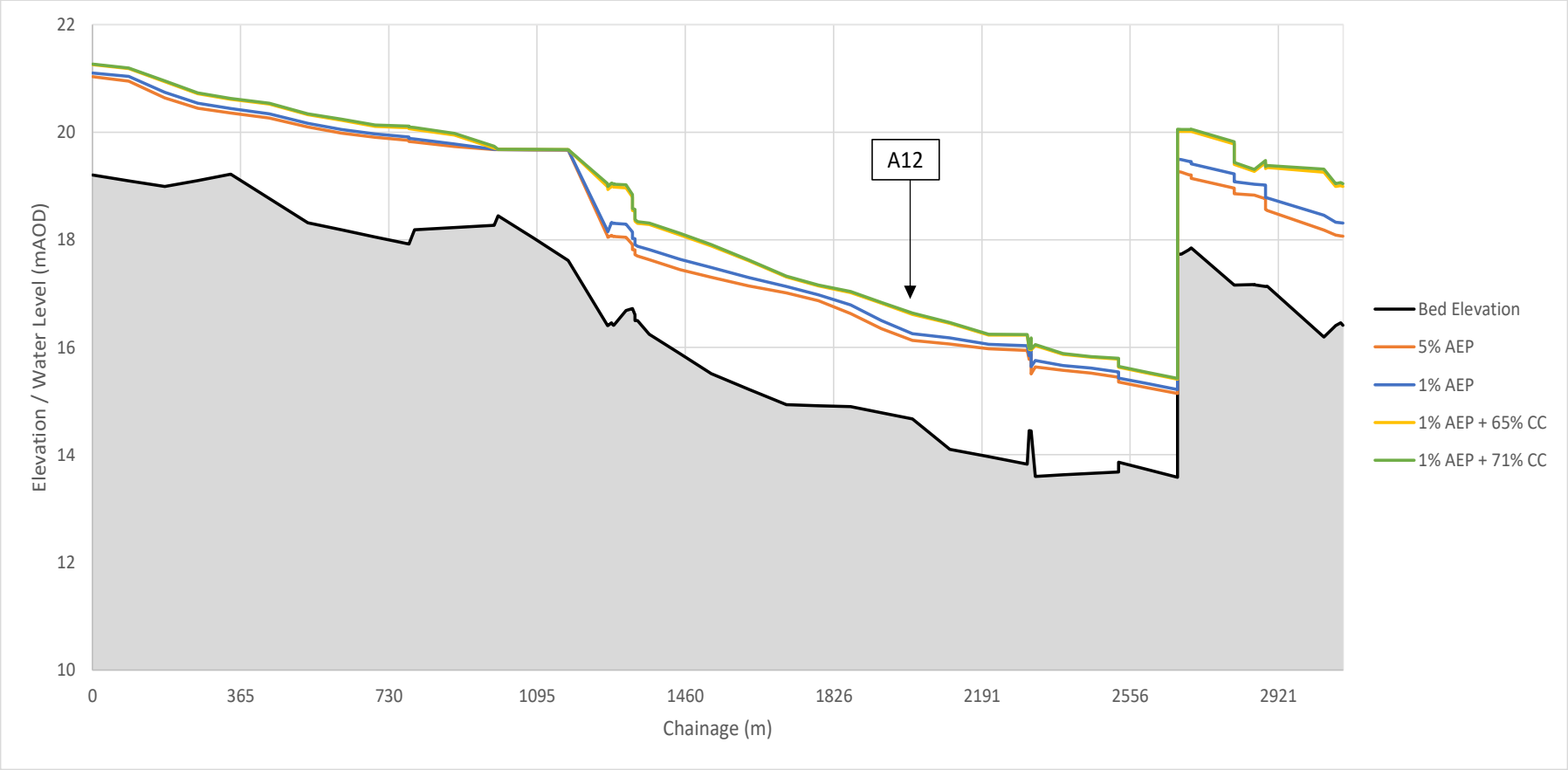
B.1 1D Structure Schedule

Table B.1 Model Structure Schedule

Structure Name	Node Label(s)	XY Coordinates	Modelling Approach
Foot Bridge 03.026	TER01_1887bu	578107, 211889	Represented as Orifice with Spill unit. Data from detailed survey (Jacobs 2017). FM default coefficients applied for all parameters.
Foot Bridge 04.0006	TER02_0374u	578163, 11818	Represented as Orifice with Spill unit. Data from detailed survey (Jacobs 2017). FM default coefficients applied for all parameters.
Culvert under railway embankment 04.005	TER02_0268cu TER02_0268cd	578225, 211745	Box culvert represented with symmetrical conduit and inlet/outlet nodes. Spill not required
04.003	TER02_0191bu	578240, 211695	Represented as Arch Bridge with Spill unit. FM default coefficients applied for all parameters
Railway bridge 03.024	TER01_1684bu	578120, 211677	Represented as USBPR Bridge, 3 arch openings with Spill unit. FM default coefficients applied for all parameters
A12 Road Bridge	TER01_1394bu	578336, 211532	Represented as USBPR Bridge with Spill unit. FM default coefficients applied for all parameters
B1137 Main Road	TER01_1348c TER01_1348cd TER01_1335c TER01_1335cd	578350, 211493	Represented as culvert with spill. Full arch conduit (L=6.4 m) and rectangular conduit (L=6.4 m) with head loss unit for change of shape (coef. set to 0.7 equivalent to sudden enlargement of 1:2.8) FM default coefficients applied for all other parameters.
Crabb's Hill Road bride 03.003	TER01_0151bu	578854, 210722	Represented as Arch Bridge with Spill unit. FM default coefficients applied for all parameters

B.2 Long Section

Plate A1.1 Long Section showing In-Channel Peak Water Levels for all modelled events



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVER TER HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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1 Executive Summary

1.1.1 This document has been prepared to support the A12 River Ter with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This record sheet is to be used where flood estimates are needed at a single location.

3 Method Statement

3.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Peak flow or hydrograph? • Range of return periods • Approx. time available	The A12 is proposed to be widened and therefore design peak flow/hydrographs are required for input into the River Ter numerical hydraulic model that is to be developed for assessing the flood risk in the vicinity of the proposed scheme. Design peak flows/hydrographs are required for the following AEP events: 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 1% plus climate change adjustment (plus CC), 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 100 plus CC, 200 and 1000-year design return periods). For the climate change uplift the following factor will be adopted: • 1% AEP event plus 65% uplift (based on the Environment Agency (EA) 2017b guidance[1]- Anglian region upper limit, 2080s scenario) It should be noted that adopting the latest Environment Agency climate change guidance uplift (EA, 2021) would result in a lower climate change uplift of 38% (Combined Essex Management Catchment peak river flow allowance, higher 2080s scenario). Given the proposed scheme is outwith the 1% AEP event plus 65% uplift flood extent no revision has been made to the climate change uplift.

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The River Ter catchment is a lowland catchment in the South East of England. The catchment is essentially rural with small urban areas around Bannister Green, Great Leighs, Little Leighs, Terling and Hatfield Peverel. There are small pockets of woodland within the catchment and areas of grassland but the majority of the catchment is classified as being of arable / horticultural use. The River Ter has a number of small waterbodies / reservoirs within its catchment

[1] Environment Agency (2017b). Flood Risk Assessments: Climate Change Allowances guidance.

	including the Leez Lodge Lakes in the upper reaches of the catchment. The waterbodies / reservoirs are predominantly offline and are assessed as provide only minor attenuation of flow, as represented by a FARL value of close to 1.0. The geology of the catchment has been assessed as being predominantly London Clay overlain with superficial deposits of mainly bolder clay and some sands and gravels. A small area in the upper catchment (approximately 1% of the catchment) has also been classified as being of generally high permeability superficial deposits with the remainder of the catchment being classified as being of mixed permeability superficial deposits. The soils within the catchment are classified as predominantly of the chalky till variety with a small area of tertiary clay and aeolian silty drift. Refer to Annex A.1 for the location of the River Ter catchment.

3.3 Source of flood peak data

Was the NRFA/HiFlows UK dataset used? If so, which version? Record any changes made.	Yes, WINFAP-FEH v5 data files were used which were downloaded on the 04/09/2017. These contain the most up to date NRFA/HiFlows UK dataset at the time of assessment.

3.4 Gauging stations (flow or level)

(at the site of the flood estimate or nearby at potential donor sites)

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
River Ter	Crabbs Bridge	037003	37003	TL786107	77.8	Level	1964 to present

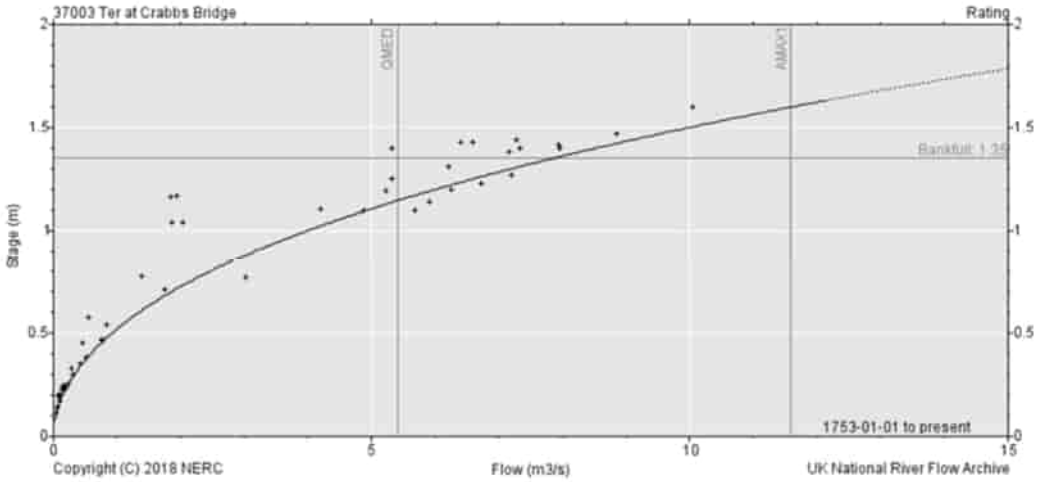
3.4.1 Other gauging stations were investigated as potential donor gauges for the River Ter including the following NRFA gauges: 37009, 37002, 37008, 37013, 37010, 37020 and 37011. None of these gauging stations are on the River Ter and were not assessed as having an advantage over the River Ter @ Crabbs Bridge gauge (37003) for the purposes of this study. All gauges with the exception of 37002 have a similar record length to the River Ter @ Crabbs Bridge gauge. Gauge 37002 does have a considerably longer record length but the gauge is not classified as a high flow gauge and it has a substantially larger catchment. These factors limit the usefulness of the gauge for the purposes of this assessment

3.5 Data available at each flow gauging station

Station name	**Start and end of data in HiFlows-UK**	**Update for this study?**	**Suitable for QMED?**	**Suitable for pooling?**	**Data quality check needed?**	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Crabbs Bridge (River Ter)	1964 to 2016	No	Yes	Yes	No	Trapezoidal flume with throat tapping. Theoretically rated. Modular limit 0.9m, no level yet recorded above 1.6m. There is increased scatter in gaugings beyond QMED leading to some uncertainty in the accuracy of the high flow rating. One peak flow rating has been applied to this engineered structure across period of record. The EA supplied for the study the 15-minute flow (1979 to 2017) and AMAX data (1964 to 2016). (It was noted that the 2015 AMAX value did not match the 15-minute flow data, and the 2016 water year was incomplete. The 2015 / 2016 AMAX values were therefore not used in this analysis. There was also a difference observed in the supplied EA AMAX series to the values recorded in the Peak Flow dataset on the NRFA website for water year 1963. The EA supplied value was favoured. Water year 1980 was further excluded from QMED assessment as it has been rejected in WINFAP. The EA AMAX QMED = 5.69m^3/s whereas to the equivalent Peak Flow QMED = 5.42m^3/s. The EA AMAX series was accepted as the most up to date interpretation of the record (presented in Annex A.6) and was used in this study.
Give link/reference to any further data quality checks carried out	No further data quality checks were undertaken.					

3.6 Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Crabbs Bridge	Theoretical	No, though a high level check on the data provided was made.	The rating is based upon a theoretical understanding developed by the gauging authority (the EA). Flow is modular beyond 0.9m (circa 3.2m^3/s) - 0.95m (circa 3.8m^3/s) and the ratings do not include bypass or non-modularity for which the EA noted they make adjustments for in the data they provided (as per email communication with the EA on 21 July 2017 – see Annex A.7). The Peak Flows graph showing the spot gaugings against the high flow rating curve for the gauge is reproduced below for information.
Give link/reference to any rating reviews carried out			



Dotted line indicates where rating is extrapolated beyond the maximum stage of the upper limb. AMAX1 is the highest AMAX value on record.

3.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)	No detailed rating review is proposed to be undertaken for this study.				
Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies (e.g. CFMPs, Strategies)					
Other data or information (e.g. groundwater, tides)					

3.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes - FEH Statistical method considered appropriate for this essentially rural catchment. ReFH2.2 will also be used to derive design peak flows beyond 1% AEP (100-year return period) event.
Outline the conceptual model, addressing questions such as: • What is likely to cause flooding at the site (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might the site flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir?	Flooding at the site is most likely to be caused by excess fluvial flows (peak flows and flood volume). Flooding is indicated on the EA Flood Maps for Planning along the watercourse throughout the model extent. The flood extent is indicated as Flood Zone 3 (land assessed as having a 1 in 100 or greater annual probability of river flooding in any year). There are numerous small waterbodies / reservoirs within the catchment as well as

• Is there a need to consider temporary debris dams that could collapse?	numerous springs. Given the waterbodies / reservoirs generally tend to be offline it is not anticipated that they will greatly impact the catchment flood mechanisms i.e. any flood risk is likely to be associated with processes across the catchment. No debris dam issues are known about for this catchment.
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, use permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	There is a small area in the upper catchment which has been assessed as having a geology which is composed of generally highly permeable superficial deposits. The catchment BFIHOST (0.461) and SPRHOST (41.76) values at the gauge are, however, in the normal range and do not indicate high permeability across the catchment as a whole. The River Ter catchment to the Crabbs Bridge gauge (37003) is classified by its FEH $URBEXT_{2000}$ value as being 'essentially rural'. There are numerous small waterbodies / reservoirs within the River Ter catchment. The majority of these have been assessed as being offline and therefore are not considered to unduly impact design peak flow estimates. The FEH Web Service records a FARL of 0.994 at the Crabbs Bridge gauging station. FPEXT = 0.1153 which indicates there are some limited areas of flood plain within the catchment.
Initial choice of method(s) and reasons	The FEH Statistical (enhanced single site analysis) and ReFH2.2 methods are used to derive independent estimates of the peak flood flows. Design hydrographs shapes are developed from average flood shapes obtained from the flood record at Crabbs Bridge. The typical hydrograph is linearly scaled to agree with the finalised peak flows.
Software to be used (with version numbers if relevant) e.g. FEH Web Service (date accessed) WINFAPv4 WINFAP-FEH v5 data files (previously HiFlows) ReFH2.2	FEH Web Service – accessed 6th September 2017 WINFAPv4 WINFAP-FEH v5 data files – downloaded 4th September 2017 ReFH2.2

3.9 Site details

Watercourse	Site	Easting	Northing	AREA on FEH Web service (km^2)	Revised AREA if altered
River Ter	Crabbs Bridge	578600	210750	77.8	77.56

3.10 Catchment descriptors (incorporating any changes made)

FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT2000	FPEXT
0.994	0.31	0.461	13.81	18.6	570	41.76	0.0116	0.1153

3.11 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	The FEH Web Service catchment boundary was reviewed using contour data. Slight alterations were made to the River Ter catchment in the vicinity of the A12 / railway.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	The soils within the catchment were assessed using the Soil Map of England and Wales at a scale of 1:250,000 (published 1983). Soils were noted to be of predominantly chalky till variety with some areas of tertiary clay and aeolian silty drift. Based on this assessment the FEH SPRHOST / BFIHOST values were not amended.
Source of URBEXT	$URBEXT_{2000}$ has been extracted for the River Ter catchment from the FEH Web Service.
Method for updating of URBEXT	$URBEXT_{2000}$ amended within WINFAPv4 using the urban adjustment procedure.

4 Statistical Method – Target Location is Crabbs Bridge Gauging Station

4.1 Overview of estimation of QMED

Used	Condition	Approach followed
√	N >=30	Estimate QMED using annual maxima
	14=< N =<29	Estimate QMED from annual maxima & optionally adjust for climatic variation
	2=< N= <13	Estimate QMED from POT data & adjust for climatic variation
	N <2 & suitable donor site with 20 years or more of record	Ignore record at subject site; transfer QMED from donor site
	N <2 & suitable donor with 10 to 19 years of record & 12 month overlap between records	Estimate QMED using procedure based on flood peak regression
	N <2 & suitable donor with 10 to 19 years of record but no 12 month overlap	Ignore record at subject site; transfer QMED from donor site
	N <2 & no long-record site nearby	Estimate QMED from very short POT record
	N <2 & no long-record site nearby	Treat site as ungauged catchment
	N <2 & no long-record site nearby	Defer analysis until longer flow record available
	N <2 & no long-record site nearby	(Abstract flood event information and apply the UH rainfall-runoff model as an alternative, to the pooling group procedure. Particularly recommended when site is urbanised)
√	Ungauged catchment	Estimate QMED from catchment descriptors

	Ungauged catchment	Estimate QMED by data transfer from donor catchment
	Ungauged catchment	Estimate QMED by data transfer from analogue catchment
	Ungauged catchment	Estimate QMED from channel dimensions
	Ungauged catchment	Compare to regional pattern of mapped QMED adjustment factors

4.2 Search for donor sites for QMED

Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, URBEXT, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	The target site the River Ter @ Crabbs Bridge is gauged. And for this target site no donor gauge is required. However, to initially supply the required input into the project hydraulic model there is a need to consider a donor adjustment to upstream inflows and for this reason the $QMED_{data}/QMED_{cd}$ ratio is calculated below.

	Subject site
NAME	River Ter @ Crabbs Bridge*
Gauge Number	37003
AREA	77.56
SAAR	570
FARL	0.994
SPRHOST	41.76
BFIHOST	0.461
Years	50
QMED cd's (urban)	6.51
QMED data	5.69
Ratio (R) $QMED_{data}/QMED_{cds}$	0.8745

Method	Initial estimate of QMED (m^3/s)	Data transfer				Final estimate of QMED (m^3/s)
		NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	
AM	5.69	37003	Done as a simple scaling where needed. But note that the target location is the gauge and the river model need's to agree at this point.			

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added below.

When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why.

The data transfer procedure is the revised one from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

If more than one donor has been used, give the weights used in the averaging.

4.3 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
37003	Good for QMED	AM	No	5.69	6.51 (urban)	0.8745

4.4 QMED estimation from catchment descriptors

Attribute	Value
AREA	77.56
SAAR	570
FARL	0.994
SPRHOST	41.76
BFIHOST	0.461
URBEXT2000	0.0116
URBEXT2017	0.012

$QMED_{Catchment\ descriptors\ rural}$ = 6.41 m^3/s 68% confidence interval = (4.48/9.17)

95% confidence interval = (3.13/13.13)

$QMED_{Catchment\ descriptors\ urban}$ = 6.51 m^3/s

Final estimate of Qmed at the Crabbs Bridge target location

Gauge 37003 has been selected for QMED adjustment.

$QMED_{AM}$ = 5.69m^3/s

4.5 Derivation of pooling group

Name of saved**.feh** group file: River Ter at Crabbs Bridge Pooling
Target return period (years) for 5T rule *: T = 100 years

*(Updated recommendations advise that a standard 500 years is used regardless of target return period i.e.T=100yr)

For initial pooling group details (using the Enhanced Single Site analysis method) refer to Annex A.3.

Total number of sites	10
Total number of years	504
Total number of initial highly discordant sites	0
Discordant sites	N/A
Total number of short records (< 7 years) removed	0
List sites removed with reasons	34012 / 26003 / 33032 / 39042 – SPRHOST values less than 20. Suggesting these to be permeable catchments with substantial groundwater components, whilst the target catchment has a SPRHOST of 41.8% which suggests a moderately impermeable catchment.
Number of pooled years after sites removed	311

Is subject site included in pooled group: yes / ~~no~~

If no state reason why: _____

Test statistics on validity of pooling group for flood frequency analysis

Heterogeneity test H2 value = 4.367

Status	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable		2 < H2 < 4
	Review essential	X	H2 > 4

				Value
Goodness-of-fit test	Z values	GL	~~acceptable~~ /not acceptable	3.406
		GEV	acceptable /~~not acceptable~~	-0.441
		PT3	acceptable /~~not acceptable~~	-0.282
	other			

(Note: in the FEH the GL is the generally favoured distribution for use)

ACTION is construction of flood frequency curve valid?

YES: / NO: review the pooling group further

Comment ?	No need to add sites for 5T Rule

4.6 Revision of Pooling Group

Revision No. 1

Station Number	Reason for changes in pooling group
54036 / 35008 / 38002 / 33011	Next ranked sites added to meet the requirements of the 5T rule.

Number of sites	10	Years	528

Heterogeneity test H2 value = 1.89

Status			
	Review not necessary		H2 < 1
	Review optional	X	1 < H2 < 2
	Review desirable		2 < H2 < 4
	Review essential		H2 > 4

Note: FEH Vol.3, chapter 16.3.2: "The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review."

				Value
Goodness-of-fit test	Z values	GL	acceptable / ~~not acceptable~~	0.957
		GEV	~~acceptable~~ / not acceptable	-2.162
		PT3	~~acceptable~~ / not acceptable	-2.004

ACTION Is construction of flood frequency curve valid?

~~No~~ Yes

Comment ? Construction of FFC acceptable.

Derivation of flood growth curves

Method: SS – Single site ESS – enhanced single site P – Pooled J – Joint analysis	**If P / SS / ESS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 100-year return period**
ESS	GL – L-median GEV – L-median	Location: 1.000 (GL) Scale: 0.277 (GL) Shape: -0.057 (GL)	2.457
SS (just for comparison)	GL – L-median GEV – L-median	Location: 1.000 (GL) Scale: 0.262 (GL) Shape: 0.009 (GL)	2.180
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

Pooling group details are presented in Annex A.2 to A.3

4.7 Flood estimates from the statistical method

GL	Flood peak (m^3/s) for the following return periods (in years)									
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**200**	**1000**
Growth Factor - ESS	1.000	1.400	1.649	1.889	2.03	2.209	2.353	2.457	2.714	3.349
Flow - ESS	**5.7**	**8.0**	**9.4**	**10.7**	**11.6**	**12.6**	**13.4**	**14.0**	**15.5**	**19.1**
Growth Factor - SS	1.000	1.361	1.571	1.762	1.870	2.003	2.107	2.180	2.355	2.755
Flow - SS	5.7	7.7	8.9	10.0	10.6	11.4	12.0	12.4	13.4	15.7

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)									
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**200**	**1000**
Growth Factor-ESS	1.000	1.446	1.695	1.903	2.01	2.134	2.224	2.284	2.414	2.661
Flow – ESS	5.7	8.2	9.6	10.8	11.4	12.1	12.7	13.0	13.7	15.1
Growth Factor-SS	1.000	1.403	1.603	1.757	1.832	1.913	1.968	2.004	2. 078	2.199
Flow – SS	5.7	8.0	9.1	10.0	10.4	10.9	11.2	11.4	11.8	12.5

Refer to Annex A.4 for plots of the Enhanced Single Site (ESS) and Single Site (SS) growth curves.

From the four versions provided above the GL, ESS flood frequency estimates are preferred.

5 Revitalised Flood Hydrograph (ReFH2.2) Method

5.1 Parameters for ReFH2.2 model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible, so it is not essential to enter them in the table.

Location	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
FEP01 – Top of Model	CD	12.10	371.52	56.78	1.07
FEP03 – Crabbs Bridge (37003)	CD	12.92	376.37	58.71	1.08
Brief description of any flood event analysis carried out (further details should be given below or in a project report)	No flood event analysis undertaken.				

5.2 Design events for ReFH2.2 method

Location	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
FEP01 – Top of Model (located u/s of Crabbs Bridge, with CA = 71.54km^2)	Rural with urban adjustment	Winter	18.1	c/a
FEP03 – Crabbs Bridge (37003)	Rural with urban adjustment	Winter	22.1	c/a

5.3 Flood estimates from the ReFH2.2 method

Using ReFH2.2 Duration	Flood peak (m^3/s) for the following return periods (in years) (Urbanisation taken into account)									
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**200**	**1000**
FEP01 – Top of Model	8.5	11.2	13.0	14.9	16.1	17.9	19.5	20.9	25.8	41.7
FEP03 – Crabbs Bridge (37003)	8.6	11.4	13.2	15.2	16.4	18.1	19.8	21.2	26.2	42.3

6 Discussion and Summary of Results

6.1 Comparison of results from different methods

6.1.1 This table compares peak flows from various methods with those from the FEH Statistical method for two key return periods.

	Ratio of ReFH2.2 peak to FEH Statistical ESS peak					
	Return period 2 years			Return period 100 years		
	ReFH2.2	ESS	Ratio	ReFH2.2	ESS	Ratio
FEP01	8.5	5.4*	1.6	20.9	13.2*	1.6
FEP03	8.6	5.7	1.5	21.2	14.0	1.5

**Based on donor application of the ESS flood frequency curve for the River Ter @ Crabbs Bridge gauging station (FEP03) to this upstream site.*

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	ReFH2.2 gives higher peak flows than the FEH Statistical method (ESS and SS) for the River Ter @ Crabbs Bridge (at gauge 37003). Given the relatively long gauged record at Crabbs Bridge the FEH Statistical ESS design peak flows can be adopted in the project for up to and including the 1% AEP event. At the more common return periods where the single site analysis is considered to be robust and the ReFH2.2 estimates are shown to overestimate, and this ratio also persists when comparing the ReFH2.2 to the ESS at the rarer events. For this reason, the ReFH2.2 estimates are not favored. EA guidance indicates that, for a gauged site with 50-year of AMAX data, the design peak flows of rarer than the 1% AEP (100-year) event can be estimated by multiplying the 1% AEP (100-year) peak flow estimated by the statistical method by the ratio of the ReFH2.2 derived T-year peak flow, Q_T (for T>100yr) and the ReFH2.2 derived 1% AEP (100-year) flow, Q_{100}. The River Ter hydrological analysis was originally undertaken in 2017 but the project was subsequently put on hold. When the project was restarted in 2020 it was decided that the River Ter hydrological analysis was appropriate to take forward for the River Ter due to the watercourse being unlikely to be significantly impacted by the proposed scheme and due to any changes in the design hydrology likely to be minimal.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions used in this study are: • the AMAX data available from the Environment Agency is fit for purpose; • the rating equation and gauging station are of suitable quality;

<table>
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- With AMAX of 50-years, the FEH Statistical method (Enhanced Single Site Analysis) estimated return period peak flows are more appropriate than those estimated using the ReFH2.2 method for up to the 1% AEP (100-year) event;
- The most appropriate means of extending the flood frequency curve beyond the 1% AEP (100-year) event is to estimate by multiplying the 1% FEH Statistical peak by the ratio of ReFH2.2 T-year event peak and ReFH2.2 100—year event peak.

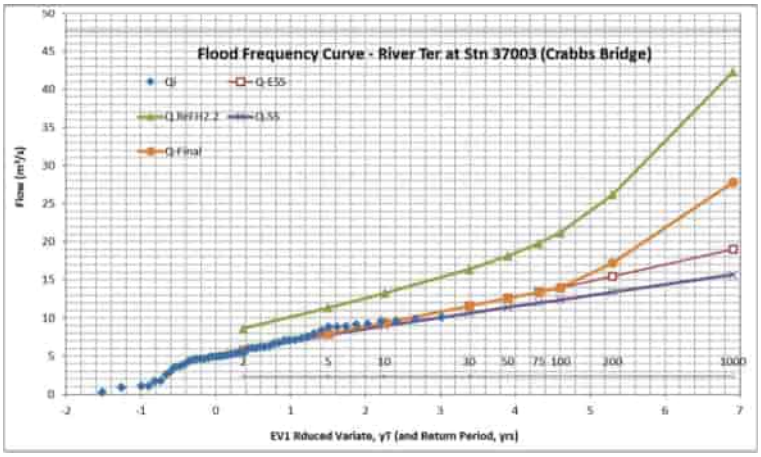
</td></tr>
<tr><td>Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed</td><td>(The methods used are those advocated in the EA Flood Estimation Guidelines for the AEPs targeted). No particular features of the catchment suggest significant atypical behavior is likely that would invalidate the approach used. However, it is noted that using the ratio of the ReFH2.2 estimates for return periods greater than the 100-year event to that of the 100-year event to extrapolate the flood frequency curve beyond the 100-year event does result in quite a significant steepening of the flood frequency curve. This may introduce a layer of conservatism into these rarest of events.</td></tr>
<tr><td>Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).</td><td>QMED has been estimated for the River Ter @ Crabbs Bridge using about 50 years of AMAX data (1964 to 2014 – 1988 AMAX is missing and 1980 has been excluded as per WINFAP). The estimated QMED is 5.69m^3/s.
The 68% and 95% confidence intervals for QMED calculated using FEHv3 are presented in the table below:

<table>
<tr><td>River Ter @ Crabbs Bridge</td><td>Lower limit (m^3/s)</td><td>Upper Limit (m^3/s)</td></tr>
<tr><td>68%</td><td>5.27</td><td>6.24</td></tr>
<tr><td>95%</td><td>4.99</td><td>6.94</td></tr>
</table>
The confidence intervals are based on a 50-year record.</td></tr>
<tr><td>Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.</td><td>The design flows estimated have been derived specifically for the River Ter @ Crabbs Bridge. Before the flows are used for future studies assessment of the suitability of the flows at that point in time and for the location proposed should be carried out. Flow derivation methods change in time and this should be taken into consideration along with the similarity of the target site requiring design peak flows to the target location.</td></tr>
</table>

Give any other comments on the study, for example suggestions for additional work.	If new high flow spot gauging's were available, it would be beneficial to check the validity of the rating equation.

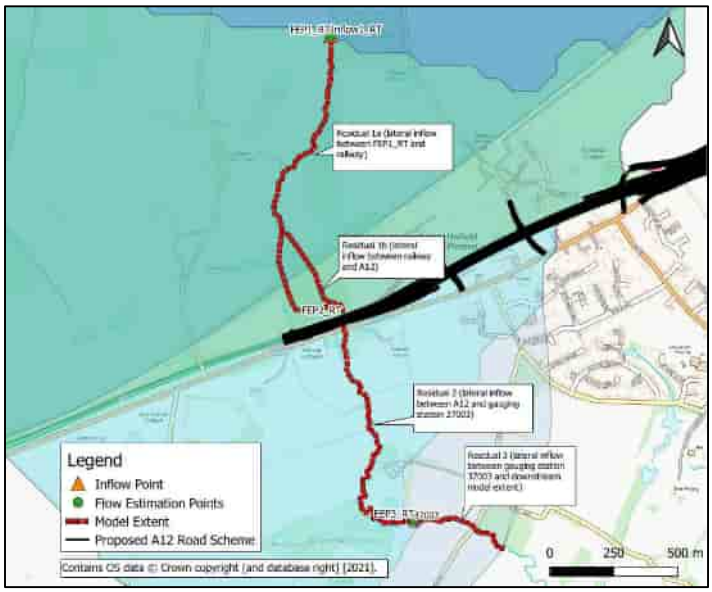
6.4 Checks

What do the results imply regarding the return periods of floods during the period of record?	The FEH Statistical ESS flows seem consistent with the AMAX record which records a peak flow of 11.6m^3/s (21st Oct 2001). This would correspond to approximately 3.3% AEP (30-year return period) flow from the ESS growth curve and approximately 2% AEP (50-year) from the SS growth curve. The ReFH2.2 based flood frequency curve is much higher than the FEH Statistical method-based frequency curves. On the ReFH2.2 frequency curve the highest flow in the 50-year period of record is only suggested to have a rarity of 20% AEP (5-year return period). This does not seem reasonable. Given there are 50 years of AMAX data, ESS design peak flows appear to be more consistent with the gauged data at lower return periods.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	The adopted 1% AEP (100-year return period) growth factor from ESS is: 2.457. This is considered within the expected range.
If 1000-year flows have been derived, what is the ratio for 1000-year flow over 100-year flow?	The 0.1% AEP (1000-year return period flow) / 1% AEP (100-year return period flow) = 1.99 (adopted from ReFH2.2 method derived flows).
What specific runoff (l/s/ha) does the design flow equate to?	The corresponding 1% AEP (100-year) and 50% AEP (2-year) specific runoffs are 1.8l/s/ha and 0.734l/s/ha respectively.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	At the time of assessment other studies on the River Ter were not available for comparison. However, for context, a previous study by CH2M (FEH Blackwater Calculations Record, CH2M, 2017) on the River Blackwater (a nearby but larger watercourse with reasonably similar soils and geology) reported a 1% AEP (100-year) design flow equivalent to 3.5l/s/ha at Copford (63km^2), 2.2l/s/ha at Stisted (140km^2), 1.3l/s/ha at Appleford (249km^2) and 3.0l/s/ha at Guithavon (61km^2) gauging stations. The 50% AEP (2-year) design flows for these gauges were also reported as being equivalent to

	1.3l/s/ha at Copford, 1.0l/s/ha at Stisted, 0.5l/s/ha at Appleford and 0.7l/s/ha at Guithavon.
Are the results compatible with the longer-term flood history?	Information on the longer-term flood history of the River Ter was not readily available at the time of assessment.
Describe any other checks on the results	The flood frequency curve for the River Ter @ Crabbs Bridge gauging station from various methods (FEH Statistical SS, ESS, ReFH2.2 and combined FEH Statistical and ReFH2.2) have been plotted together with the AMAX data at the station. 

6.5 Final results

	Flood peak (m^3/s) for the following return periods (in years)										
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**100+65% CC Uplift**	**200**	**1000**
FEP01 Combined FEH Statistical / ReFH2.2	5.4	7.5	8.9	10.2	10.9	11.9	12.7	13.2	21.8	16.3	26.3
Combined FEH Statistical / ReFH2.2 at the gauge	5.7	8.0	9.4	10.8	11.6	12.6	13.4	14.0	23.1	17.2	27.8

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	A flood hydrograph has been derived by comparing the flow hydrographs of the top five flood events at the River Ter @ Crabbs Bridge gauging station and a representative hydrograph shape adopted (only flood events post 1979 were assessed as the 15-minute flow from the EA was only available from 1979 onwards). The selected hydrograph shape is from the 21st October 2001 flood event. The inflow hydrographs for input into the hydraulic model were derived using this hydrograph shape which was linearly scaled to match the finalised AEP design peak flow estimates at the top of the model (FEP01) and at key locations within the modelled reaches (residual 1a, 1b, 2 and 3). A map showing the hydraulic model inflow locations is shown below:  The design peak flows at these locations were derived using the following methods: • At FEP01 (the top of the model) design peak flows have been derived using the methods presented above. QMED was derived using catchment descriptors (adjusted using the River Ter @ Crabbs Bridge adjustment factor) and the FEH ESS growth curve (derived at FEP03 – the River Ter @ Crabbs Bridge gauging station) was used to derive design peak flows for AEPs up to the 1% AEP event. For events greater than this the 1% AEP event

	FEH ESS derived peak flow is multiplied by the ratio of ReFH2.2 T-year event peak and ReFH2.2 100—year event peak. • For the residual catchments (residual 1a, 1b, 2 and 3) QMED was derived from catchment descriptors for the component areas (adjusted using the River Ter @ Crabbs Bridge adjustment factor) and the FEH ESS growth curve (derived at FEP03 – the River Ter @ Crabbs Bridge gauging station) was used to derive design peak flows for AEPs up to the 1% AEP event. For events greater than this the 1% AEP event FEH ESS peak flow is multiplied by the ratio of ReFH2.2 T-year event peak and ReFH2.2 100—year event peak. The same hydrograph shape was used as above based upon the October 2001 recorded event, which was scaled to agree with the peak flow estimates for each residual catchment • A check was then carried out to ensure that the hydraulic model design peak flow estimates at the River Ter @ Crabbs Bridge gauge (FEP03) agree with the design flow estimates presented above. If not, then there is scope to make minor adjustments to the residual catchment inflows to gain agreement if necessary. It was anticipated that minor reduction in the residual catchment inflows may be needed, though in practice this was not found to be necessary. The 5% and 1% AEP (20-year and 100-year return period) hydrographs for FEP01 is presented in Annex A.5. Flood hydrographs for all model inflow locations are saved in the following location: A12 Hydrological Calculation Sheet_Stage III_River Ter_Rev3.0 Typical hydrograph shapes for the 5% and 1% AEP events are presented in Annex A.5.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
EA	Environment Agency
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

References

Environment Agency (2017a). Flood Estimation Guidelines. Technical Guidance 197_08.

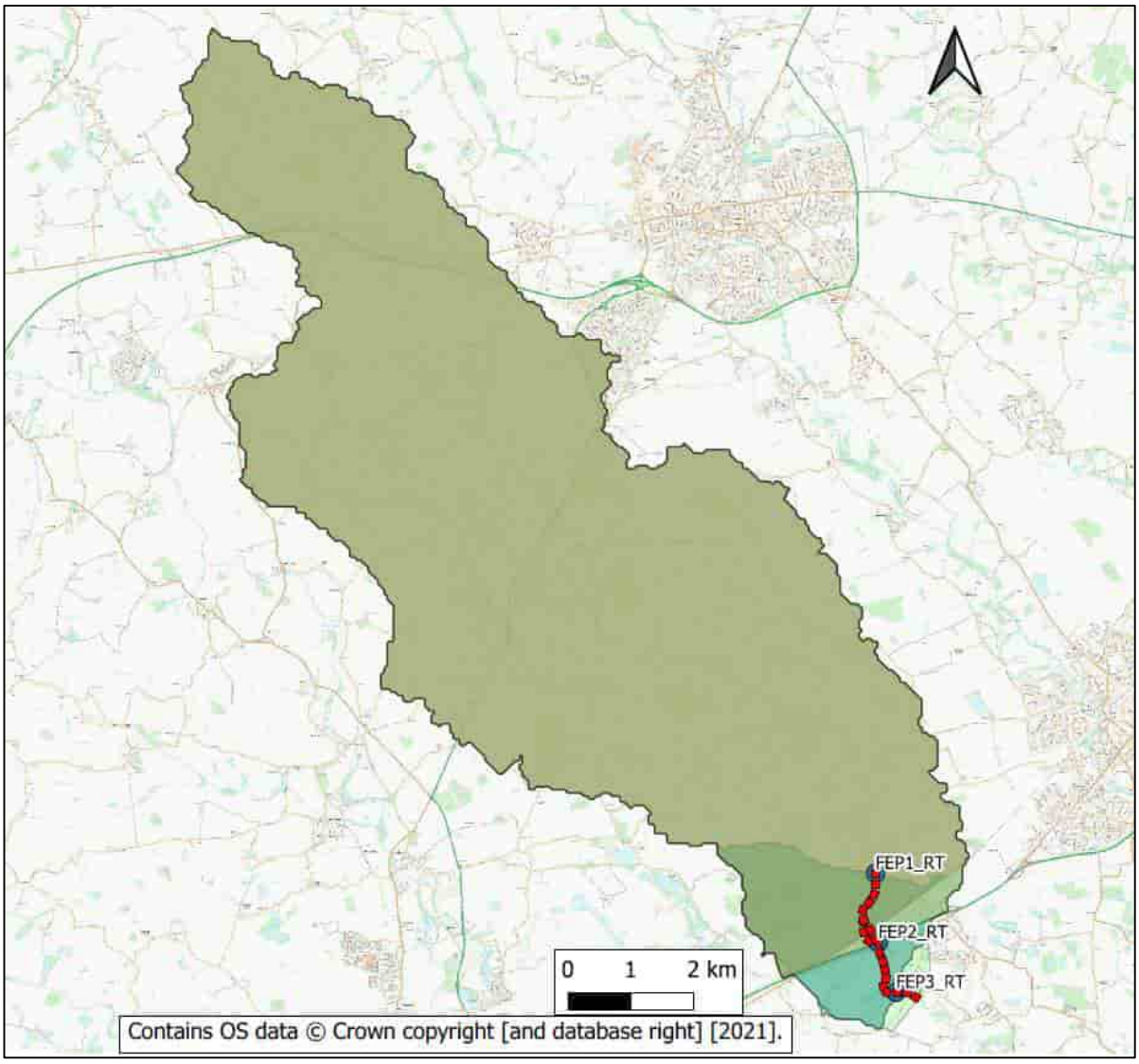
Environment Agency (2020a). Flood Estimation Guidelines. Technical guidance 197_08.

Environment Agency (2017b). Flood Risk Assessments: Climate Change Allowances guidance.

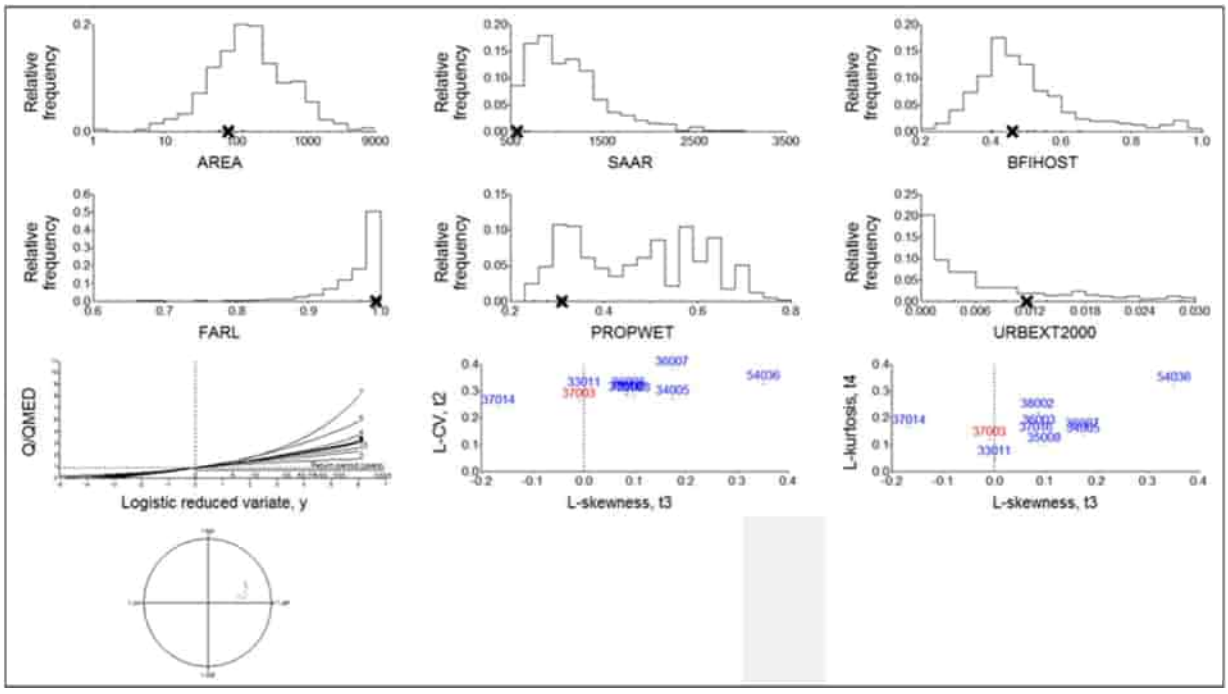
Environment Agency (2021 update). Flood Risk Assessments: Climate Change Allowances guidance.

Annex A Supplementary Information

A.1 Location of Catchment



A.2 Pooling Group Details - Graphs



A.3 Pooling Group Details - Table

A.3.1 Initial Pooling Group Details: (show default selection) – using Enhanced Single Site analysis method.

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
37003 (Ter @ Crabbs Bridge)	0	50	5.690	0.264	-0.009	0.066
37014 (Roding @ High Ongar)	0.281	51	11.1	0.24	-0.166	2.036
34012 (Burn @ Burnham Overy)	0.374	49	1.037	0.22	-0.142	1.529
36003 (Box @ Polstead)	0.504	54	3.935	0.304	0.087	0.460
26003 (Foston Beck @ Foston Mill)	0.548	55	1.718	0.249	0.012	0.443
34005 (Tud @ Costessey Park)	0.562	54	3.129	0.273	0.173	0.863
36007 (Belchamp Brook @ Bardfield Bridge)	0.576	50	5.025	0.383	0.173	1.620
33032 (Heacham @ Heacham)	0.584	47	0.453	0.311	0.109	1.180
37016 (Pant @ Copford Hall)	0.588	51	7.47	0.289	0.081	0.093
39042 (Leach @ Priory Mill Lechlade)	0.626	43	3.194	0.193	0.055	1.710
Total		504				
Weighted means		504		0.266	0.021	

A.3.2 Initial Pooling Group Catchment Descriptors: (show default selection) – using Enhanced Single Site analysis method.

Station	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37003 (Ter @ Crabbs Bridge)	0	77.76	570	0.115	0.994	0.012
37014 (Roding @ High Ongar)	0.281	92.65	597	0.107	0.986	0.008
34012 (Burn @ Burnham Overy)	0.374	83.87	668	0.098	0.997	0.005
36003 (Box @ Polstead)	0.504	56.72	566	0.093	0.993	0.012
26003 (Foston Beck @ Foston Mill)	0.548	59.59	698	0.106	0.987	0.004
34005 (Tud @ Costessey Park)	0.562	72.11	649	0.158	0.973	0.029
36007 (Belchamp Brook @ Bardfield Bridge)	0.576	58.16	560	0.079	0.996	0.004
33032 (Heacham @ Heacham)	0.584	56.16	688	0.116	0.983	0.006
37016 (Pant @ Copford Hall)	0.588	63.8	588	0.069	0.997	0.009
39042 (Leach @ Priory Mill Lechlade)	0.626	77.62	736	0.083	0.971	0.003

A.3.3 Final Pooling Group Details – using Enhanced Single Site analysis method

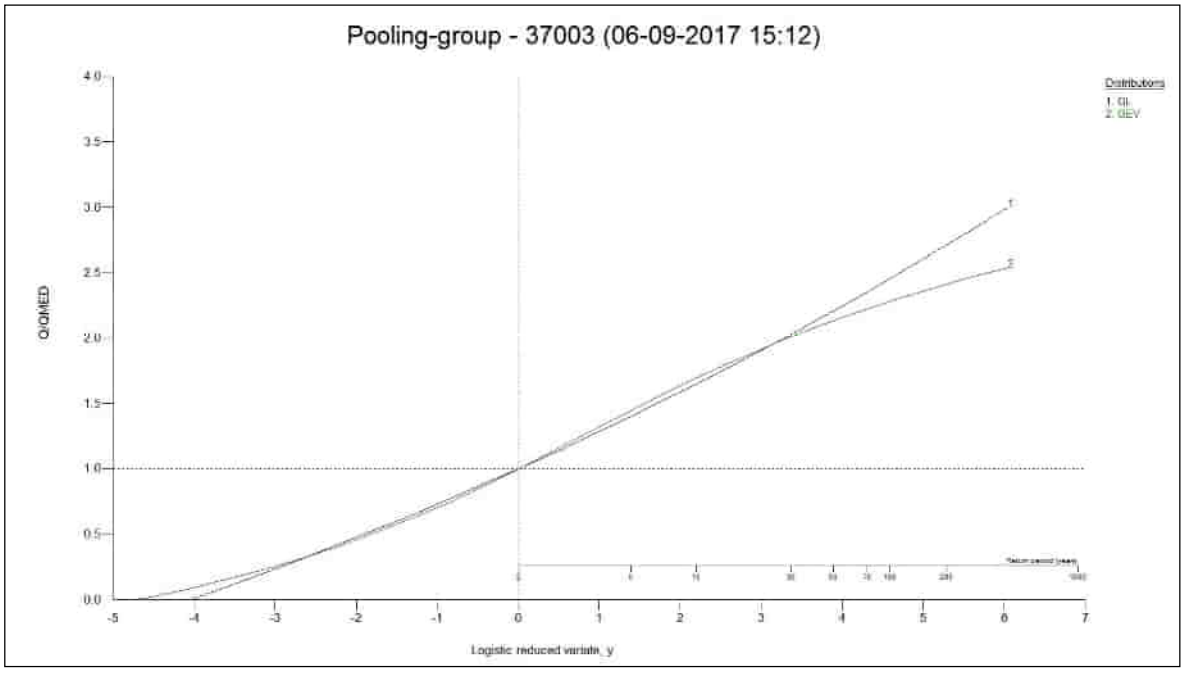
Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
37003 (Ter @ Crabbs Bridge)	0	50	5.69	0.264	-0.009	0.298
37014 (Roding @ High Ongar)	0.281	51	11.1	0.24	-0.166	2.066
36003 (Box @ Polstead)	0.504	54	3.935	0.304	0.087	0.040
34005 (Tud @ Costessey Park)	0.562	54	3.129	0.273	0.173	1.270
36007 (Belchamp Brook @ Bardfield Bridge)	0.576	50	5.025	0.383	0.173	2.235
37016 (Pant @ Copford Hall)	0.588	51	7.47	0.289	0.081	0.065
54036 (Isbourne @ Hinton on the Green)	0.693	42	13.992	0.329	0.352	2.125
35008 (Gipping @ Stowmarket)	0.698	49	13.078	0.286	0.097	0.536
38002 (Ash @ Mardock)	0.76	73	6.764	0.29	0.085	0.461
33011 (Little Ouse @ County Bridge Euston)	0.796	54	3.908	0.305	0.000	0.904
Total		528				
Weighted means				0.272	0.057	

A.3.4 Final Pooling Group Catchment Descriptors – using Enhanced Single Site analysis method

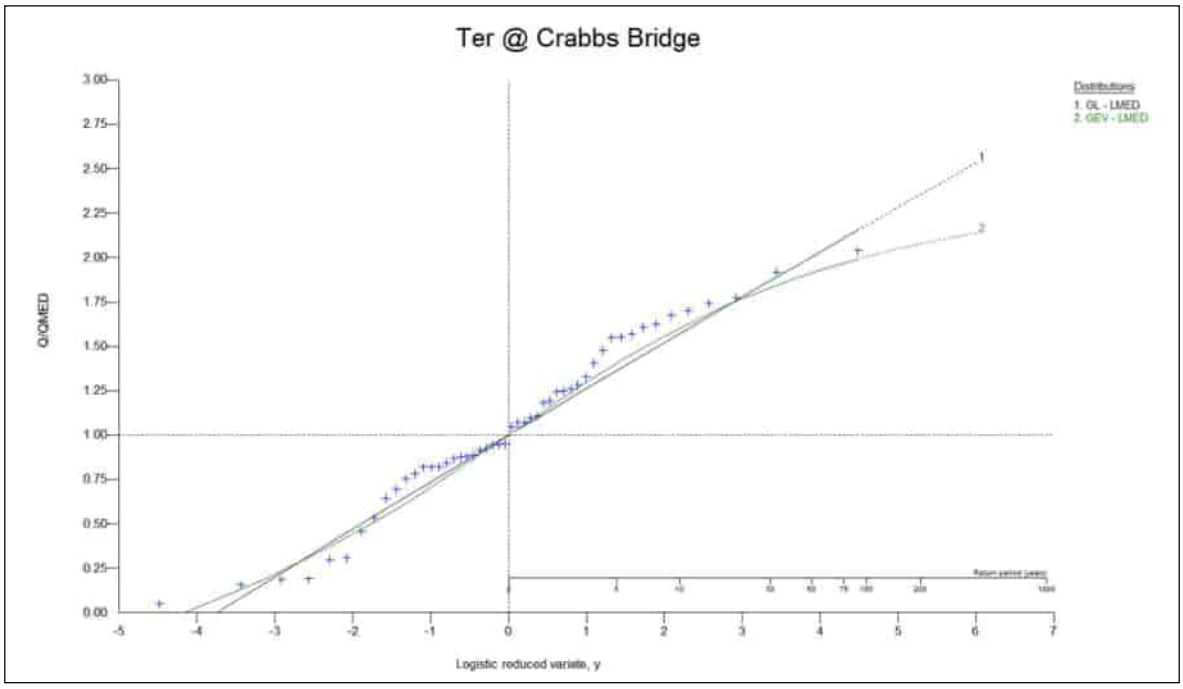
Station	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37003 (Ter @ Crabbs Bridge)	0	77.76	570	0.115	0.994	0.012
37014 (Roding @ High Ongar)	0.281	92.65	597	0.107	0.986	0.008
36003 (Box @ Polstead)	0.504	56.72	566	0.093	0.993	0.012
34005 (Tud @ Costessey Park)	0.562	72.11	649	0.158	0.973	0.029
36007 (Belchamp Brook @ Bardfield Bridge)	0.576	58.16	560	0.079	0.996	0.004
37016 (Pant @ Copford Hall)	0.588	63.8	588	0.069	0.997	0.009
54036 (Isbourne @ Hinton on the Green)	0.693	92.83	701	0.07	0.99	0.012
35008 (Gipping @ Stowmarket)	0.698	125.85	577	0.099	0.996	0.028
38002 (Ash @ Mardock)	0.76	78	619	0.049	1	0.014
33011 (Little Ouse @ County Bridge Euston)	0.796	129.35	596	0.146	0.985	0.008

A.4 Growth Curves

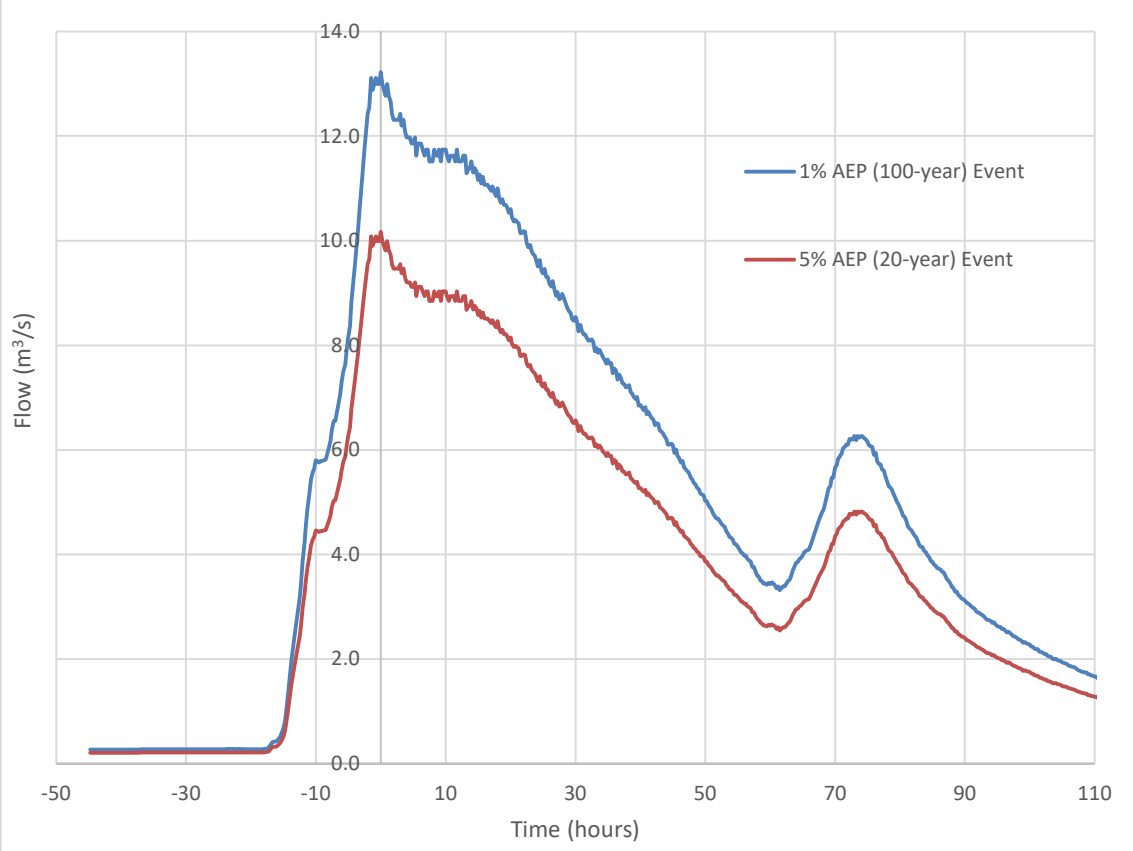
A.4.1 Enhanced Single Site Growth Curves



A.4.2 Single Site Growth Curves



A.5 5% and 1% AEP (20-year and 100-year return period) Event Hydrographs - FEP01



A.6 AMAX Data for the River Ter @ Crabbs Bridge

Water year	Date	Cumecs
1963	17/03/1964	6.80***
1964	05-Sep-65	3.05
1965	11-Dec-65	5.04
1966	11-Dec-66	7.08
1967	15-Jan-68	4.79
1968	14-Mar-69	7.56
1969	13-Mar-70	5.43
1970	24-Jan-71	4.67
1971	28-Jan-72	3.67
1972	07-Jul-73	1.06
1973	28-Sep-74	4.3
1974	23-Nov-74	10.1
1975	30-Jan-76	1.76
1976	22-Feb-77	7.18
1977	07-May-78	5.4
1978	02-Feb-79	7.3
1979	18-Mar-80	4.46
1980	16-Oct-80	0.958**
1981	30-Dec-81	9.67
1982	10-Dec-82	8.93
1983	27-Jan-84	4.66
1984	24-Nov-84	4.93
1985	27-Dec-85	5.95
1986	23-Aug-87	8.4
1987	16-Oct-87	9.53
1988	Missing period	n/a
1989	04-Feb-90	9.15
1990	19-Feb-91	0.883
1991	31-Mar-92	1.08
1992	27-Nov-92	5.22
1993	07-Jan-94	6.24
1994	30-Jan-95	7.1
1995	26-Feb-96	1.7
1996	26-Feb-97	0.299
1997	01-Aug-98	5.27

1998	28-Jan-99	6.09
1999	28-May-00	6.72
2000	08-Feb-01	9.91
2001	21-Oct-01	11.6
2002	01-Jan-03	8.85
2003	20-Aug-04	3.95
2004	20-Nov-04	5.37
2005	03-Nov-05	2.61
2006	15-Feb-07	4.99
2007	21-Jan-08	4.68
2008	10-Feb-09	10.9
2009	28-Feb-10	8
2010	18-Jan-11	9.25
2011	03-May-12	6.1
2012	20-Dec-12	6.3
2013	07-Feb-14	5
2014	24-Nov-14	8.82*

**AMAX flow provided by EA and added in WINFAP.*

***The 1980 AMAX flow has been excluded from the analysis as it is noted in WINFAP as being rejected.*

**** The 1964 AMAX is adopted from EA provided AMAX, which is much higher than that available in the WinFAP data.*

A.7 Environment Agency Email

From: Enquiries East Anglia [mailto:Enquiries_EastAnglia@environment-agency.gov.uk]
Sent: 21 July 2017 15:47
To: *****
Subject: EAn/2017/52400 - Hydrometric Data Request

Dear ****

Thank you for your enquiry which we received on 29 June 2017, requesting hydrometric data in relation to the A12 scheme.

Please find the attached the ratings that were requested, however the rest of the data is too large to send via email. We have therefore uploaded the data to the following sharefile link:

[redacted]

Please note that the ratings do not include bypass or non-modularity for which we may make adjustments for manually in the data provided. Please see the approximate Historic Modular Limit below:

Site	Historic Non Mod Limit (m)
Appleford Bridge	1.25
Bounstead Bridge	0.36
Crabbs Bridge	0.9
Guithavon Valley	0.95
Stisted	0.62

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

Please refer to the Open Government Licence available here: http://www.nationalarchives.gov.uk/doc/open-government-licence/version/3/ which explains the permitted use of this information.

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

Kind regards

Sarah Reedman

Customers & Engagement Officer, Customers & Engagement Team, East Anglia Area

Environment Agency | Bromholme Lane, Brampton, Huntingdon, Cambridgeshire, PE28 4NE

Environment Agency | Iceni House, Cobham Road, Ipswich IP3 9JD

[redacted]



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVER BRAIN MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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0 Executive Summary

0.1.1 This report describes the hydraulic modelling and the results obtained for the existing and proposed A12 highway crossing of the River Brain and Lower Blackwater (NGR 582808 213756). The modelling and hydrology have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1-dimensional/2-dimensional (1D/2D) linked type using Flood Modeller software (Jacobs 2022) for the 1D domain and TUFLOW (BMT 2022) for the 2D domain and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 38% climate change (CC). Flow reconciliation was carried out to match the design hydrology. Model calibration was not undertaken as it had previously been completed by CH2M in 2017 using historic events that were larger than any more recent events. The model results were verified through comparison to Environment Agency (EA) flood maps. The model was then used to simulate baseline and with-scheme scenarios for the three design events. In addition, a with-scheme temporary works scenario (that included haul roads) was simulated for the 5% (1 in 20) and 1% (1 in 100) AEP events.

0.1.3 The with-scheme scenario involved minor widening of the highway embankment on both sides and the extension of the existing bridge.

0.1.4 The analysis demonstrates that the proposed-scheme and its temporary works have negligible impact on fluvial flood risk at the River Brain and Lower Blackwater, the former including an allowance for climate change and therefore no mitigation is proposed.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA, which took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) were:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater (this report)
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Five minor watercourses were also modelled for the proposed-scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for the River Brain and Lower Blackwater to assess the baseline and with-scheme scenarios. This is a technical report, focused on the hydraulic modelling and catchment hydrology, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

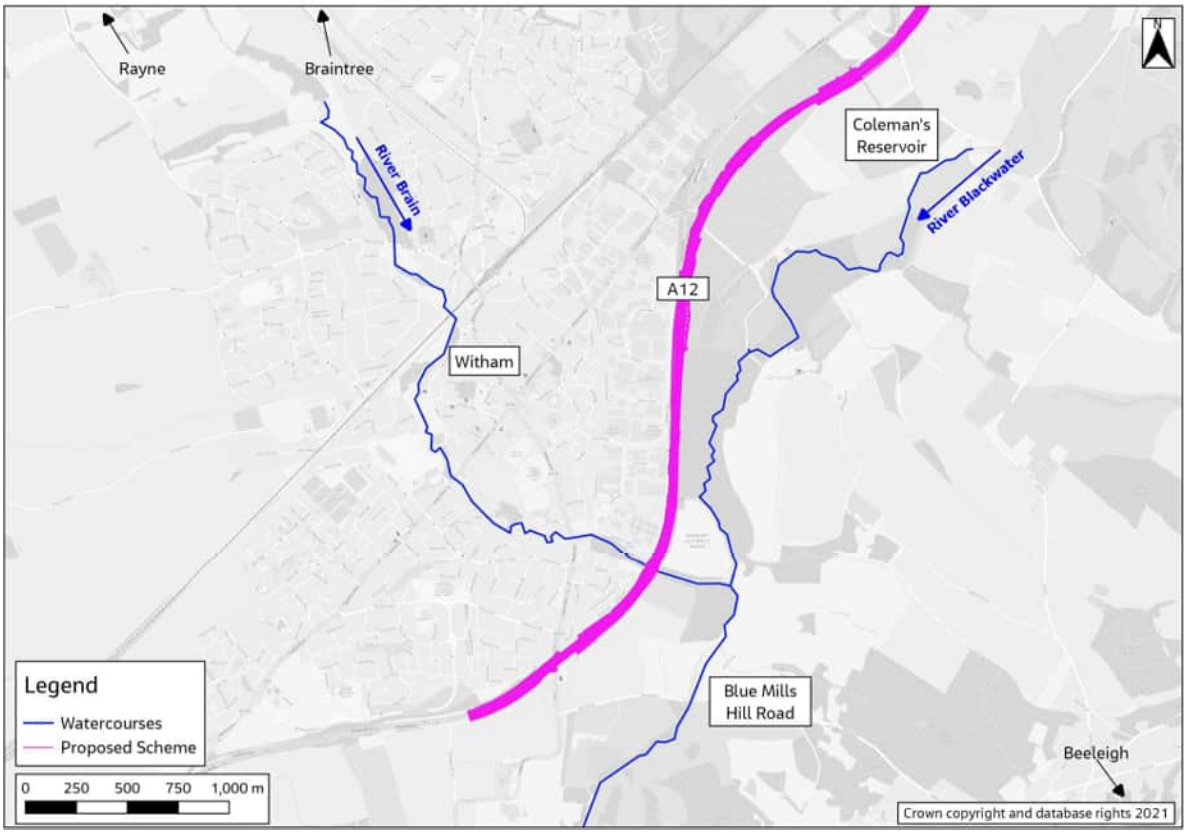
1.1.5 At the A12 crossing of the River Brain, the highway will be upgraded to three lanes per carriageway. This will require minor widening of the highway embankment on both sides and the extension of the existing bridge. Full details of the with-scheme arrangements are given in Section 8.1.

1.1.6 This report should be read in conjunction with the FRA.

1.2 Study area

1.2.1 The River Brain flows from the north-east of Rayne village in Essex to its confluence with the River Blackwater to the east of Witham. The model includes 2.7km of the River Brain, of which 385m is downstream of the A12. The River Blackwater flows from the north of Braintree to Beeleigh where it joins the River Chelmer. The model represents the Lower reach of the River Blackwater, covering 4.47km of the River Blackwater from (next to) Coleman's Reservoir to the south of Blue Mills Hill Road. Plate 1.1 shows the location of the River Brain and River Blackwater in relation to the proposed-scheme.

Plate 1.1 River Brain and River Blackwater study area



2 Input data

2.1.1 The data used to construct the hydraulic model of the River Brain and River Blackwater is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR	2m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection and Ranging) data. Used to represent the topography for the whole model extent	Environment Agency 2012
OS maps	Mastermap data	National Highways
Hydraulic model for River Brain	One-dimensional/Two-dimensional (1D/2D) model built with Flood Modeller and TuFLOW software - Brain_v36.DAT This version of the model was produced by CH2M in 2017	Environment Agency
Hydraulic model for River Blackwater	One-dimensional/Two-dimensional (1D/2D) model built with Flood Modeller and TuFLOW software - BWT_Lower_Model_SC_2016-07-19.DAT This version of the model was produced by CH2M in 2017	Environment Agency

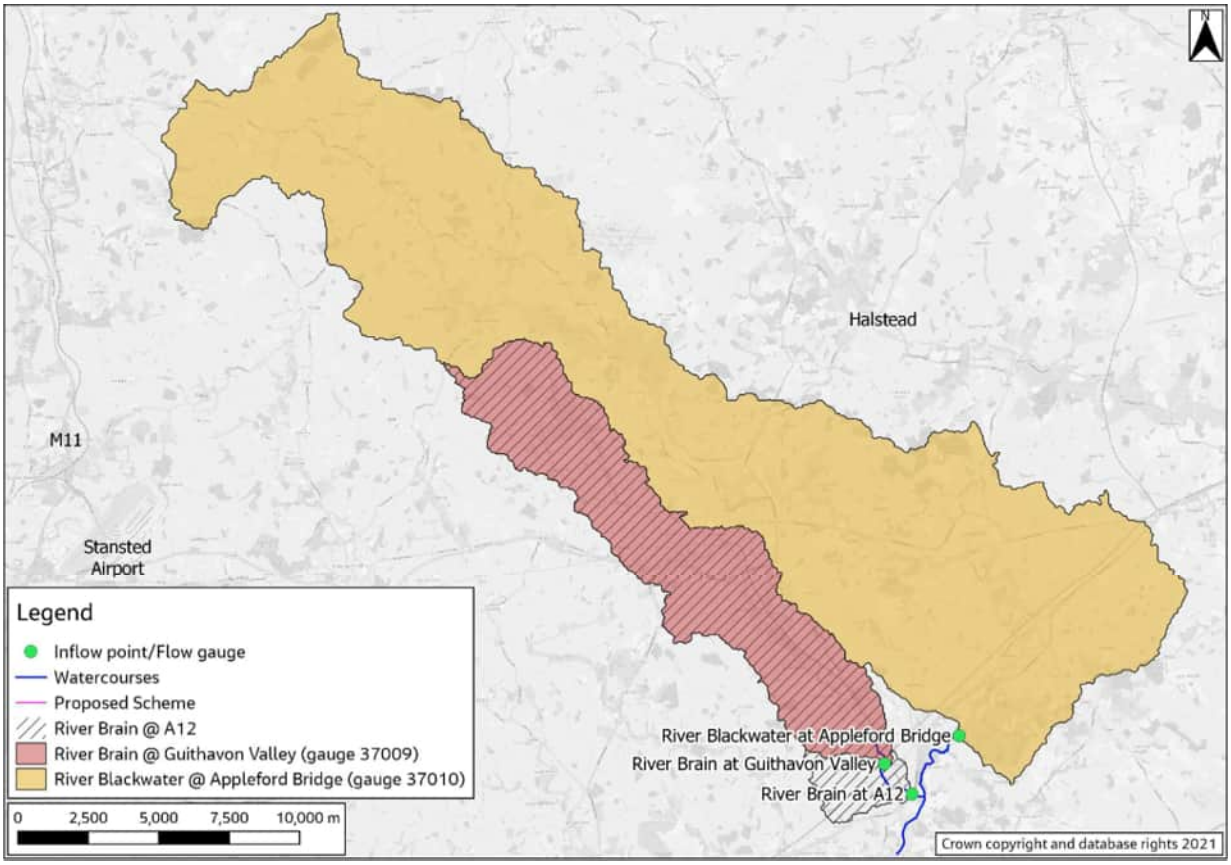
3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of the lower reach of the River Blackwater and River Brain for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event was also required including an allowance for climate change.

3.1.2 Plate 3.1 indicates the Lower River Blackwater and River Brain hydraulic model catchment extent including flow gauge locations within the modelled reaches.

Plate 3.1 Lower Blackwater and Brain model catchment



3.2 Background

3.2.1 Hydraulic modelling of the full River Blackwater and River Brain catchments was previously undertaken by CH2M in 2017 as part of a flood mapping exercise for the Environment Agency who provided them for use in this assessment. After a review of the CH2M hydrology it was decided to retain some of the original ReFH1 inflow units for the Lower River Blackwater and River Brain hydraulic models but also to update the

models for this assessment, to ensure the hydrology analysis met with the latest methods and guidance's. This included creating new ReFH1 inflow units for the upstream extent of the current modelled reaches (as shown in Plate 3.1) and updating peak flow estimates using updated methods of hydrological analyses and incorporating the most up-to-date hydrometric data post-2017 assessment.

3.2.2 The River Blackwater at Appleford Bridge (37010) and River Brain at Guithavon Valley (37009) gauging stations are both located within the hydraulic model extent. Hydrometric data available at these gauges have therefore been assessed and included in this analysis where appropriate. For full details of the estimation of target peak flows, derivation of hydraulic model inflow hydrographs and background hydrology refer to Appendix A.

3.3 Hydrological considerations

River Blackwater

3.3.1 The River Blackwater and River Brain are located in south east England. The River Blackwater has a catchment area of approximately 350km^2 to Maldon where it flows into the North Sea. The River Blackwater is a relatively rural catchment (arable, with areas of grassland and woodland) with urban areas including Braintree, Coggeshall, Witham and Maldon. The River Blackwater has five flow / level gauges in its catchment.

- Copford Hall (River Pant),
- Stisted,
- Appleford Bridge,
- Guithavon Valley (on the River Brain) and
- Langford.

The Appleford Bridge gauge (37010) is located within the modelled section of the River Blackwater. The River Blackwater flow regime is identified (National River Flow Archive data sheet) as being affected by groundwater abstractions and intermittently affected since 1971 by the Ely/Ouse Transfer Scheme pumping.

3.3.2 The predominant land use in the catchment is arable, with areas of grassland and woodland. The geology of the catchment has been assessed as being composed of predominantly London Clay with chalk in the headwaters, overlain with boulder clay and some sand and gravels. The soils within the catchment have been assessed using Landis online mapping (Landis 2020) as predominantly lime rich loamy and clayey soils with impeded drainage in the upper reaches and freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The catchment to the River Blackwater at

Appleford Bridge gauge (37010) has a BFIHOST of 0.475 and a SPRHOST of 40.1.

River Brain

3.3.3 The River Brain is a tributary of the River Blackwater and has a catchment area of approximately 68km^2 to their confluence. The River Brain has a moderately urbanised catchment with urban areas including Witham and Braintree. The River Brain has the Guithavon Valley (37009) gauging station located within its catchment, within the town of Witham. The predominant land use within the catchment is arable with areas of grassland and woodland.

3.3.4 The geology of the catchment is London Clay with superficial deposits of mainly bolder clay with some sands and gravels. The soils within the catchment have been assessed using Landis (2020) as being composed of predominantly lime rich loamy and clayey soils with impeded drainage, freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The River Brain catchment at the Guithavon Valley (37009) gauging station has a BFIHOST of 0.534 and a SPRHOST of 36.4.

3.4 Model inflow units

3.4.1 As noted in Section 3.2 new ReFH1 inflow units were created at the start of the modelled reaches; to the Appleford Bridge gauging station on the River Blackwater and for the River Brain to the Guithavon Valley gauging station. These new ReFH1 inflow units allowed assessment of flood risk in the vicinity of the proposed-scheme and reduce model computation time. The lateral ReFH1 inflow boundary units between the gauges and the confluence of the River Blackwater and River Brain were retained. The ReFH1 inflow units were used within the hydraulic model to derive the inflow flood hydrograph shape. The use of ReFH1 boundary units was assessed and judged to be acceptable for deriving hydrograph shape for these catchments. This was based on the assessment that there is limited floodplain storage within the modelled reaches, limited catchment change post the 2017 study and the results of the ReFH2.3 hydrograph shape sensitivity analysis (for the Middle Blackwater model) which showed that for the 1%(1 in 100) AEP event the ReFH1 hydrograph shape (scaled to the final peak flow estimates) produced a more conservative flood extent outline at the A12 crossing location of the River Blackwater than when the ReFH2.3 flood hydrograph shape was adopted (for further details refer to Appendix A).

3.5 Peak flow derivation

3.5.1 In order to incorporate increased availability of flow data within the area of the proposed-scheme since the CH2M study was undertaken in 2017 and to include updates to software such as WINFAP and ReFH, design peak flow estimates were updated as part of this study using a variety of

methods, including FEH (Flood Estimation Handbook) Statistical (using both simple Pooling Group and Enhanced Single Site (ESS) analyses) and ReFH2.3 rainfall runoff method. As two Peak Flow gauging stations: River Blackwater at Appleford Bridge (37010) and River Brain at Guithavon Valley (37009), are located within the modelled reaches of the River Blackwater and River Brain respectively the hydrometric data available at these gauges have also been evaluated and further analysed where appropriate.

3.5.2 The Appleford Bridge gauge (37010) hydrometric data was reviewed to estimate peak flows for the River Blackwater. This Environment Agency gauging station has a flood record from 1962 to present. Its flood record is classified in the Peak Flows database as suitable for both QMED estimation and pooling group analysis. Review of the gauging station by the National River Flow Archive indicates some uncertainty associated with the higher flow estimates, but the gauge is believed to perform satisfactorily at QMED. QMED was therefore derived for the River Blackwater at Appleford Bridge using the AMAX data recorded at the gauge. Standard FEH Statistical Pooling and Enhanced Single-Site (ESS) analyses were undertaken to derive peak flow estimates at the gauging station for up to and including the 1% (1 in 100) AEP event. ReFH2.3 method was also used (with default catchment descriptors) in order to derive peak flow estimates. After reviewing the peak flow estimates and growth curves produced by these methods it was decided to take forward the higher of the FEH Statistical Pooling and ESS analysis growth curves for the River Blackwater at Appleford Bridge (gauge 37010). The FEH Statistical ESS growth curve was therefore adopted for flows up to and including the 1% (1 in 100) AEP event. A figure showing the different flood frequency curves for the River Blackwater at Appleford Bridge is provided in Appendix A. For consideration of rarer events, the Environment Agency guidance was followed which indicates that these flows can be estimated by multiplying the FEH Statistical 1% (1 in 100) AEP flow by the ratio of the ReFH2.3 derived T-year peak flow, QT (for T>100-years) and the ReFH2.3 derived 1% (1 in 100) AEP flow, Q100.

3.5.3 The Environment Agency River Brain gauging station at Guithavon Valley (37009) also has a flood record from 1962 to present. Its flood record is classified in the Peak Flows database as suitable for both QMED estimation and pooling group analysis. There is some uncertainty regarding the higher AMAX flows for this gauge. Due to this uncertainty the CH2M 2017 study derived a rating for the gauge using their hydraulic model. This, however, produced a very steep growth curve. It appears the Environment Agency have reviewed the AMAX series since the CH2M 2017 study and have preferred estimates that make explicit allowance for non-modularity. These adjustments seem to alter the higher annual maximum flows on the record. The updated AMAX data appears in the current National River Flow Archive (NRFA) Peak Flows dataset and was therefore used in this assessment. QMED was derived for the Guithavon Valley gauging station using the current AMAX data recorded at the

gauge. Standard FEH Statistical Pooling and Enhanced Single-Site (ESS) analysis were therefore used to derive growth curves and thereby peak flow estimates for up to and including the 1% (1 in 100) AEP event. ReFH2.3 was also used (with default catchment descriptors) in order to derive peak flow estimates. After reviewing the peak flow estimates (see Appendix A for River Brain at Guithavon Valley Flood Frequency Curves) and growth curves produced by these methods it was decided to take forward the higher of the FEH Statistical Pooling and ESS analysis for the River Brain at Guithavon Valley (37009) gauge. The FEH Statistical ESS growth curve was therefore adopted for flows up to and including the 1% (1 in 100) AEP event. For consideration of rarer events the Environment Agency (EA) guidance was followed which indicates that these flows can be estimated by multiplying the FEH Statistical 1% (1 in 100) AEP flow by the ratio of the ReFH2.3 derived T-year peak flow, QT (for T>100-years) and the ReFH2.3 derived 1% (1 in 100) AEP flow, Q100.

3.5.4 At the other flow estimation point, the A12 crossing of the River Brain the AEP flows were derived using the following approach:

- QMED was derived using catchment descriptors (CDs); and adjusted using the River Brain at Guithavon Valley gauge as the donor. These estimates were extended to rarer events up to and including the 1% (1in 100) AEP event by multiplying the QMED by the flood growth curve derived from the ESS analysis at the River Brain at Guithavon Valley gauge

- The ratio of the ReFH2.3 estimates of the QT/Q100 was again used to extend the flood frequency curve from the 1% (1 in 100) AEP event to rarer events

3.5.5 The modelled flows obtained from the application of inflow hydrographs from the ReFH1 method were reconciled with the corresponding peak flows at the three target locations, as presented in Table 3.1.

Table 3.1 River Blackwater Lower and River Brain peak flows (m^3/s) for range of AEPs at the three target locations

Inflow	Peak Flow (m^3/s)										
	AEP 50%	AEP 20%	AEP 10%	AEP 5%	AEP 3.33%	AEP 2%	AEP 1.33%	AEP 1%	AEP 1% +38%CC uplift	AEP 0.5%	AEP 0.1%
	1 in 2	1 in 5	1 in 10	1 in 20	1 in 30	1 in 50	1 in 75	1 in 100	1 in 100+CC	1 in 500	1 in 1000
River Blackwater @ Appleford gauging station	13.2	18.9	22.9	27.2	29.8	33.3	36.3	38.5	53.2	46.2	71.4
River Brain @ Guithavon Valley gauging station	3.9	6.2	7.8	9.5	10.6	12.1	13.3	14.3	19.7	17.6	28.5
River Brain @ A12	4.5	7.2	9.0	11.1	12.3	14.0	15.5	16.6	22.9	20.4	33.0

3.6 Model runs and the critical storm duration

Introduction

3.6.1 As the Lower Blackwater and Brain Model incorporates both the River Blackwater and the River Brain watercourses which both flow in close proximity to the proposed-scheme (with the River Brain crossing the A12), the following two model runs were undertaken, to ensure the worst-case flood risk scenario was identified for each of the river systems:

- Run 1 - using the River Blackwater critical storm duration and rainfall for all catchments/sub-catchments; and
- Run 2 - using the River Brain critical duration and rainfall for all catchments/sub-catchments.

Findings of the two scenario model assessments are provided in Section 5.

Run 1 simulation

3.6.2 For the Run 1 simulation, the Lower River Blackwater and River Brain hydraulic model required a total of five inflows, including two-point inflows (at the top ends of the River Blackwater and River Brain models) and three lateral inflows from the residual catchments, as follows:

- Inflow 1 – River Blackwater at Appleford Bridge gauging station
- Inflow 2 – River Brain at Guithavon Valley gauging station
- Lateral – B60int (as per CH2M design hydrology) from the River Blackwater residual catchments between the Appleford Bridge gauging station and River Brain confluence
- Lateral – BRA30int (as per CH2M design hydrology) from the River Brain residual catchment between Guithavon Valley gauging station and the River Blackwater confluence
- Lateral – B80int (as per CH2M design hydrology) from the River Blackwater residual catchment downstream of River Blackwater and River Brain confluence up to the Blackwater downstream modelling extent

3.6.3 For Run 1 the critical duration for the River Blackwater is 29.7 hours, which was derived using the ReFH1 unit for the River Blackwater at Appleford Bridge (default FEH catchment descriptors). In order to assess the potential worst-case flood scenarios from the River Blackwater at the A12 road improvements, the model was run using the River Blackwater critical storm duration and rainfall for both the River Blackwater and the River Brain catchments.

3.6.4 The modelled flows have been reconciled with the target peak flow at the Appleford Bridge gauging station on the River Blackwater using suitable

scaling factor applied across the model. The following scaling factors have been applied to all inflows;

- 5% (1 in 20) AEP event = 0.75; and
- 1% (1 in 100) AEP event = 0.72.

Run 2 simulation

3.6.5 For the Run 2 simulation, the Jacobs Lower River Blackwater and River Brain hydraulic model requires two inflows; one inflow is required at the top end of the River Brain hydraulic model and one lateral inflow:

- Inflow 2 – River Brain at Guithavon Valley gauging station; and
- Lateral – BRA30int (as per CH2M design hydrology) from the River Brain residual catchment between Guithavon Valley gauging station and the River Blackwater confluence.

3.6.6 In addition to the two inflows for the River Brain, a steady state flow equivalent to the 3.33% (1 in 30) AEP event peak flow (for the River Blackwater at Appleford Bridge) was also required for the River Blackwater, to be applied at the downstream boundary of the River Brain model. The 3.33% (1 in 30) AEP event for the River Blackwater was selected based on the outcome of a sensitivity analysis of the model downstream boundary which showed the model was largely insensitive to the event simulated as the downstream boundary for the River Blackwater.

3.6.7 For Run 2 within the Lower Blackwater and Brain model, the ReFH1 calculated critical storm duration for the River Brain to the A12 crossing (15.9 hours) has been used for all catchments.

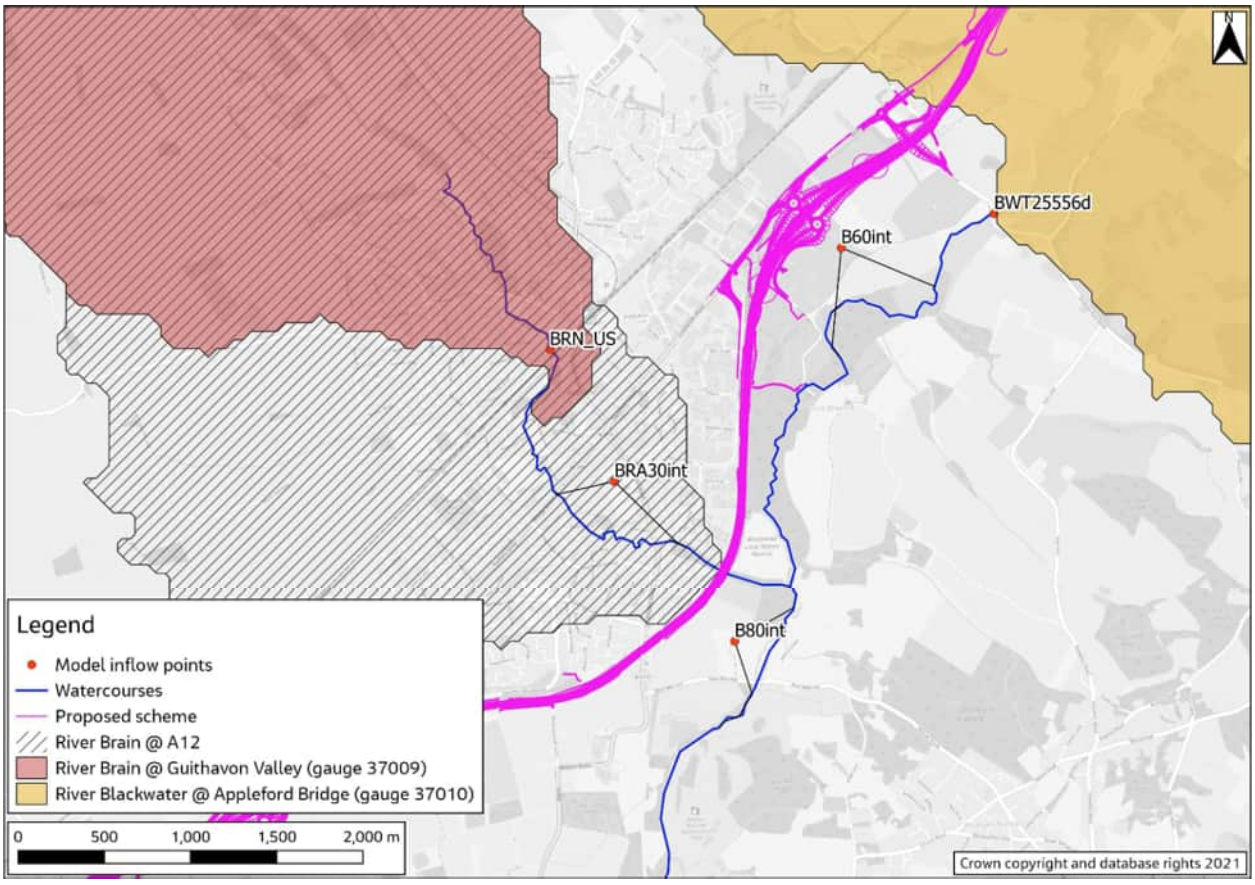
3.6.8 The modelled flows have been reconciled within the target peak flow at the A12 crossing, by applying appropriate scaling factor to the lateral inflow from the residual catchment between Guithavon Valley gauging station and at the River Brain A12 crossing (BRA30int), while applying a pre-fixed scaling factor to the inflows at the gauge (which is equal to the ratio of the FEH Statistical and ReFH1 peak at the gauge). The following scaling factors have been applied for the 5% (1 in 20) AEP and 1% (1 in 100) AEP;

- River Brain at Guithavon Valley gauging station 5% (1 in 20) AEP event = 0.75
- River Brain at Guithavon Valley gauging station 1% (1 in 100) AEP event = 0.74
- BRA30int 5% (1 in 20) AEP event = 1.20
- BRA30int 1% (1 in 100) AEP event = 1.05

3.7 Model inflow locations

3.7.1 Plate 3.2 presents the model inflow locations and indicates lateral inflow reach locations.

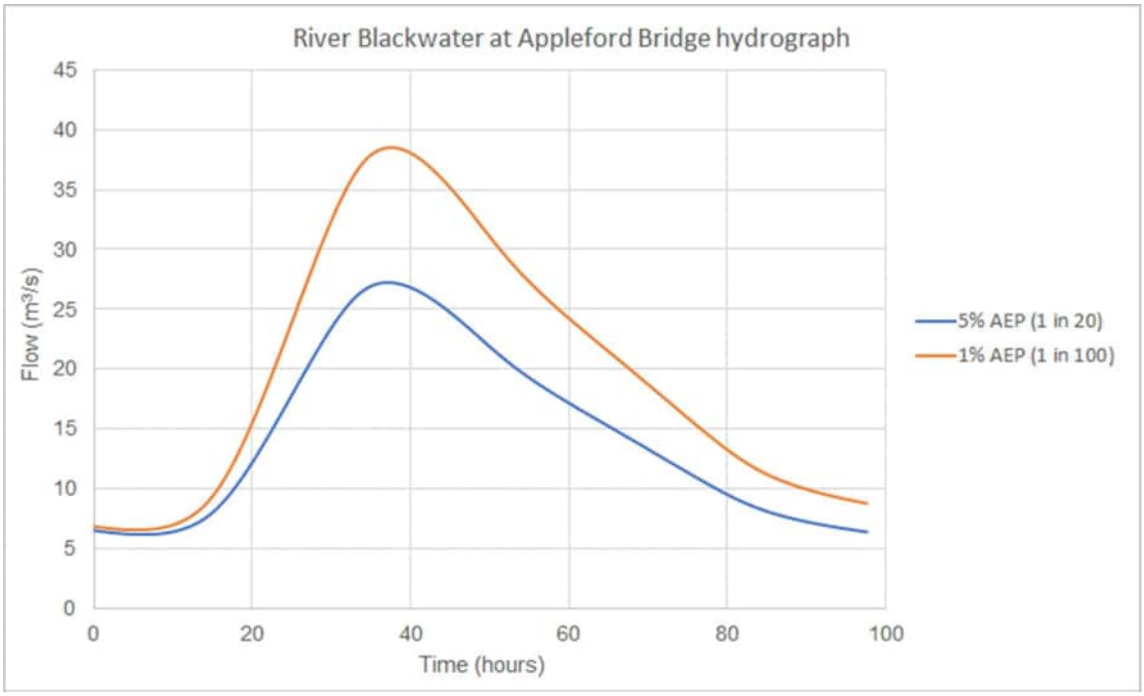
Plate 3.2 Schematisation of the River Blackwater and Brain model inflows



Inflow hydrographs

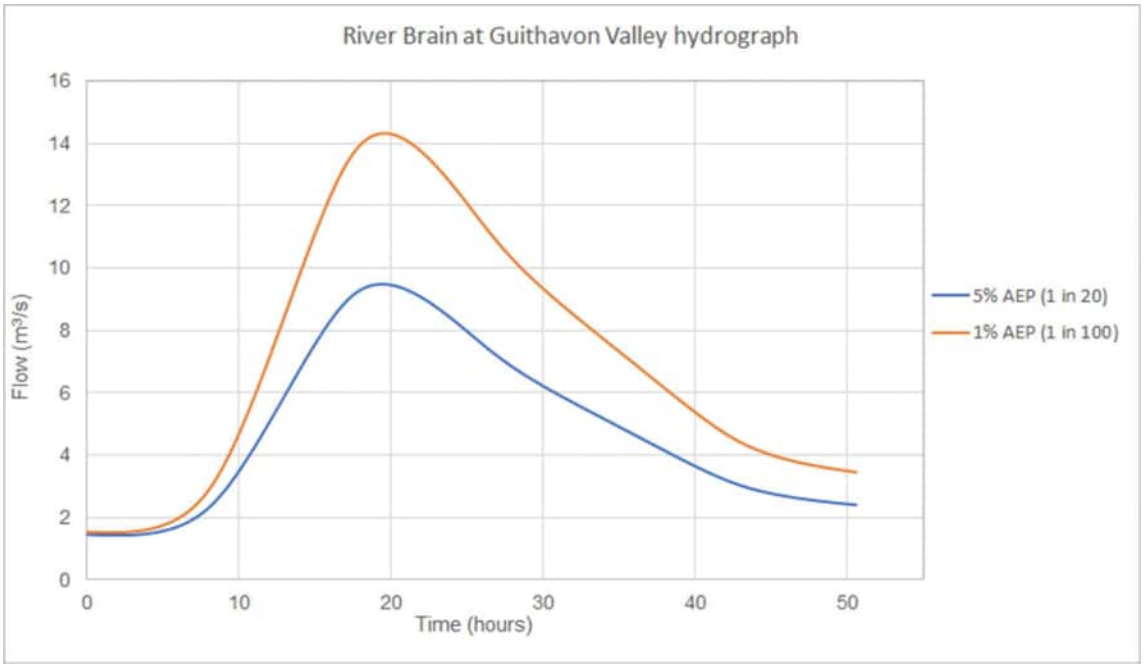
3.7.2 Plate 3.3 presents the 5% (1 in 20) AEP and 1% (1 in 100) AEP inflow hydrographs of River Blackwater at Appleford Bridge for the 29.7 hours critical storm duration of the River Blackwater (Run 1) from ReFH1 method scaled to the adopted peak flow estimate at the gauge.

Plate 3.3 Run 1 hydrographs (River Blackwater at Appleford Bridge)



3.7.3 Plate 3.4 presents the 5% (1 in 20) AEP and 1% (1 in 100) AEP inflow hydrographs of River Brain at Guithavon Valley for the 15.9 hour critical storm duration (Run 2) from ReFH1 method scaled to the adopted peak flow estimate at the gauge.

Plate 3.4 Run 2 hydrographs (River Brain at Guithavon Valley)



3.8 Climate change

3.8.1 A climate change allowance for peak river flows was obtained from the latest government guidance at the time of assessment (Environment Agency 2021). This produced an uplift allowance of +38% (the Combined Essex Management catchment, higher 2080s scenario) which was applied to the 1% (1 in 100) AEP design events giving a peak flow of 53.2m^3/s for the River Blackwater at Appleford Bridge gauge (37010) and 19.7m^3/s for the River Brain at Guithavon Valley gauge (37009).

3.9 Summary

3.9.1 Design peak flow estimates for the Rivers Blackwater and Brain were derived using the FEH methodology applying the available hydrometric data at the gauges, catchment descriptors available from the FEH Web Service; and adopting the climate change allowance as per latest EA guidance. The adopted hydrograph shape was from the ReFH1 model. Design inflow hydrographs were derived for both watercourses for two hydraulic model runs, namely, Run 1 – for the longer storm duration (same as the CSD of the River Blackwater at Appleford Bridge), and Run 2 – for the individual critical storm duration for the River Brain to the A12 crossing. After undertaking the hydraulic analysis with both sets of inflow hydrographs, it was noted that Run 1 resulted in the worst-case scenario for the River Blackwater. This is as expected given the River Blackwater at Appleford Bridge ReFH1 critical storm duration was adopted. For the River Brain, the difference between the resulting flood extent from the two model runs was noted to be negligible. The results of Run 1 have therefore been presented for both watercourses. For further details refer to Section 6.

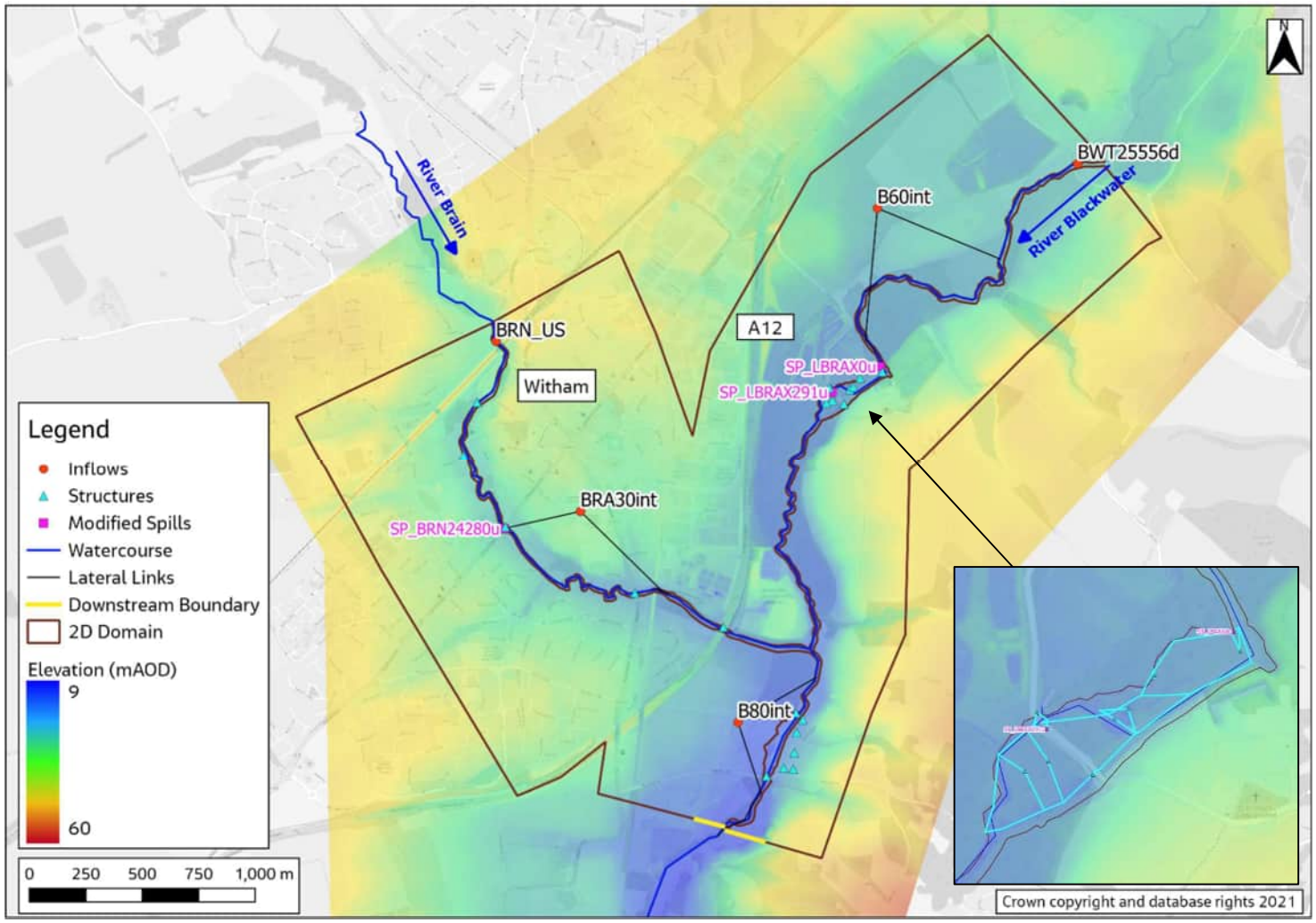
4 Baseline modelling

4.1.1 The hydraulic model was built using a linked 1D/2D schematisation, where the river channel is represented as a 1D component and is linked to the floodplain, represented by a 2D domain. A model for each watercourse: the River Brain and River Blackwater, was provided (Brain_v36.DAT and BWT_Lower_Model_SC_2016-07-19.DAT), by the Environment Agency in 2017. Both models utilise a 1D/2D schematisation. These models were updated for the present study using the latest available software versions. The river modelling package Flood Modeller Pro (version 4.6) was used for the 1D component and TUFLOW (version 2018-03-AE-iSP-w64) for the 2D component. Once updated, they were joined to create a new combined model for this assessment.

4.2 Model extent

4.2.1 The model utilised for the assessment of the proposed-scheme is an upgraded version of that developed by CH2M 2017 for the River Blackwater and River Brain. The previous models covered a more extensive area than required for this study, therefore both models were cropped. The River Brain model was trimmed in its upstream part and the Lower Blackwater model its downstream part. In addition, the flooding mechanisms around the proposed-scheme has components from both watercourses, as a consequence of the confluence of the River Brain with the River Blackwater. Therefore, both models were combined into a single model (see Plate 4.1).

Plate 4.1 River Brain and Lower Blackwater baseline model



4.3 Watercourse schematisation – 1D domain

In channel geometry

4.3.1 New topographic survey was not required since the source data for the incoming models were believed to be sufficiently recent (2010) for the purpose of the assessment. River section schematisation has been reviewed and modified where necessary along the River Brain and the River Blackwater. The main updates to the 1D model geometry are as follows:

- The total length of the incoming model river reaches was checked using GIS and found to be incorrect. The cross section spacing was therefore reviewed and corrections made as required
- To aid model performance at Saul's Bridge on the River Brain (model node: BR_BRN24280u), interpolated cross sections were added upstream and downstream from the structure. The spacing of the interpolated cross sections ranged from 30m to 45.8m
- 1D cross section widths were adjusted in order to define more appropriately the connection between the 1D and 2D model domains. These 1D changes were only required for the Lower Blackwater model as the River Brain 1D/2D linkage was found to be appropriately schematised
- Wings were added at both sides (BWT28150, BWT28650, BWT28900) and at the right side only (BWT28978ui, BWT28978di, BLUE336) in selected cross sections in the Lower Blackwater model to improve model stability
- Corrections were applied to several cross sections in order to better represent the bifurcation of River Blackwater in the area of Little Braxted Lane. The area between the channel branches is represented within the 1D domain. Therefore, spill units were added to cross connect the area in between watercourses (see inset of Plate 4.1)

In channel hydraulic friction

4.3.2 Table 4.1 shows the roughness utilised within the model.

Table 4.1 Manning's 'n' coefficients – 1D domain

Reach	In channel (Manning's 'n')	Bankside (Manning's 'n')
Lower Blackwater	0.04	0.04
River Brain - BRN23306d to BRN24000	0.04	0.05
River Brain - BRN24280u to BRN24286d	0.04	0.04
River Brain - BRN24500 to BRN25570d	0.04	0.05

Reach	In channel (Manning's 'n')	Bankside (Manning's 'n')
River Brain - BRN25635 to BRN26000	0.04	0.04

4.3.3 The hydraulic roughness (Manning's 'n' coefficient) in the incoming models was not modified as the values applied were considered to be appropriate based on an inspection of the watercourse from the available aerial and street-view photography. Manning's 'n' roughness values were set to 0.04 for the channel bed. According to Chow 1959, n = 0.04 as a Manning's 'n' roughness represents clean, winding watercourses with some pools and shoals in main channels. Some cross sections adopt n = 0.05 for banktops which corresponds to light brush and trees.

In channel hydraulic structures

4.3.4 In general, hydraulic structure dimensions were not modified from the previous models and are assumed to be correct. However, several weir coefficients were modified:

- SP_LBRAX0u (River Blackwater). Weir coefficient was changed from 2.0 to 1.4 which is deemed more suitable for a spill representing an in-channel lateral connection between the river and the by-pass channel (see inset of Plate 4.1 for location).
- SP_LBRAX291u (River Blackwater). Weir coefficient was changed from 1 to 1.2 because this value better represents the road over the bridge at Little Braxted Lane (see inset of Plate 4.1 for location).
- SP_BRN24280u (River Brain). Weir coefficient was changed from 2 to 1.5, this value better represents a bridge deck with a road and fence.

Boundary conditions – 1D domain

4.3.5 The upstream and downstream boundary conditions applied to the 1D domain are given in Table 4.2. Inflow locations are shown in Plate 4.1.

Table 4.2 Boundary conditions – 1D domain

Flood Modeller Node	Type of Boundary	Description
BWT25556d	Flow-Time (QT) Boundary	Scaled ReFH inflow boundary, applied at the upstream end of River Blackwater at cross section BWT25556d (see Plate 4.1).
B60int	ReFH Boundary	Scaled ReFH lateral inflow boundary, distributed along River Blackwater between the nodes BWT26150 and BWT27150 (see Plate 4.1).
B80int	ReFH Boundary	Scaled ReFH lateral inflow boundary, distributed along River Blackwater between the nodes BWT29150 and BWT29650 (see Plate 4.1).
BRN_US	Flow-Time (QT) Boundary	Scaled ReFH inflow boundary, applied at the upstream end of River Brain at cross section BRN23306d (see Plate 4.1).
BRA30int	ReFH Boundary	Scaled ReFH lateral inflow boundary, distributed along River Brain between the nodes BRN24286d and BRN25250 (see Plate 4.1).
BWT29900	Flow-Head (QH) Boundary	Stage discharge boundary extracted from the incoming model results and applied at the downstream extent of the truncated model (see Plate 4.1).

4.4 Floodplain schematisation – 2D domain

Floodplain topography

4.4.1 The 2D domain covers an area of 6.24km^2 along the River Brain and River Blackwater that encompasses the proposed-scheme (see Plate 4.1). The topography is represented using a 10m resolution square grid. The levels for the grid cells are based on a Digital Terrain Model (DTM) derived from Open LiDAR 1m resolution flown in 2012.

4.4.2 The 10m grid resolution was deemed appropriate following model test runs, these showed that water level results and flood mechanisms in key locations required for flood risk assessment of the proposed A12 scheme were not sensitive to the implementation of a reduced 2D cell size.

4.4.3 Only one modification has been made to the DTM. At Colemans and The Willows Reservoirs, z-shape polygons were used to depict the reservoirs being full at the onset of the simulation. The incoming model representation had variable levels across the reservoir surfaces, due to the LiDAR return being affected by the water surface. The correction enforces a flat surface. The levels used were 14.5m AOD for Colemans

Reservoir and 13.5m AOD for The Willows Reservoir. They have been estimated from inspection of the levels at the reservoir edge as shown by the LiDAR.

Floodplain hydraulic friction

4.4.4 Hydraulic roughness coefficients are applied to each grid cell of the 2D domain based on land use, which was taken from OS MasterMap data. These Manning's 'n' values have not been changed from the provided models except for building roughness which has been changed from 0.5 to 1.0. The values adopted are included in Table 4.3.

Table 4.3 Manning's 'n' coefficients – 2D domain

Land use	Manning's 'n'
Natural environment – heavy woodland and forest	0.100
Roads, tracks and paths manmade Roads tracks and paths tarmac Roads track and paths (roadside) pavement	0.020
General surface – step Roads tracks and paths tarmac or dirty tracks	0.025
General surface – grass, parkland Structures roadside	0.030
Water inland Water tidal Land (unclassified) industrial yards, car parks	0.035
Default General surface – residential yards Water foreshore Slope Pylon Land, assume grass Cliff, assumed step	0.040
Glasshouses Land heritage and antiquities Rail Structures generally on top of buildings	0.500
Buildings	1.000

Boundary conditions – 2D domain

4.4.5 No inflows have been applied directly in the 2D domain. However, to allow flood waters reaching the 2D domain downstream extent to leave the modelled area, two water level-flow (HQ) boundaries were set up (see Plate 4.1). These were placed adjacent to the River Blackwater at the downstream end of the model. The slope of the rating curve for both

boundaries was calculated as an average of the LiDAR slope in the direction of flow.

1D/2D linkage

4.4.6 The link between the 1D and the 2D domains was modified in order to improve the match between 1D top of bank elevation within the cross sections and the 2D top of bank elevation depicted by the LiDAR. These alterations were as follows for each watercourse:

- River Brain: Only a few alterations in specific locations were made to the 1D/2D linking layer. The incoming model depicted the top of bank levels in a generally accurate manner
- River Blackwater: The incoming model top of bank boundary line was defined very coarsely. The 1D/2D link was entirely re-aligned in order to better match with the top of bank depicted from the LiDAR

5 Modelled events

5.1.1 Table 5.1 shows the events and scenarios that were simulated with the hydraulic model.

5.1.2 As already mentioned, the Lower Blackwater and Brain Model incorporates both the River Blackwater and the River Brain watercourses which both flow in close proximity to the proposed road improvements (with the River Brain crossing the A12). Therefore, the following two model runs were undertaken:

- Run 1 - using the River Blackwater critical storm duration and rainfall for all catchments/sub-catchments
- Run 2 - using the River Brain critical duration and rainfall for all catchments/sub-catchments.

5.1.3 For both Run 1 and Run 2 simulations, flow reconciliation was performed as described in Section 3.6. The Run 1 simulations were seen to be the worst-case scenario on the River Blackwater, whilst the difference between the two scenarios for the River Brain was found to be negligible. Therefore, only the Run 1 results are presented in this report. Flood extents for Run 2 simulations are presented in Appendix D.

5.1.4 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope.

Table 5.1 Modelled events

Scenario	AEP Event/Return Period		
	5%	1%	1% +38%CC
	1 in 20	1 in 100	1 in 100+CC
Baseline – Run1	X	X	X
Baseline – Run2	X	X	
Roughness Sensitivity (1D and 2D) – Run1		X	
Hydrological Inflow Sensitivity – Run1		X	
Downstream Boundary Sensitivity (1D and 2D) – Run1		X	
With-scheme – Run 1	X	X	X
With-scheme and temporary works	X	X	

6 Model proving

6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution, within a specified number of iterations, for each model time step. The convergence of the 1D model was checked and is shown in Plate 6.1 and Plate 6.2 for the baseline and with-scheme scenarios respectively. Convergence for the 1% (1 in 100) AEP+ 38%CC flood events are good throughout the run durations. This convergence plot is typical for the events modelled. The mass error reported in the 1D model is around 0.1% of the total inflow volume to the 1D domain which is considered to be acceptable.

Plate 6.1 1D model convergence – 1% (1 in 100) AEP +38%CC event – baseline scenario

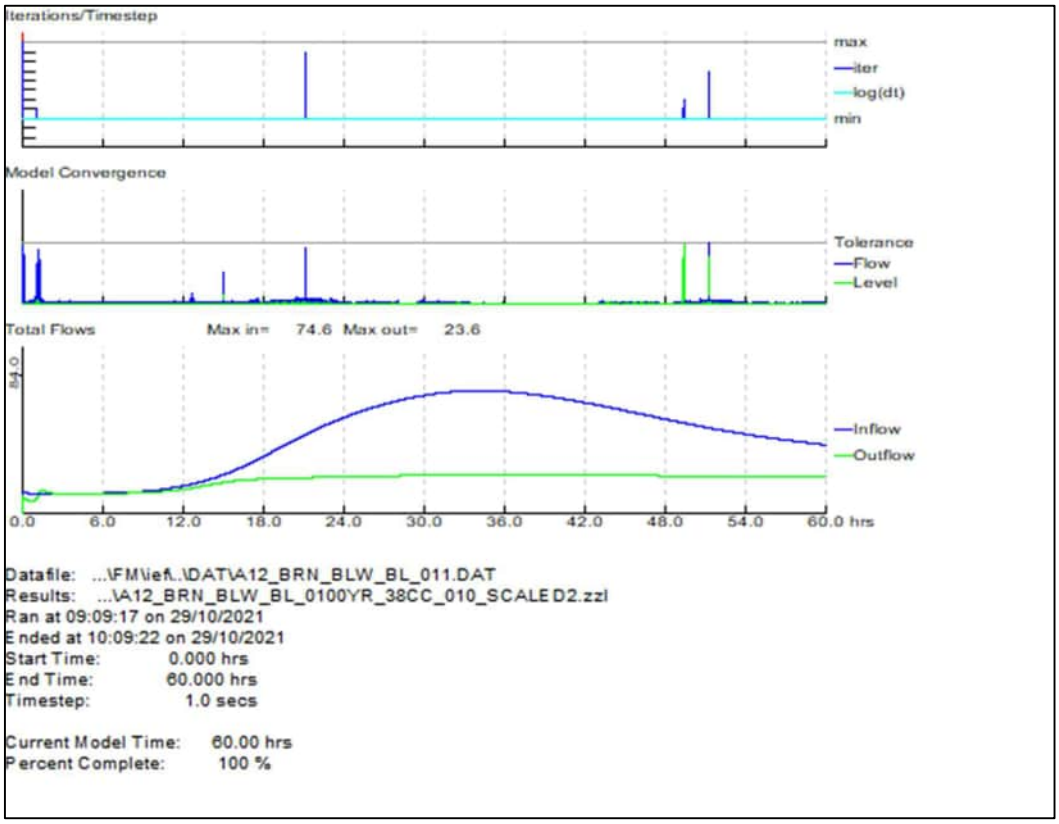
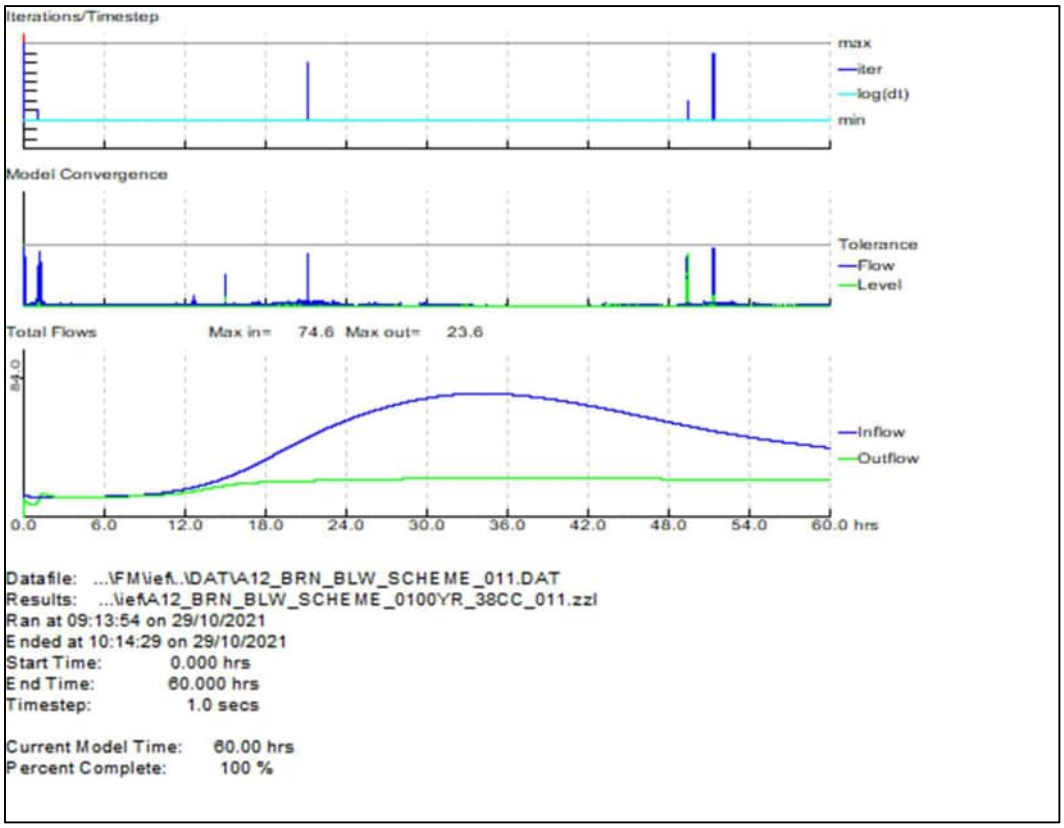


Plate 6.2 1D model convergence – 1% (1 in 100) AEP +38%CC event – with-scheme scenario



6.2.2 The cumulative mass error diagnostics output from the TUFLOW 2D model have also been checked for each model simulation. Plate 6.3 and Plate 6.4 show the cumulative mass error and the change in volume for the baseline and with-scheme 1% (1 in 100) AEP +38%CC event simulation. This plot is typical for all the events and scenarios simulated. The tolerance range recommended by the software manual is +/- 1% mass balance error. Some variability in change in volume is evident through the simulation but this is consistent with the effects of several surcharged bridge structures, deep flooding and sections of 1D channel with high sinuosity.

6.2.3 The cumulative mass error tolerance is exceeded somewhat at the onset of the floodplain wetting but stabilises to less than 1% error very early during the simulation before there is any significant volume of water in the floodplain.

Plate 6.3 2D model performance - 1% (1 in 100) AEP +38%CC event – baseline scenario

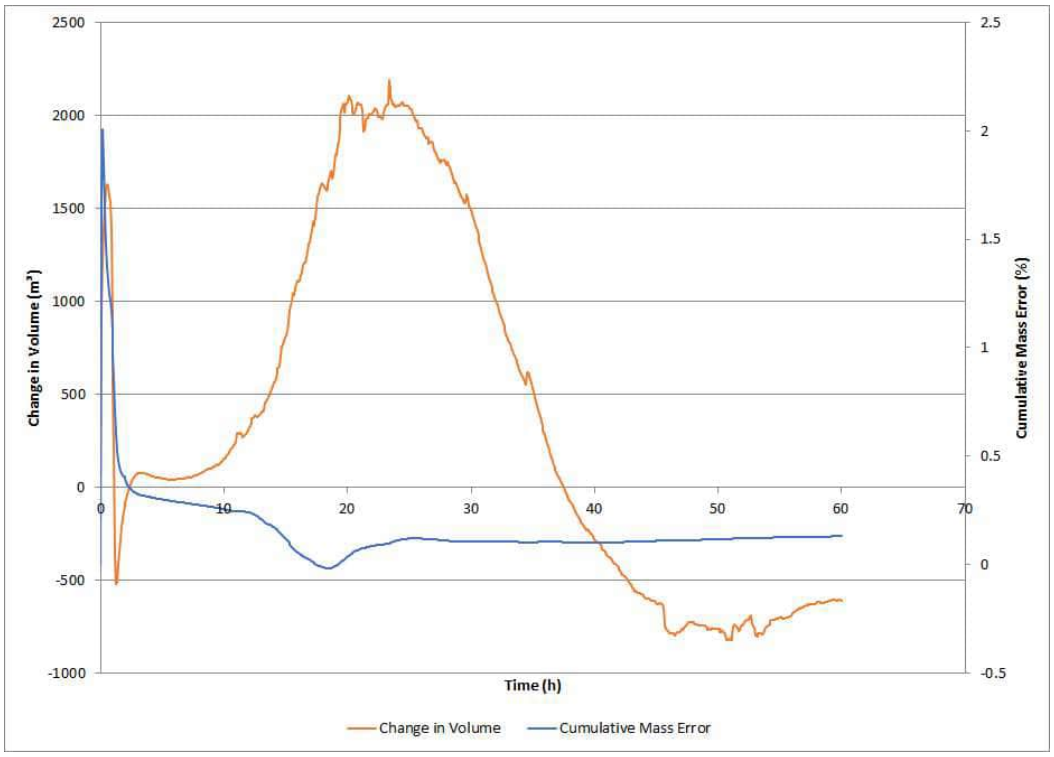
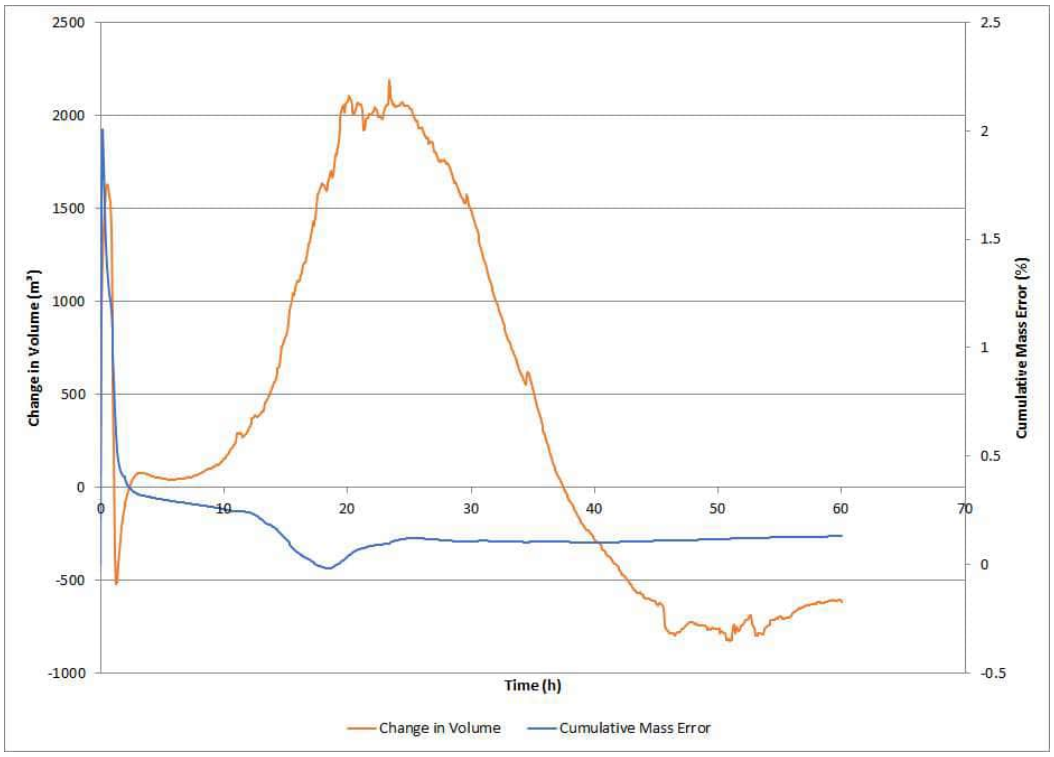


Plate 6.4 2D model performance - 1% (1 in 100) AEP +38%CC event – with-scheme scenario



6.3 Calibration and verification

Calibration

6.3.1 There are two gauges within the modelled reaches of the River Blackwater and River Brain; the River Blackwater at Appleford Bridge (37010) and the River Brain at Guithavon Valley (37009). Both gauges have been classified by the Peak Flows dataset as suitable for QMED estimation and pooling group analysis.

6.3.2 The incoming models of Lower Blackwater River and River Brain (CH2M, 2017) were calibrated using historical large magnitude flood events by the Environment Agency in 2010 (reference not available) and CH2M in 2017. These hydraulic models have been adopted for the present study but were truncated to cover the current study extent only. The design hydrology (peak flows) has been revised for the new model based on new methodology and recent hydrometric data.

6.3.3 As the hydraulic model was calibrated in 2010 and 2017 for past large magnitude flood events and given the post 2017 flood events were of a lower flood magnitude (ranked 28 or greater) with return periods equal to or less than QMED (2-year return period) only, it was considered that model re-calibration was not required for the current study.

Verification using historic data

6.3.4 Environment Agency historic flood maps are available, which show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. Plate 6.5 is a comparison of this data with the 1% (1 in 100) AEP modelled event.

6.3.5 The assessment of this data indicates that that the two maps are generally very similar. The historic flood outline is slightly wider than the modelled 1% (1 in 100) AEP flood extent event on River Blackwater, while on the River Brain the historical flood extent covers only small section of modelled river and is again somewhat wider than the modelled 1% (1 in 100) AEP extent. The assessment of the historic data does not provide any basis for changing the modelling.

Verification using EA flood maps

6.3.6 Plate 6.6 shows the modelled 1% (1 in 100) AEP flood extent and the Environment Agency published Flood Zone 3.

6.3.7 The comparison in Plate 6.6 indicates that the two extents are generally very similar. However, in some areas the Environment Agency flood map predicts a somewhat larger flood extent than the present model results. This is expected as the present study includes significant refinement of the modelling process, particularly with respect to the determination of design flows.

Verification conclusion

6.3.8 In conclusion, there is limited available data for verification of the River Brain and Lower Blackwater. As stated above, the model has been previously verified in 2010 and 2017. The Environment Agency flood map shows a generally similar flood extent to the present model (Plate 6.6). Therefore, the model is considered to be appropriate for use in the present study.

Plate 6.5 Modelled 1% (1 in 100) AEP event flood extent vs. Environment Agency historic flooding

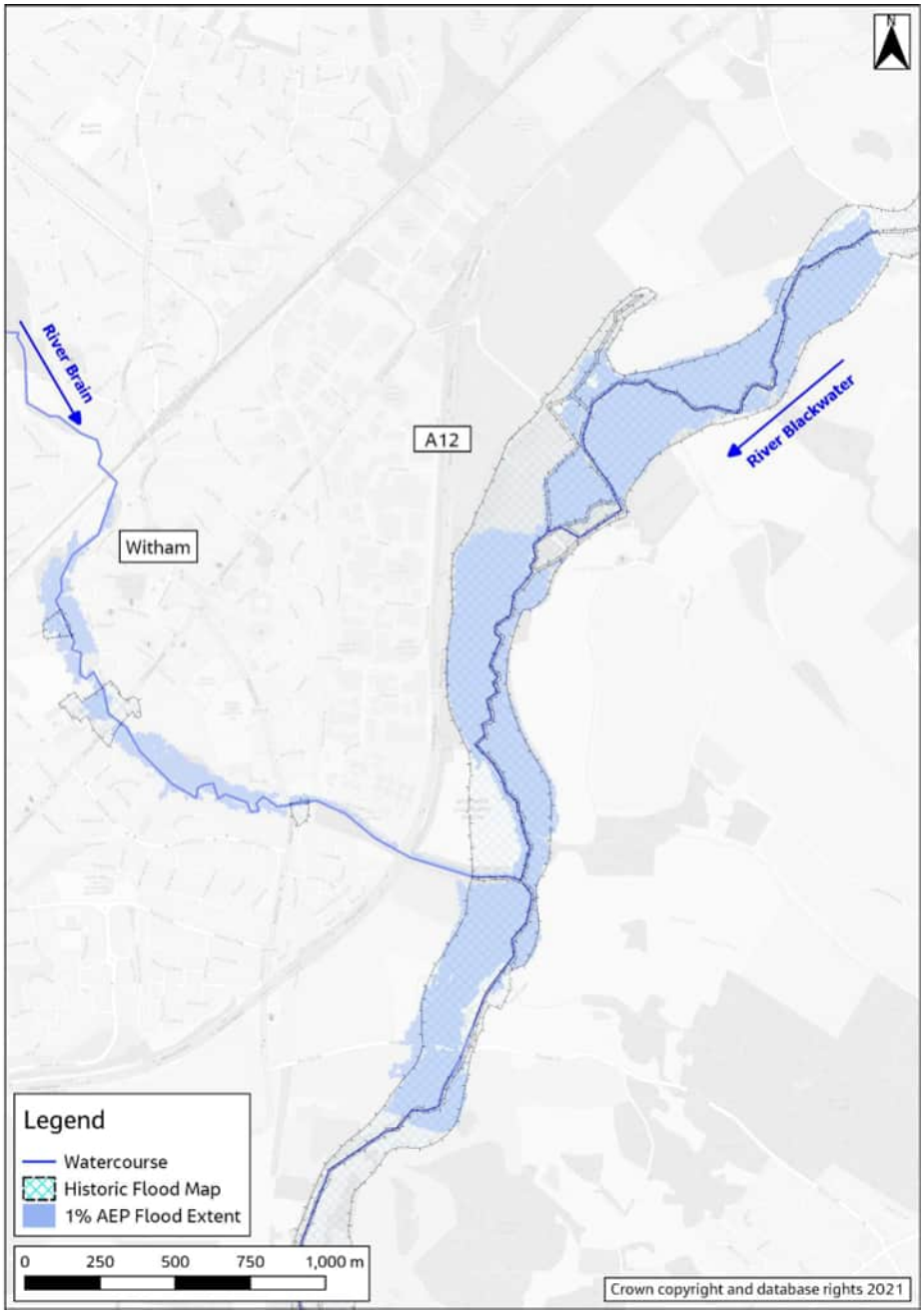
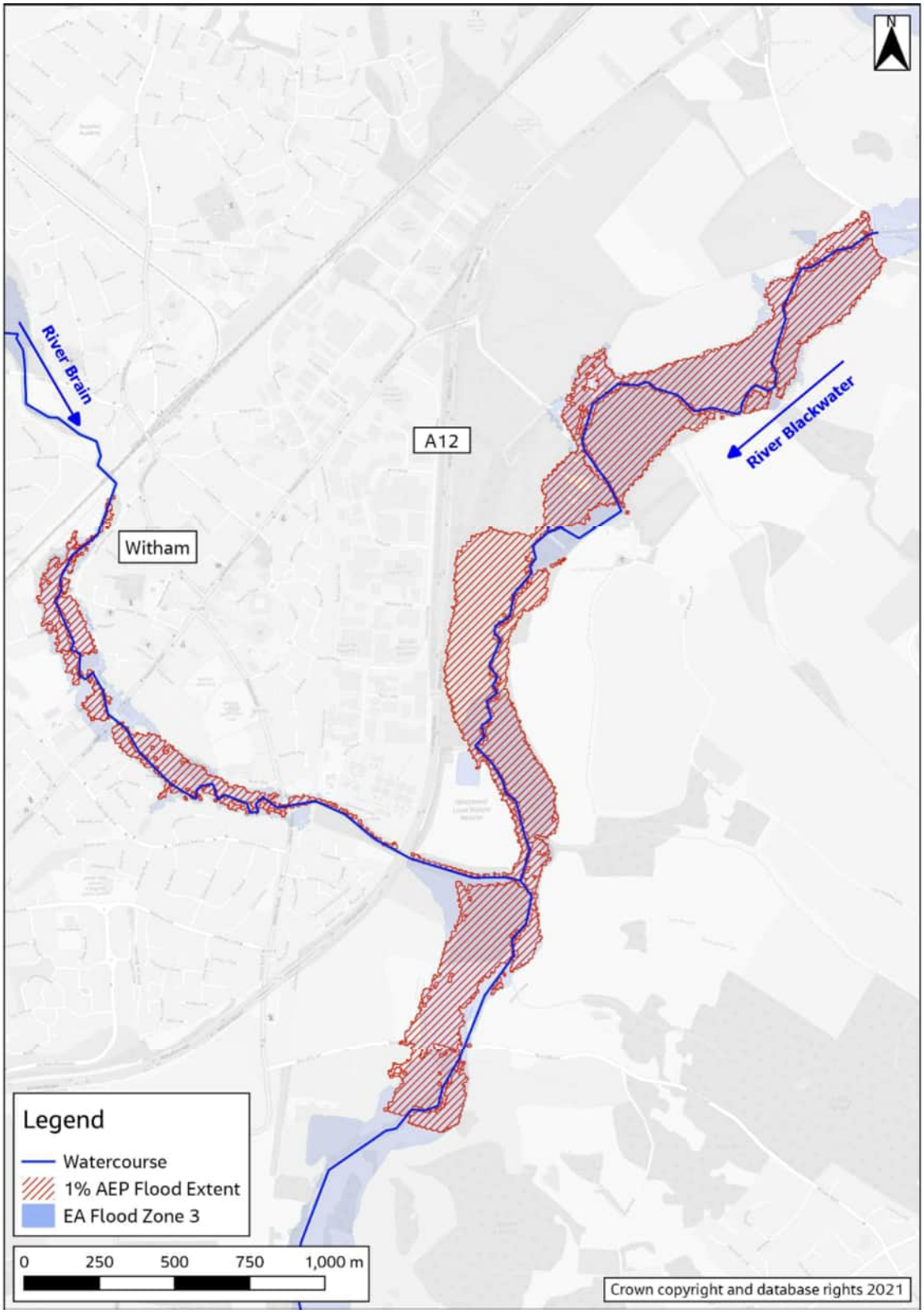


Plate 6.6 Modelled 1% (1 in 100) AEP event flood extent vs. Environment Agency Flood Zone 3



6.4 Sensitivity analysis

6.4.1 All sensitivity tests have been performed for the 1% (1 in 100) AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Table 6.1 shows the impact of changing the model roughness on predicted peak water levels.

Table 6.1 Roughness sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	0.24	-0.02	0.09
-20% Roughness	0.06	-0.13	0.05

6.4.3 The results show that the in-channel water levels are moderately sensitive to changes in roughness. Decreased and increased roughness caused only small changes across whole flood extent outline.

Hydrological inflow sensitivity

6.4.4 The flows into the model were adjusted by +20% and -20%. Table 6.2 shows the impact of changing the model inflows.

Table 6.2 Flow sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.26	0.04	0.12
-20% Flow	-0.01	-0.18	-0.08

6.4.5 The results show that the in-channel water levels are moderately sensitive to changes in flow. In the 2D floodplain domain the same general patterns of flow were maintained between the baseline and the sensitivity tests. Increase and decrease of flow by 20% caused only a slight response to the modelled flood extent.

Downstream boundary condition sensitivity

6.4.6 The water levels in the QH downstream boundary were adjusted in 1D only, for two sensitivity tests. For the first test, water levels in the downstream boundary were lowered by 0.5m, for the second raised by 0.5m. Table 6.3 shows the response at the downstream end of the model (Flood Modeller node BWT29900). The location at which there is no change in water level as a result of changing the downstream boundary has been identified. Distances from this location, in relation to the

downstream end of the model (tailwater distance) and in relation to the proposed-scheme are also shown.

Table 6.3 Downstream boundary sensitivity results

Sensitivity	Water Level Difference at BWT29900 (m)	Tailwater distance (m)	Distance to proposed-scheme (m)
Water level lowered by 0.5 m	-0.07	360	1400
Water level raised by 0.5 m	0.06	360	1400

6.4.7 The results show that the impact of altering the downstream boundary does not reach the scheme location. Adjustment of the water level at the downstream boundary resulted in no change in the wider modelled flood extent. The results indicate that the model downstream boundary is appropriately removed from the area of interest at the proposed-scheme A12 crossing.

7 Baseline model results

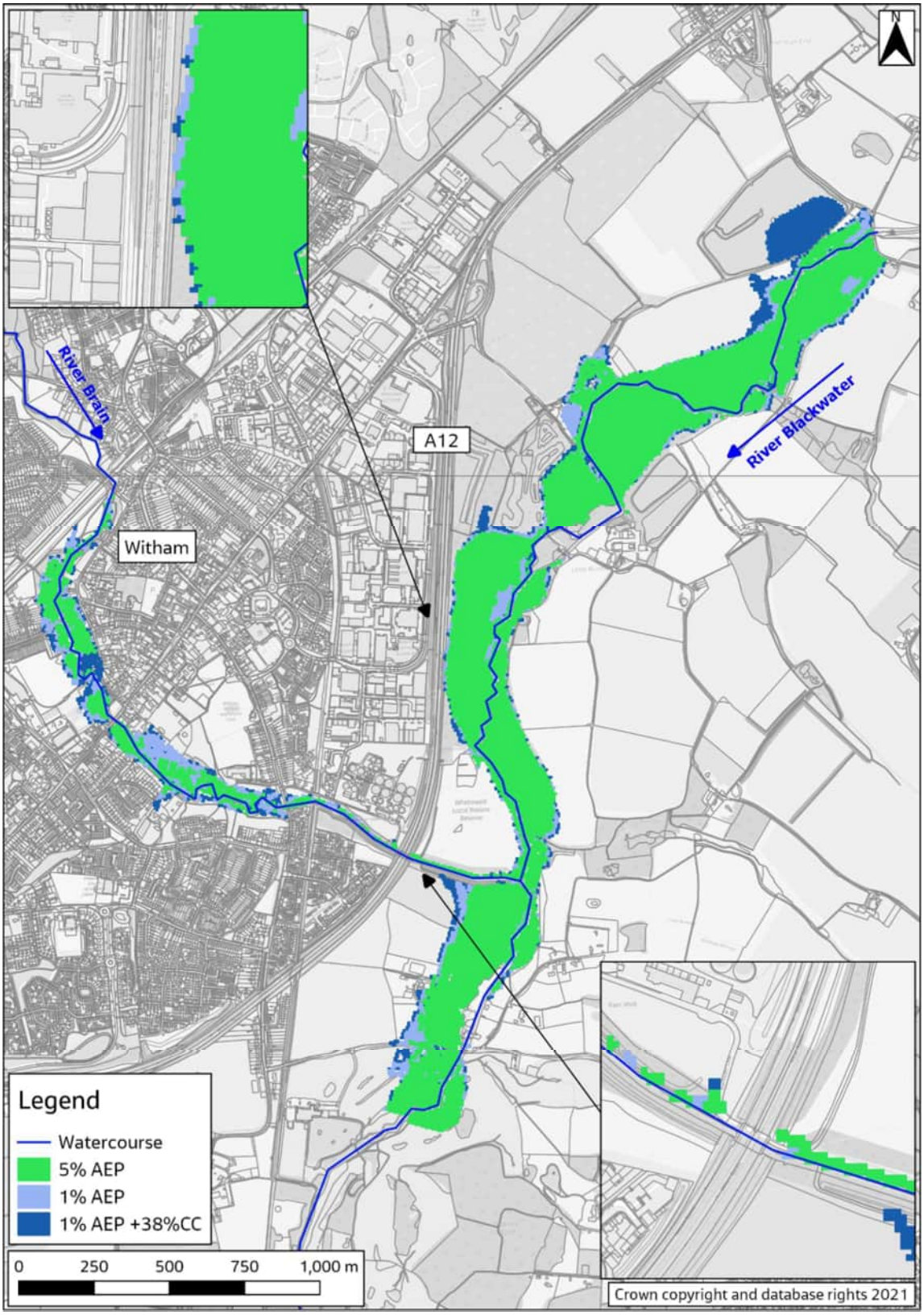
7.1.1 Plate 7.1 shows the maximum flood extent for all the modelled events for the baseline scenario.

7.1.2 For the 1% (1 in 100) AEP +38%CC event there are 32 properties situated within the modelled flood extent of the River Blackwater. The River Brain flows through an urban area (Witham) and it is noted that 35 properties that occupy low ground adjacent to the watercourse are within the modelled flood extent. All flooded properties are located a considerable distance from the scheme; on the River Blackwater the nearest are approximately 1 km upstream and 0.5 km downstream from the confluence with the River Brain. On the River Brain, the nearest properties are around 0.4 km upstream from the proposed-scheme.

7.1.3 The model results show that at the River Brain crossing, none of the simulated storm events cause flooding to the A12 highway. The freeboard to the bridge soffit is approximately 2.9 m for the 1% (1 in 100) AEP +38%CC event.

7.1.4 On the River Blackwater, the flood extent reaches the toe of the A12 embankment at one location (see Plate 7.1), therefore the carriageway is not at risk for the design events. The flooding is limited to very shallow (less than 20mm) water at the periphery of the flood extent.

Plate 7.1 Modelled events flood extent for the baseline scenario



8 With-scheme modelling

8.1 Model modifications

8.1.1 The proposed-scheme involves widening the existing A12 highway bridge over the River Brain to accommodate the widened carriageway. In the model, this was represented in the 2D domain using elevation polygons to change the terrain upstream and downstream of the bridge. The effective widening of the highway embankment is 6m on the upstream side and 4m on the downstream side. An additional polygon was added 500m upstream on the River Blackwater on the east side of the A12 (see Plate 8.1), where the scheme footprint touches the modelled flood extent, to represent the widened A12 embankment which in the order of 5m. The levels and shapes for these embankment modifications were obtained from the scheme design drawings. Cross section spacing in the 1D Flood Modeller schematisation was adjusted to account for the relocation of the River Brain A12 bridge upstream face.

Plate 8.1 New embankment representation in the model



8.2 Design model results

8.2.1 The change in maximum water levels local to the scheme at the River Brain Crossing is less than 1mm in the 1D domain, indicating that there is no impact on water levels or flows due to the scheme.

8.2.2 Flood extents for 1% (1 in 100) AEP +38%CC is shown in Plate 8.2 for the baseline and with-scheme scenarios. It shows that the flood extent for the with-scheme model is the same as the baseline model. Plate 8.3 shows the difference in peak water level between the with-scheme and the baseline scenarios for the 1% (1 in 100) AEP +38%CC event. There is a very small area (a single 2D model cell) immediately adjacent to the upstream embankment where maximum water levels are increased by up to approximately 10mm, otherwise the change in modelled depth in the floodplain at the River Brain crossing is less than 10mm. The model shows zero change in peak water levels between the baseline and with-scheme model results for locations further than 10m from the bridge. The existing flood extent displaced by the new carriageway embankment is shown as reduction in existing flood depth in Plate 8.3. The A12 carriageway does not flood for all modelled events.

8.2.3 There is zero change in modelled flood depth on the River Blackwater in the location of the upstream embankment widening.

8.2.4 The negligible flood risk impact described above is replicated for the other modelled events. Water level difference maps and flood extents for all events are included in Appendix D.

Plate 8.2 Flood extent comparison between baseline and with-scheme for the 1% (1 in 100) AEP +38%CC event

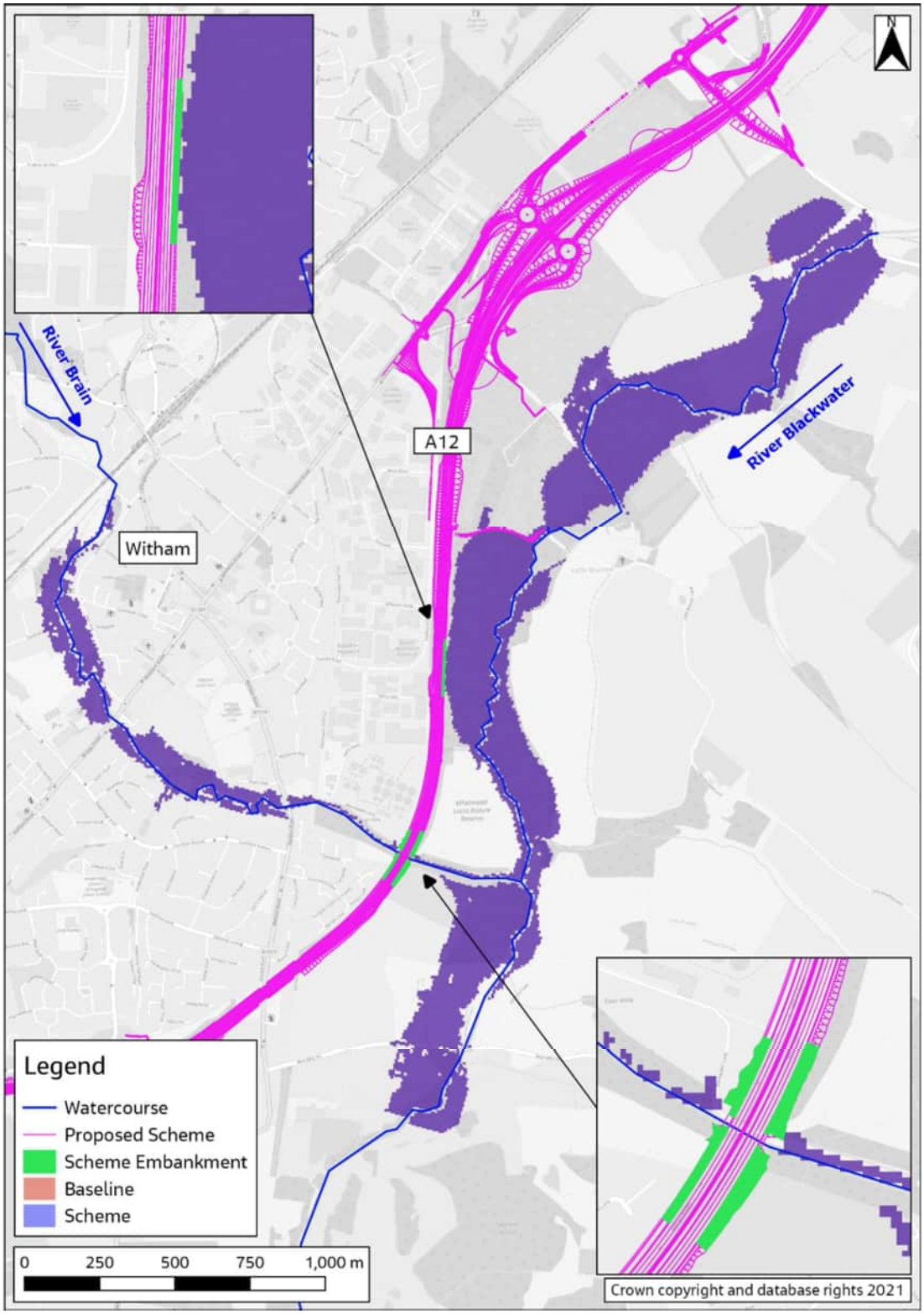
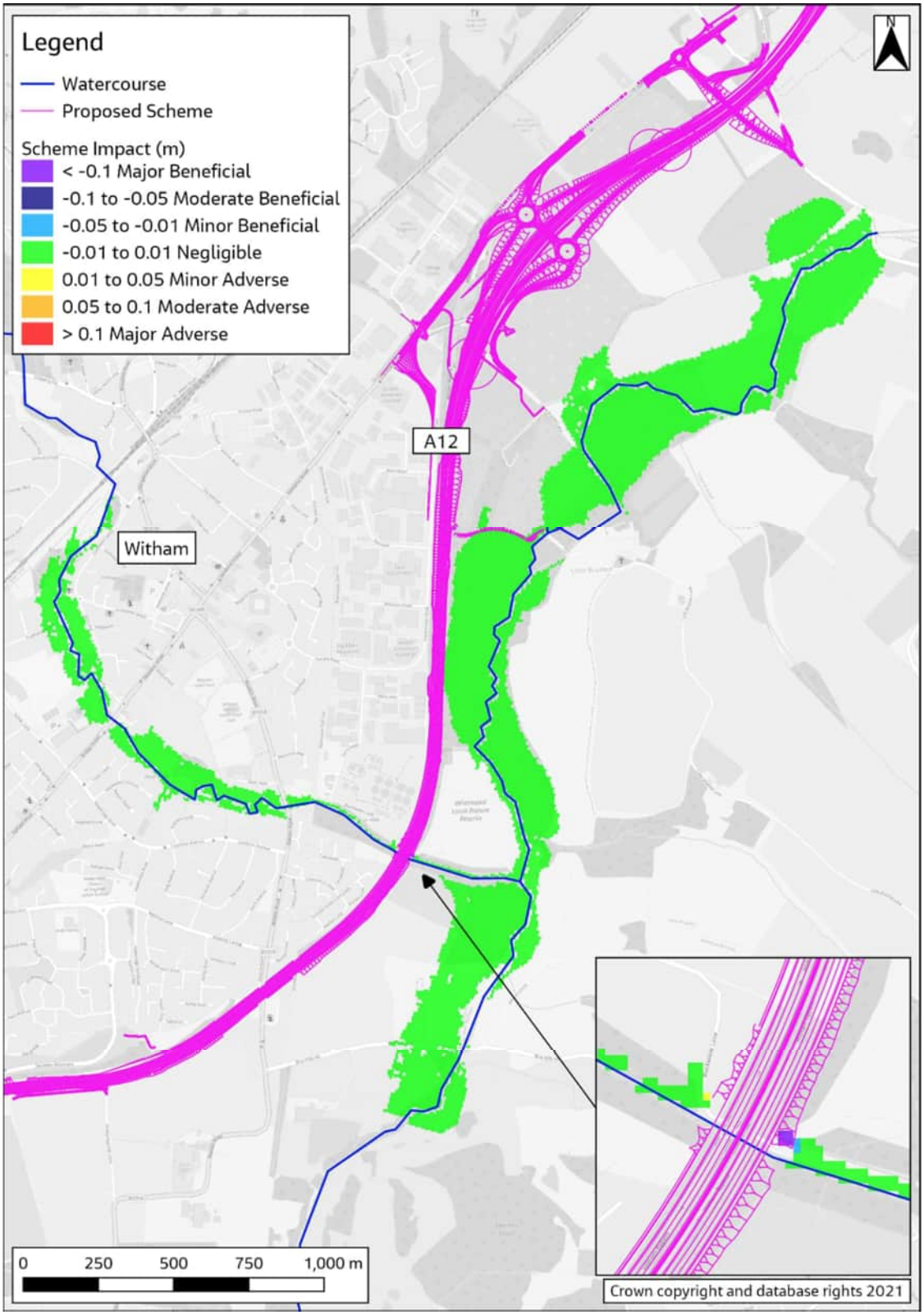


Plate 8.3 Water level difference (with-scheme – baseline) for the 1% (1 in 100) AEP +38%CC event



9 Temporary works modelling

9.1 Temporary works changes

9.1.1 Modelling of the temporary works required to facilitate the construction of the proposed-scheme was undertaken. A haul road of 10m width east of the A12 carriageway (see Plate 9.1) was incorporated into the model by modifying the elevation of the ground in the 2D domain. No changes were made to the 1D domain. Plate 9.2 shows the temporary change in ground level over the length of the 2 km long haul road.

Plate 9.1 Haul road location

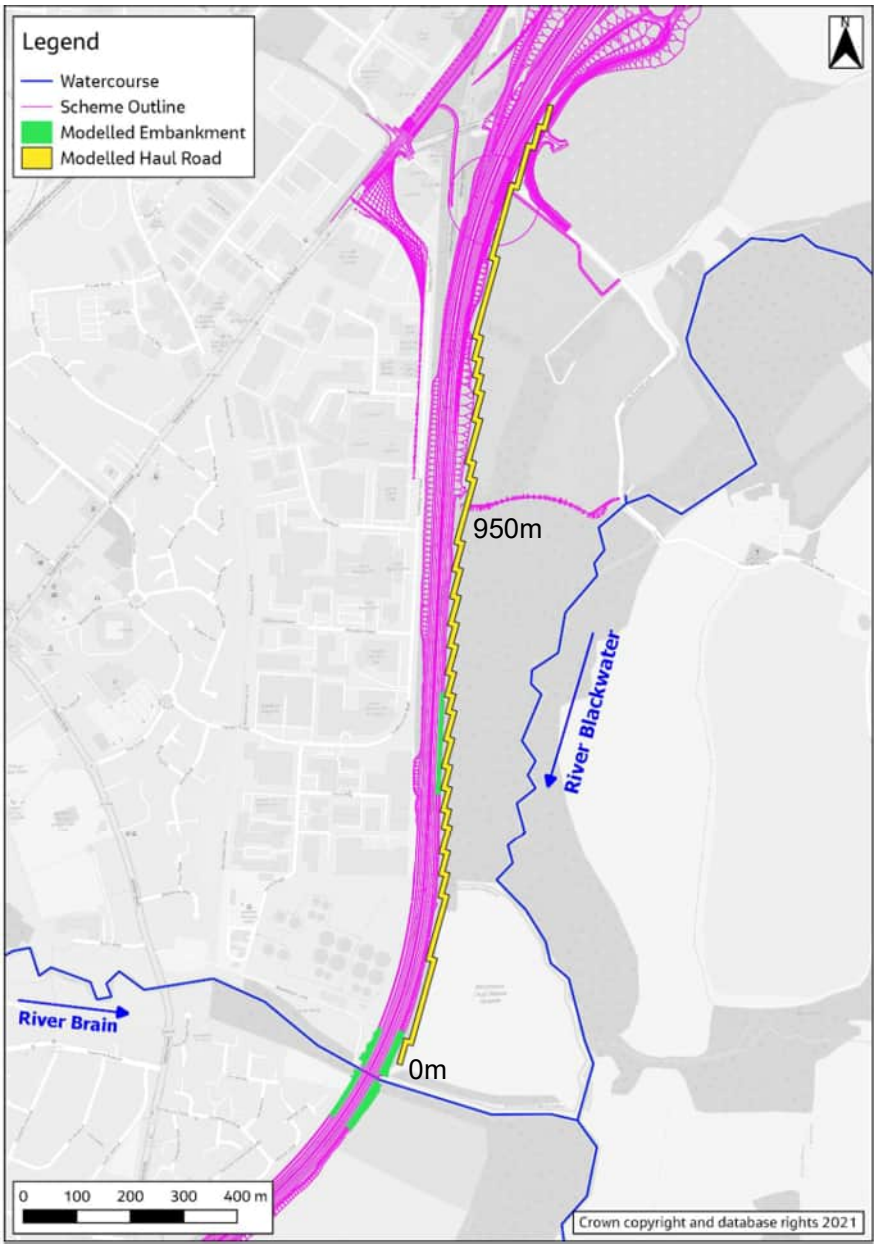
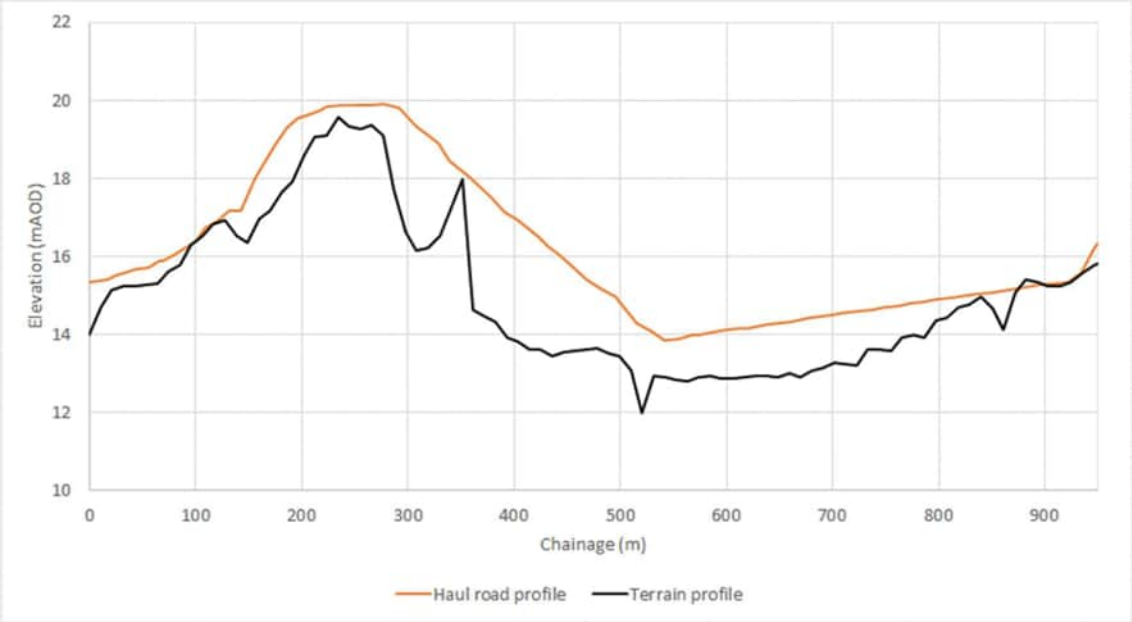


Plate 9.2 Comparison of existing and haul road ground level profiles



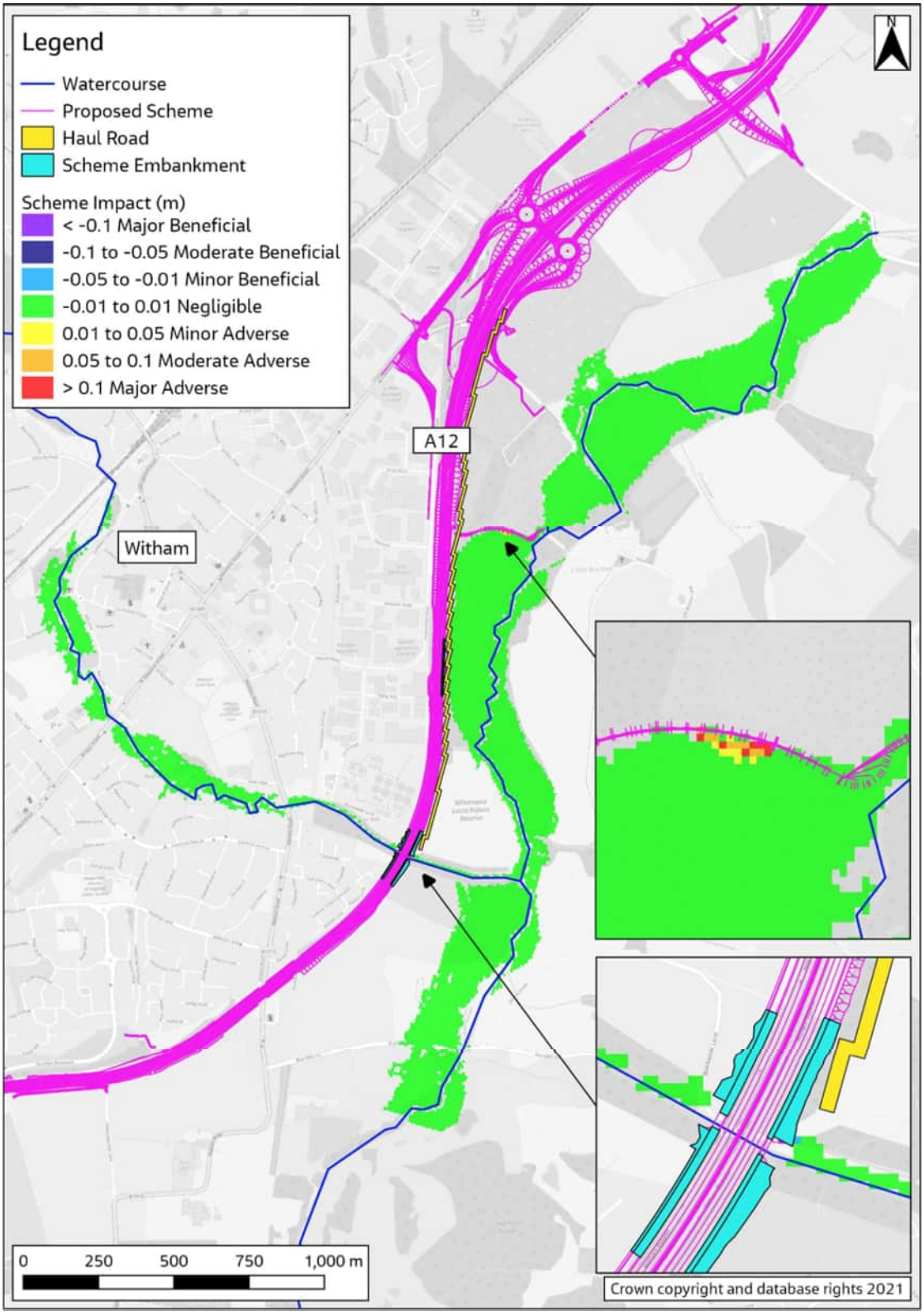
(see chainage labels in Plate 9.1 for long section alignment)

9.2 Temporary works results

9.2.1 Plate 9.3 shows the peak water level difference between the baseline, temporary works and the with-scheme scenarios for the 1% (1 in 100) AEP event. In general, the increase in the modelled water level due to the haul road is 10mm or less compared to baseline scenario. One small area is noted within the Blackwater right (west) floodplain just downstream of Little Braxted Lane, where the general increase of less than 10mm causes a small increase in the flood extent of approximately three 2D cells width (30m), into lower ground adjacent to the baseline flood extent (see insert on Plate 9.3). Within this area there is a water level increase of up to 370mm. The area is woodland / wetland, and the effect is only evident for the 1% (1 in 100) AEP temporary works scenario.

9.2.2 The water level difference for the 5% (1 in 20) AEP event can be found in Appendix D which shows there to be no significant effect on modelled flood risk due to the with-scheme plus temporary works scenario.

Plate 9.3 Water level difference (with-scheme and temporary works – baseline) for the 1% (1 in 100) AEP event



10 Model assumptions and limitations

10.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological and topographic data included in the model. Therefore, the most appropriate available information has been used to construct the model.

10.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are considered to be generally conservative for modelled water levels at the proposed-scheme and are therefore appropriate for the flood risk assessment.

10.2 Hydrology

10.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- ReFH1 hydrograph shapes are suitable to use in this investigation
- The ReFH1 units from the incoming model are suitable for the residual (lateral inflow) catchments
- The River Blackwater at Appleford Bridge and River Brain at Guithavon Valley gauges are suitable to estimate design peak flows up to the 1% (1 in 100) AEP event using Enhanced Single Site statistical analysis
- ReFH2.3 is likely to be overestimating lower severity event peak flows, that is, for peak flow estimates up to and including the 1% (1 in 100) AEP event, for the target catchments (based on gauged data for the River Blackwater at Appleford Bridge and River Brain at Guithavon Valley gauges) and therefore ReFH2.3 peak flow estimates were not adopted for lower return periods
- ReFH2.3 ratios are more appropriate for higher return period flood events and therefore this method was adopted for the 0.5% (1 in 200) AEP and rarer flood events

10.3 Hydraulic modelling

10.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for the River Brain and Lower Blackwater are as follows:

- Channel roughness has been retained from the incoming model and confirmed as appropriate using the best available information (survey data and aerial photographs), with reference to the available guidance (Chow 1959).

- Hydraulic coefficients for structures have been retained from the incoming model and confirmed as appropriate using available guidance within the Flood Modeller software.
- The hydraulic model was calibrated and verified by CH2M in 2017. No recalibration was deemed necessary for the present study.

11 Conclusion

11.1.1 This report has detailed the hydraulic modelling carried out to assess the baseline and with-scheme flood risk for River Brain and Lower Blackwater with reference to the location of the proposed highway scheme.

11.1.2 The results have shown that there are 67 properties at risk of flooding within the model domain for the baseline 1% (1 in 100) AEP +38%CC event. However, these are all notably remote from the scheme.

11.1.3 The A12 carriageway does not flood for all events tested.

11.1.4 The proposed-scheme has no impact on modelled in-channel water levels for all events tested. There is no change to peak water levels in the wider floodplain, with the only exception a single (10m) 2D model cell showing 10mm increase in water level compared to the baseline immediately upstream of the River Brain road bridge affecting woodland. It was shown that the proposed-scheme has no impact to the flood risk outside of the area immediately adjacent the scheme.

11.1.5 The temporary works required to construct the proposed-scheme do not cause any additional flooding except for a small area of approximately 300m^2 within the 1% (1 in 100) AEP simulation, where no properties or roads are affected.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
AMAX	Annual Maximum
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
EA	Environment Agency
ESS	Enhanced Single Site
FEH	Flood Estimation Handbook
NRFA	National River Flow Archive
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SPRHOST	Standard percentage runoff derived using the HOST soil classification
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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Appendix A **River Brain Hydrology Appendix**

Please see document HE551497-JAC-EWE-SCHW-RP-LE-0074.docx

Appendix B **Structure Schedule**

Table B.1 Model Structure Schedule

Node Label(s)	XY Coordinates	Modelling Approach
BR_BWT29590u	583030, 213089	Represented as USBPR1978. Spill unit to allow overtopping not required. Data based on detailed survey carried out by Jacobs in 2020. FM default coefficients applied for all parameters.
BR_LBRAX291u	583313, 214804	Not modelled – 0.5 m width will not affect flow pattern.
BR_BRN23828u	581687, 214512	Represented as Arch Bridge. Spill unit to allow overtopping not required. Data based on detailed survey. FM default coefficients applied for all parameters.
BR_BRN24280u	581874, 214187	Represented as Symmetrical conduit. Spill unit to allow overtopping not required. Data based on detailed survey. FM default coefficients applied for all parameters.
BR_BRN25120u	582448, 213896	Represented as Arch Bridge. Spill unit to allow overtopping not required. Data based on detailed survey. FM default coefficients applied for all parameters.
BR_BRN25525u	582833, 213732	Represented as Estry circular culvert. Spill unit to allow overtopping not required. Data based on HADDMS records. Default coefficients applied for all parameters.
BWT27600OrU	583374, 214741	Represented as Orifice. Data based on detailed survey. FM default coefficients applied for all parameters.
BWT29540OrU	583061, 213142	Represented as Orifices. Data based on detailed survey.

		FM default coefficients applied for all parameters.
BWT29540Or2U	583061, 213142	Represented as Orifices. Data based on detailed survey. FM default coefficients applied for all parameters.
LBRAX130OrU	583444, 214861	Represented as Orifices. Data based on detailed survey. FM default coefficients applied for all parameters.
SP_BRN23549u	581750, 214746	Crest of Guithavon Valley Gauge. Represented as Spill. Spill coefficient 1.7. FM default coefficients applied for all other parameters.

Appendix C **Maximum Water Level Tables and Long Sections**

Table C.1 Maximum Water Level at Key Locations for All Events Modelled

Model Node	Description	5% (1 in 20) AEP Maximum Water Level (mAOD)			1% (1 in 100) AEP Maximum Water Level (mAOD)			1% (1 in 100) +38%CC AEP Maximum Water Level (mAOD)		
		Baseline	With-Scheme	Difference (m)	Baseline	With-Scheme	Difference (m)	Baseline	With-Scheme	Difference (m)
BWT25556d	Upstream extent of the River Blackwater. 2.3km upstream of the scheme	15.98	15.98	0.00	16.10	16.10	0.00	16.21	16.21	0.00
BWT27900	River Blackwater meander adjacent to the scheme	13.34	13.34	0.00	13.48	13.48	0.00	13.65	13.65	0.00
BWT28400	River Blackwater meander adjacent to the scheme	13.02	13.02	0.00	13.23	13.23	0.00	13.45	13.45	0.00
BWT29900	Downstream end of the River Blackwater. 1.4km downstream of the scheme	11.07	11.07	0.00	11.17	11.17	0.00	11.28	11.28	0.00
BRN_US	Upstream extent of the River Brain. 1.1 km upstream of the scheme	17.32	17.32	0.00	17.53	17.53	0.00	17.73	17.73	0.00
BRN25525u	River Brain immediately upstream of the scheme	12.89	12.89	0.00	13.09	13.09	0.00	13.28	13.28	0.00

Plate C.1 Maximum Water Level in Channel for All Events for the Baseline Scenario

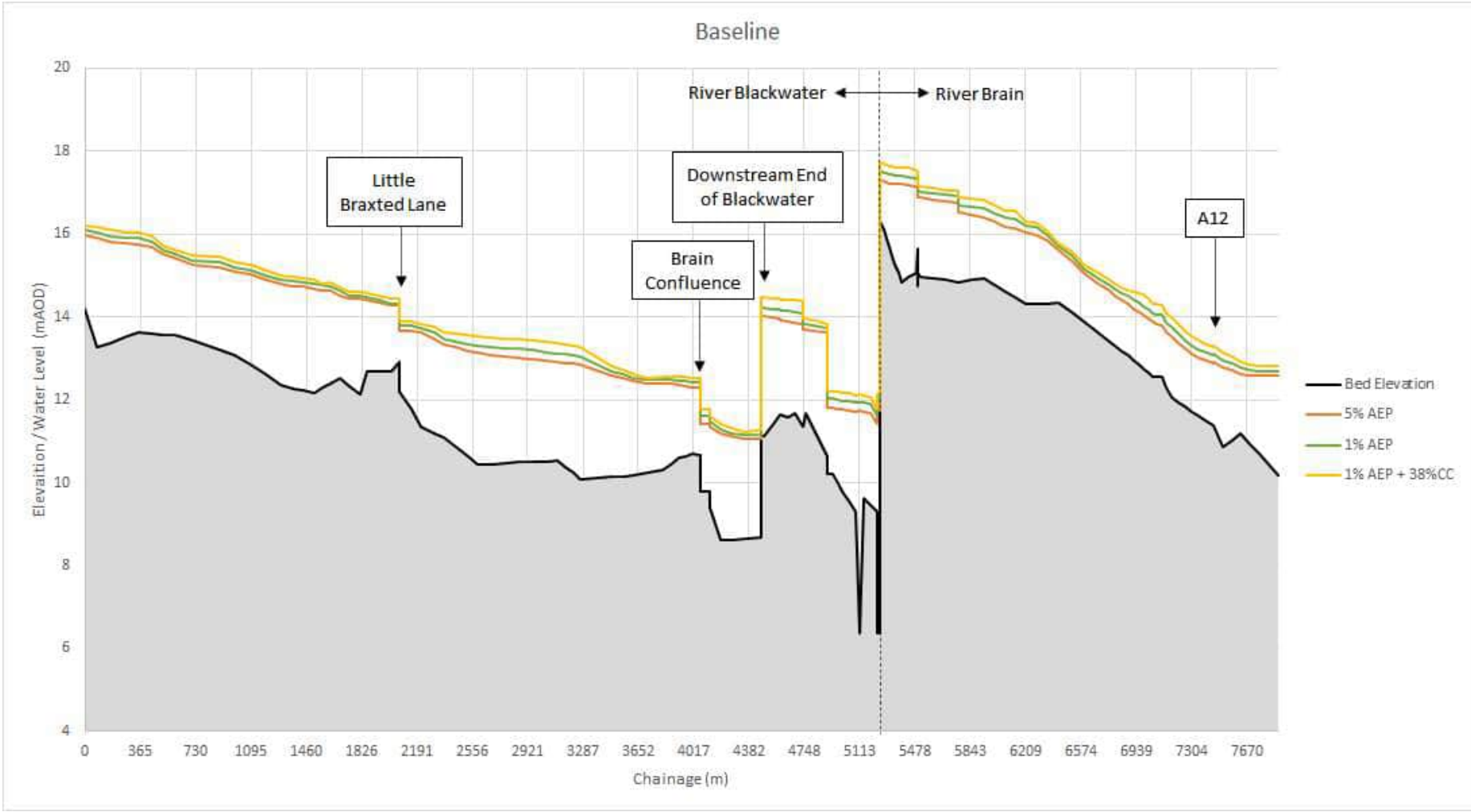
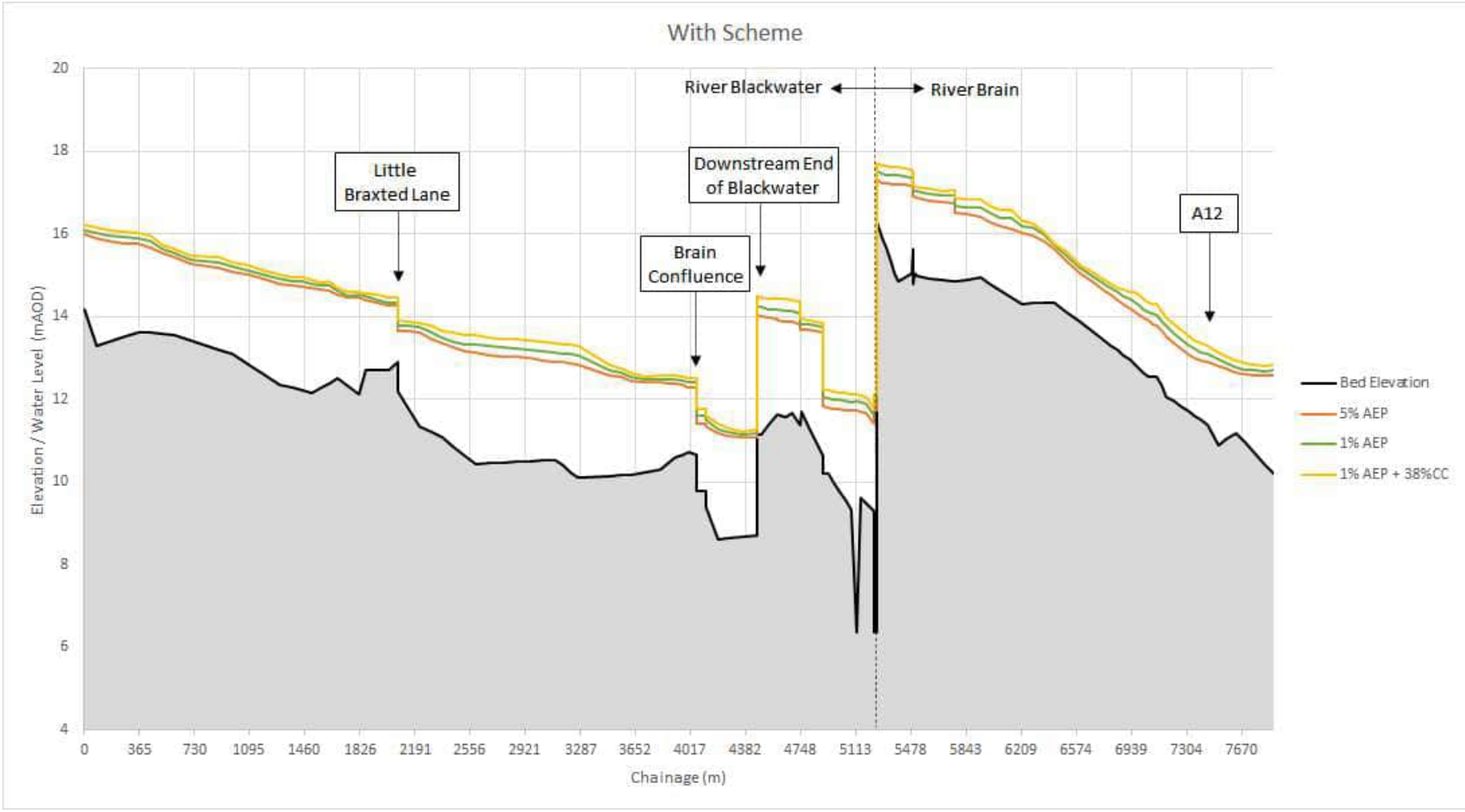


Plate C.2 Maximum Water Level in Channel for All Events for the With-scheme Scenario



Appendix D **Flood Maps**

Plate D.1 Flood Extent for all modelled events for Run 2 Simulation for Baseline Scenario

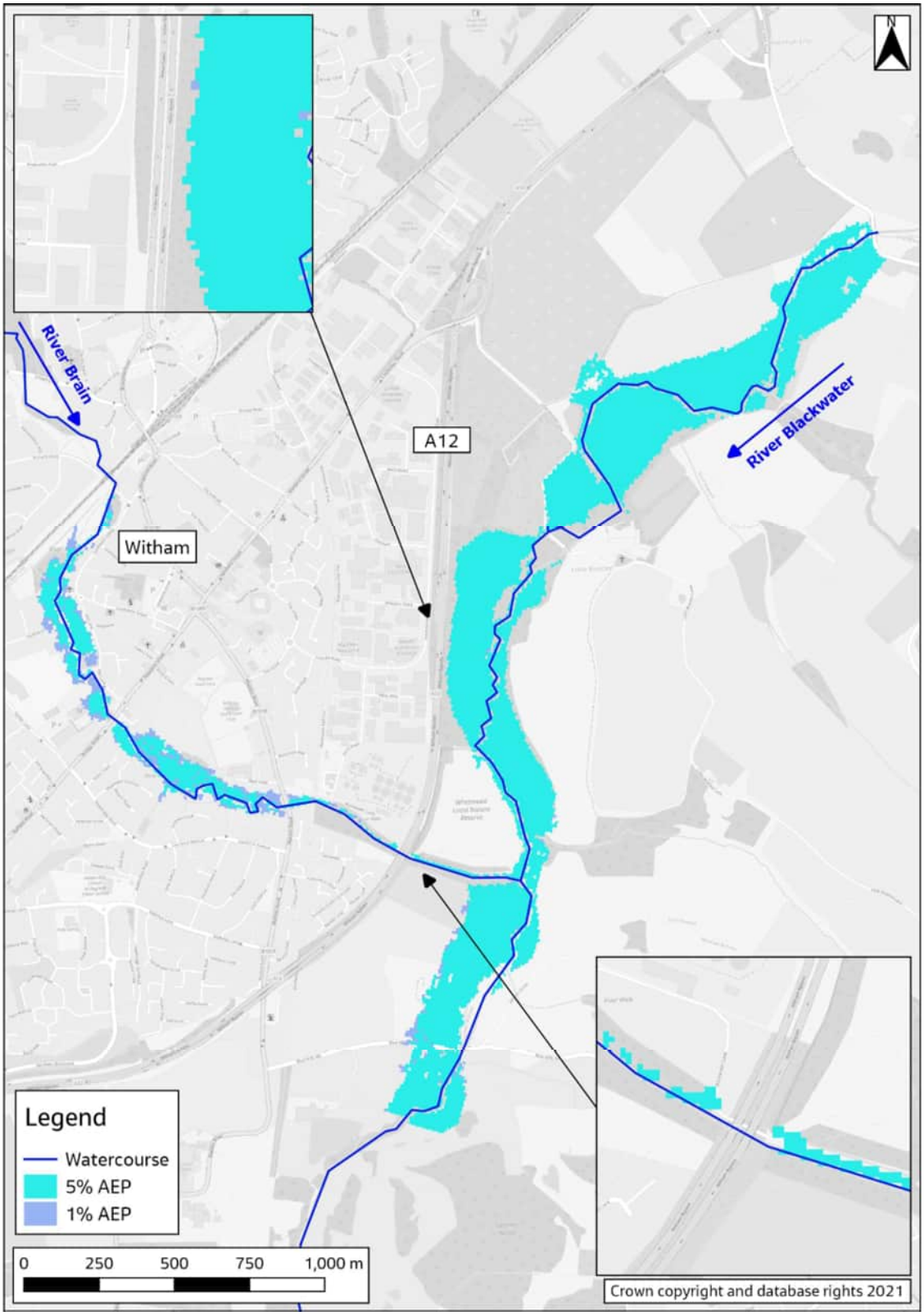


Plate D.2 Water Level Difference (With-Scheme minus Baseline) for the 5% (1 in 20) AEP event

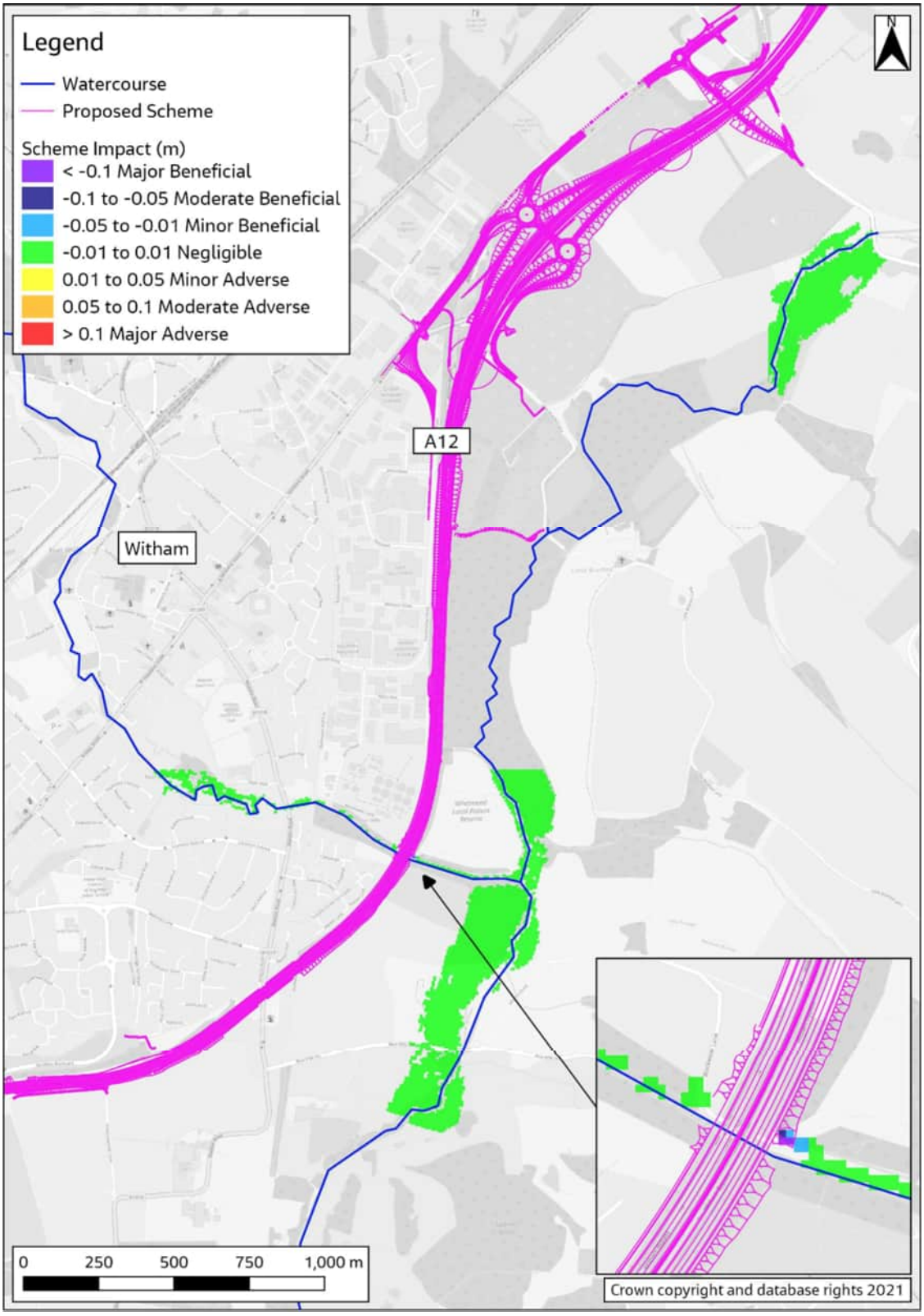


Plate D.3 Water Level Difference (With-Scheme minus Baseline) for the 1% (1 in 100) AEP event

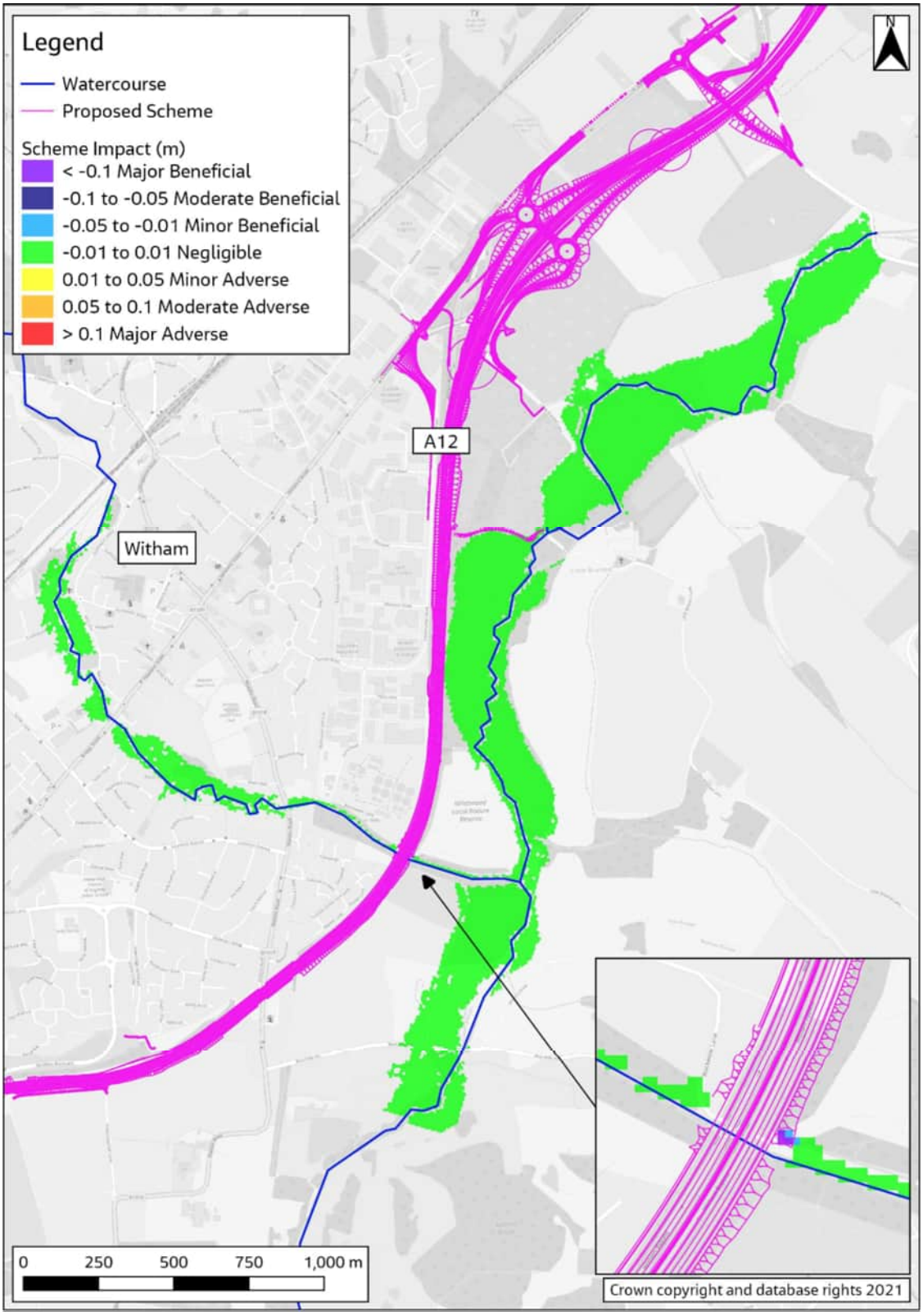


Plate D.4 Water Level Difference (With-Scheme minus Baseline) for the 1% (1 in 100) AEP +38%CC event

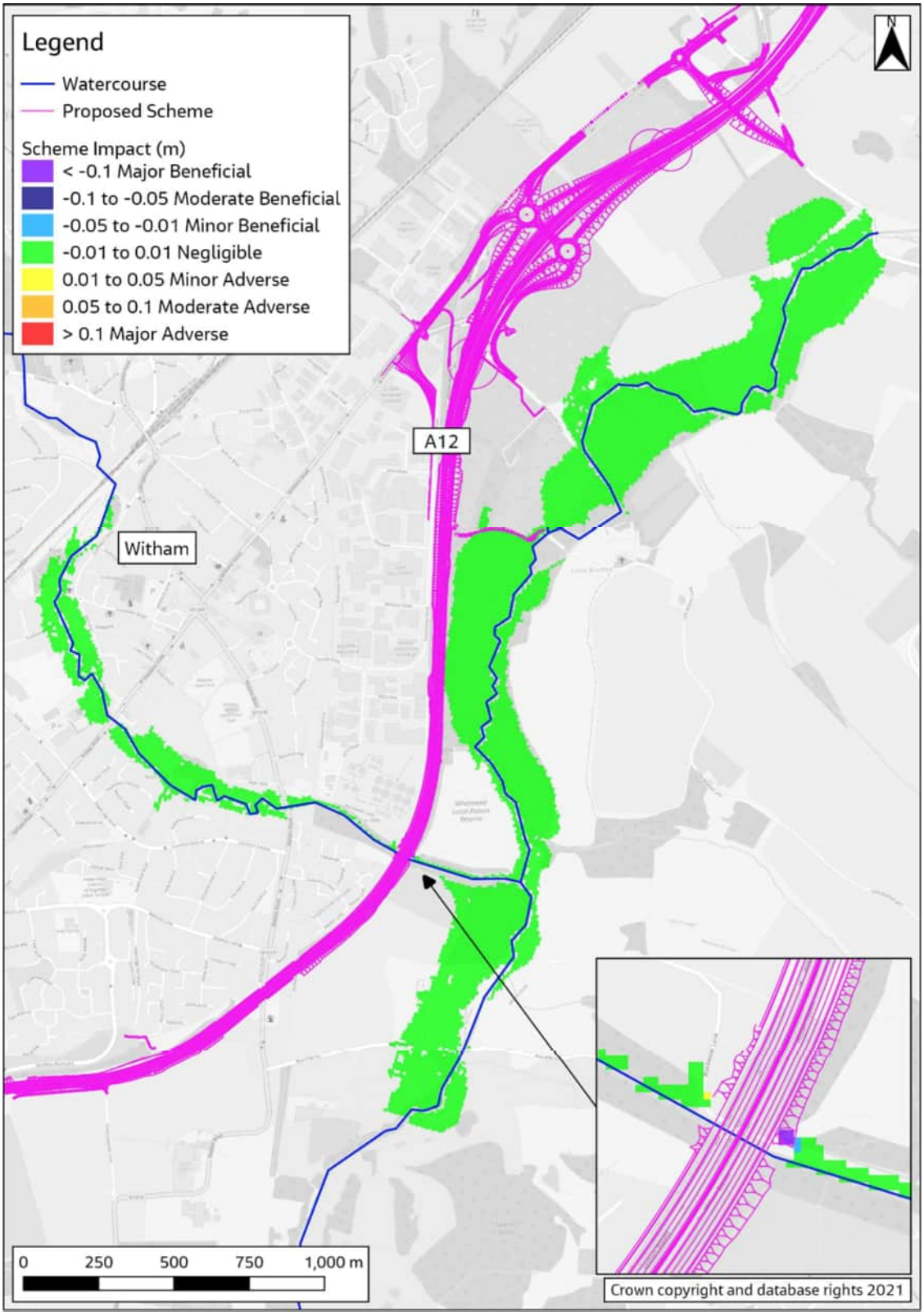


Plate D.5 Water Level Difference (With-Scheme and Temporary Works minus Baseline) for the 5% (1 in 20) AEP event

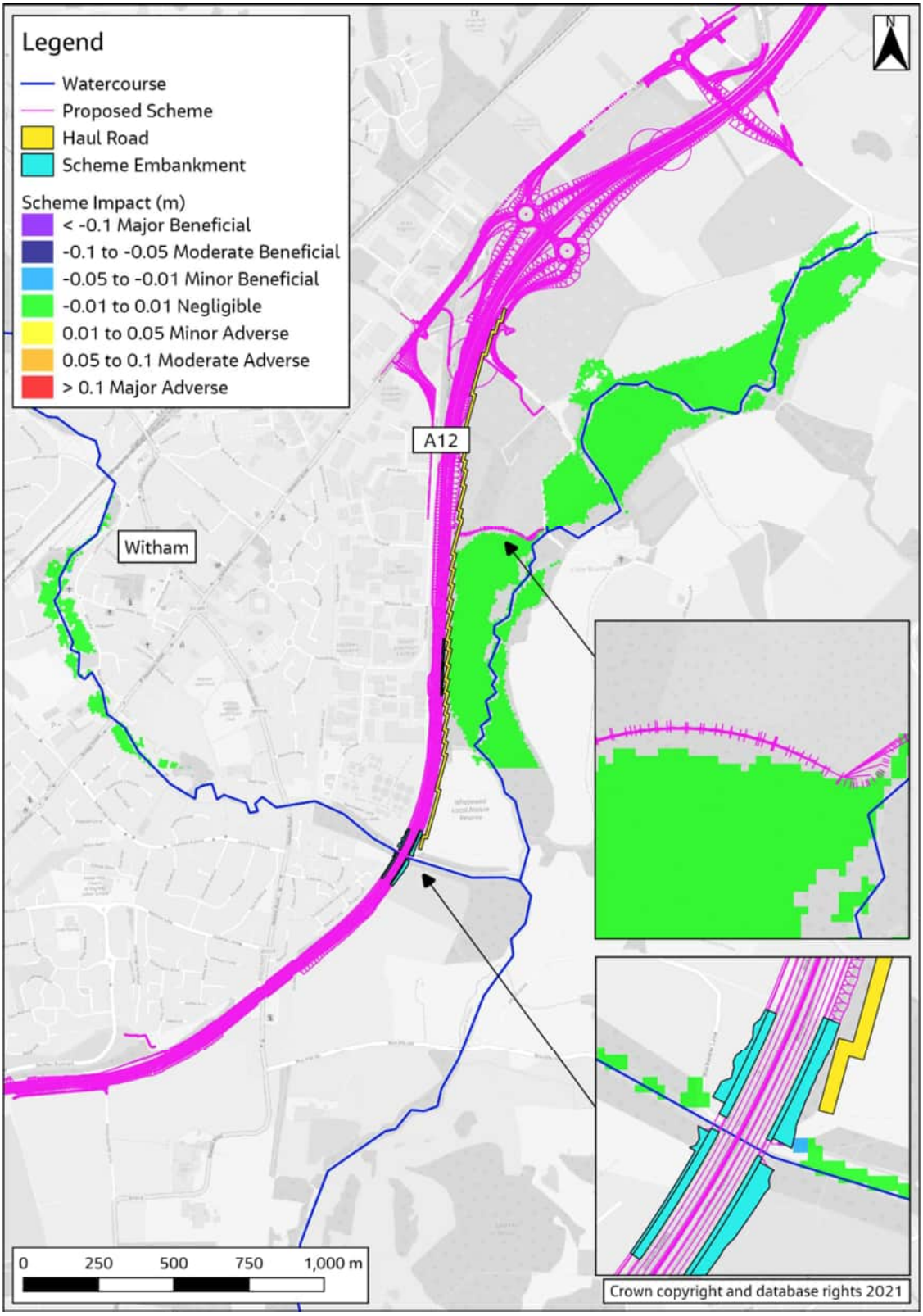
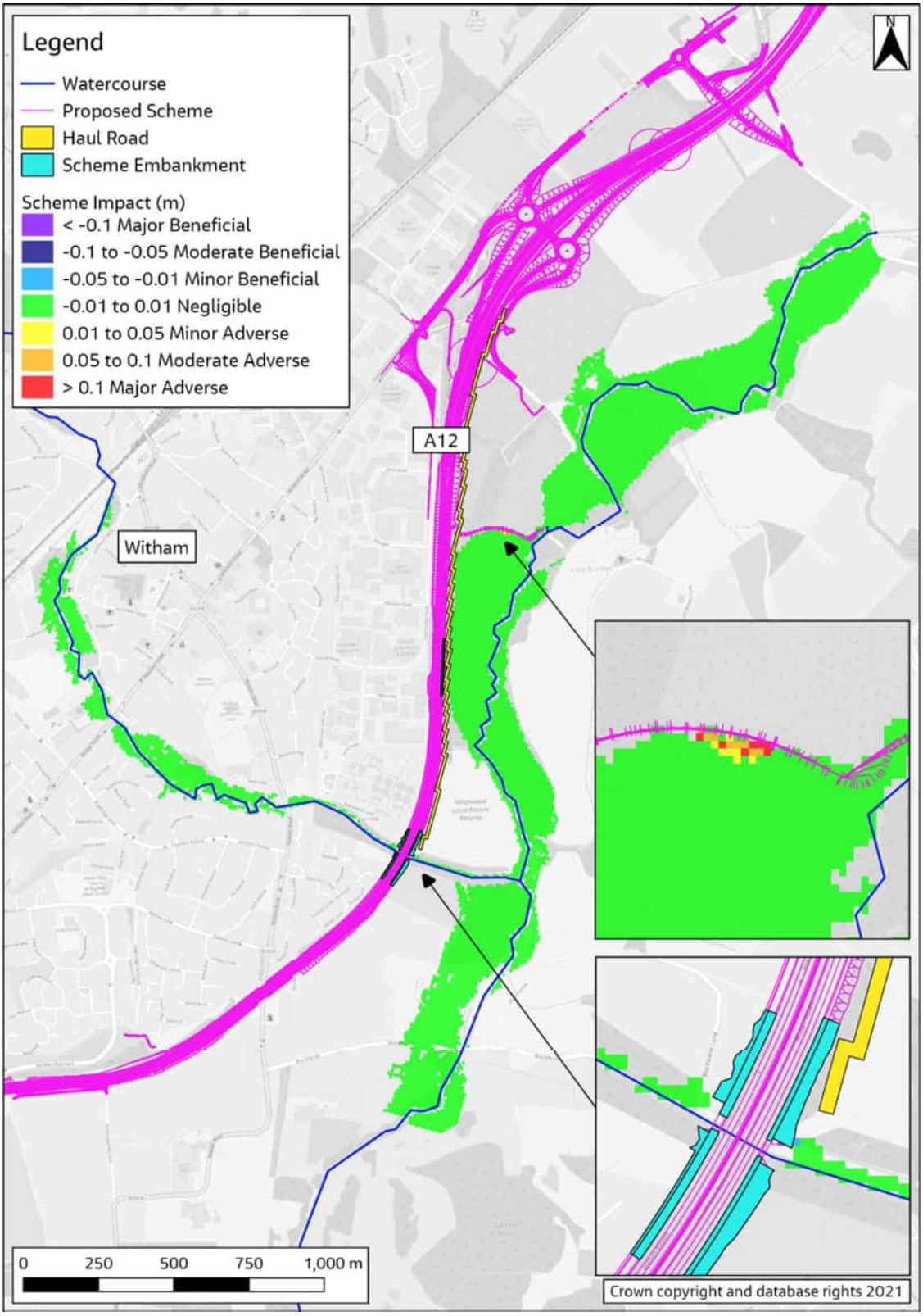


Plate D.6 Water Level Difference (With-Scheme and Temporary Works minus Baseline) for the 1% (1 in 100) AEP event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVER BRAIN HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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1 Executive Summary

1.1.1 This document has been prepared to support the A12 Lower Blackwater and Brain Baseline with scheme hydraulic modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This record sheet is to be used where flood estimates are needed at a single location.

3 Method Statement

3.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Peak flow or hydrograph? • Range of return periods • Approx. time available	The A12 is proposed to be widened and therefore design peak flow and hydrographs are required for input into the River Blackwater and River Brain numerical hydraulic models in order to assess flood risk at the A12 crossing and in the vicinity of the A12 road improvements. Design peak flows and hydrographs are required for the following Annual Exceedance Probability (AEP) events: 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 1% plus climate change adjustment (plus CC), 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 100 plus CC, 200 and 1000-year design return periods). For the climate change uplift the following factor will be adopted: • 1% AEP event plus 38% uplift (based on the Environment Agency (EA) 2021 guidance[1] - Combined Essex Management Catchment peak river flow allowances higher, 2080s scenario). Hydraulic modelling of the River Blackwater and River Brain was undertaken by CH2M in 2017 as part of a flood mapping exercise (refer to Annex A.1 for the location of the two catchments). After a review of the CH2M hydrological analysis it has been decided to retain some of the ReFH1 method IED units for the River Blackwater model but with new ReFH1 method IED units being created at the start of the Jacobs hydraulic model on the River Blackwater and River Brain. This is due to this assessment only being required to assess flood risk in the vicinity of the A12 road improvements

[1] Environment Agency (2021). Flood Risk Assessments: Climate Change Allowances guidance.

	scheme and not for the full River Blackwater and River Brain catchments. In order to incorporate increased availability of data post-2017 assessment and updates to WINFAP software, the design peak flows in the vicinity of the scheme will also be updated for this assessment. This assessment will be undertaken using the Appleford (37010) gauge on the River Blackwater and Guithavon Valley (37009) gauge on the River Brain which are located within the Jacobs model extents. For further details refer to Annex A.2 (River Blackwater and River Brain Hydrology Update Memo - issued to the Environment Agency April 2020).

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The River Blackwater and River Brain are located in the south east of England. The River Blackwater has a catchment area of approximately 350km^2 to Maldon where it flows into the North Sea. The River Blackwater has a relatively rural catchment with urban areas such as Braintree, Coggeshall, Witham and Maldon. The River Blackwater has five flow / level gauges in the catchment: Copford Hall, Stisted, Appleford, Guithavon Valley (on the River Brain) and Langford. The Appleford gauge is located within the modelled section of the River Blackwater. The River Blackwater flow regime is identified (NRFA data sheet) as being affected by groundwater abstractions and intermittently affected since 1971 by the Ely/Ouse Transfer Scheme pumping. The predominant land use is arable, with areas of grassland and woodland. The geology of the catchment has been assessed as being composed of predominantly London Clay with chalk in the headwaters, overlain with boulder clay and some sand and gravels. The soils within the catchment have been assessed using Landis[2] as predominantly lime rich loamy and clayey soils with impeded drainage in the upper reaches and freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The River Brain is a tributary of the River Blackwater and has a catchment area of 67.7km^2 to its confluence with the River Blackwater just downstream of the A12 crossing. The River Brain has a moderately urbanised catchment with urban areas including Witham and Braintree. The River Brain has the Guithavon Valley (37009) gauge located within its catchment. The Guithavon Valley gauge is located within the town of Witham. The predominant land

[2] http://www.landis.org.uk/soilscapes/

	use within the catchment is arable with areas of grassland and woodland. The geology of the catchment is London Clay with superficial deposits mainly Bolder Clay and some sands and gravels. The soils within the catchment have been assessed using Landis as being composed of predominantly lime rich loamy and clayey soils with impeded drainage, freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils.

3.3 Source of flood peak data

Was the NRFA/HiFlows UK dataset used? If so, which version? Record any changes made.	Yes, WINFAPv4 was used with the National Rivers Flow Archive Peak Flow Dataset version 8 (downloaded March 2020). These contain the most up to date Peak Flows dataset at the time of assessment. It is noted that both the Appleford and Guithavon Amax series have been revised since the CH2M (2017) analysis. It appears that the EA have reviewed these series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. This EA reviewed AMAX data has been used.

3.4 Gauging stations (flow or level)

3.4.1 As noted above the River Blackwater has five flow/level gauges located within its catchment (Copford Hall, Stisted, Appleford, Guithavon Valley and Langford). The Appleford Gauge (37010) is located within the model extent. The River Brain further has the Guithavon Valley gauge (37009) located within its catchment within the model extent.

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
Blackwater	Appleford Bridge	037010	37010	TL844158	247.3	Level	1962 to present
River Brain	Guithavon Valley	037009	37009	TL817147	60.7	Level	1962 to present
River Pant	Copford Hall	037016	37016	TL668313	62.5	Level	1965 to present
Blackwater	Stisted	037017	37017	TL792242	139.2	Level	1969 to present
Blackwater	Langford	N/A	37004	TL835090	337	Level	1932 to 1968

3.5 Data available at each flow gauging station

Station name	Start and end of data in Peak Flows dataset	Update for this study?	Suitable for QMED?	Suitable for pooling ?	Data quality check needed ?	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Appleford Bridge	1962-2018	No	Yes	Yes	No	Station with poor quality record gauging a groundwater-influenced rural catchment with major artificial influences. Groundwater - influenced regime. Intermittently affected, since 1971, by Ely / Ouse Transfer Scheme pumping, significantly affected in the late 1980s and in the 1990s. Abstractions from both chalk and gravel aquifers for private water supply. Previous studies by CH2M (2017) indicates that the rating curve for this gauge may be unreliable and further spot gauging's particularly at higher flows was recommended to improve the rating. This study has investigated the availability of additional high flows spot gauging's post the CH2M (2017) study as uncertainty with higher flows was noted. At the present time no additional high flows spot gaugings are available to improve the rating. This gauge has been assessed as suitable for QMED but there is a degree of uncertainty with the higher AMAX values. It appears that the EA have reviewed since 2017 the AMAX series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. The new interpretation appears in the current Peak Flows dataset which is used in this study.

Station name	Start and end of data in Peak Flows dataset	Update for this study?	Suitable for QMED?	Suitable for pooling ?	Data quality check needed ?	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Guithavon Valley	1962-2018	Yes*	Yes	Yes	No	Runoff increased by effluent returns and reduced by industrial /agricultural abstraction. Drowning noted to occur at low levels but noted to have minimum effect. A previous study by CH2M (2017) suggests further work is required to improve the confidence in the rating. This study has investigated the availability of additional high flows spot gauging's post the CH2M (2017) study but at the present time no additional high flows spot gauging's are available to improve the rating. This gauge has been assessed as suitable for QMED but there is a degree of uncertainty with the higher AMAX values. It appears that the EA have reviewed since 2017 the AMAX series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. The new interpretation appears in the current Peak Flows dataset which is used in this study. The Guithavon Valley updated QMED based on the revised AMAX data is similar to the previous CH2M QMED.
Copford Hall	1962-2018	No	Yes	Yes	No	No POT data are presented for this station as they have no non-modular flow correction applied. Intermittent pumping of Ely/Ouse Transfer Scheme has major effect on station, which is only 5km downstream of the Great Sampford Outfall. Some industrial / agricultural abstraction.

Station name	**Start and end of data in Peak Flows dataset**	**Update for this study?**	**Suitable for QMED?**	**Suitable for pooling ?**	**Data quality check needed ?**	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Stisted	1969-2018	No	Yes	No	No	No POT data are present for this station as they have no non-modular flow correction applied. AMAX data have been corrected. Flows can be artificially increased by Ely Ouse transfer scheme. Urban runoff from Braintree. Minor adjustments need for ground and surface water abstractions, sewage and industrial effluent and Ely Ouse/Essex discharges. Net effect – minor export of water from catchment.
Langford	1932-1968	No	No	No	No	Quality of data unknown. Complex of weirs and sluices gauging River Blackwater and monitoring intake on what was the Southend Water Company.
Give link/reference to any further data quality checks carried out		No further data quality checks were undertaken.				

** one additional AMAX value added for water year 2018*

3.6 Rating equations

Station name	**Type of rating** e.g. theoretical, empirical; degree of extrapolation	**Rating review needed?**	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Appleford Bridge	N/A	No	Full rating review outside the scope of this study. The rating curve for Appleford Bridge was assessed by CH2M (2017) and Hydro-Logic (2013-2014). The CH2M study recommended that further spot gauging's should be carried out at higher flows and the rating reviewed as there were discrepancies noted in the consistency of flow estimates between gauges on the River Blackwater. This study has investigated the availability of additional high flows spot gauging's post the CH2M study as uncertainty with higher flows was noted. At the present time no additional high flows spot gaugings are available to improve the rating. It appears that the EA have reviewed since 2017 the full AMAX series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. The new interpretation appears in the current Peak Flows dataset which is used in this study.
Guithavon Valley	N/A	No	Detailed rating review outside the scope of this study. A rating review was carried out for Guithavon Valley by CH2M (2017) and Hydro-Logic (2013-2014). The CH2M study noted the rating curve for Guithavon Valley was low and it recommended that further spot gaugings should be carried out particularly at higher flows and the rating reviewed. CH2M then derived a rating for their study from their hydraulic model. This resulted in an unrealistically steep growth curve. This study has investigated the availability of additional high flows spot gauging's post the CH2M (2017) study as uncertainty with higher AMAX values is noted. At the present time no additional high flows spot gaugings are available to improve the rating.
Copford Hall	N/A	No	Rating review outside the scope of this study.

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Stisted	N/A	No	Rating review outside the scope of this study.
Langford	N/A	No	Rating review outside the scope of this study.
Give link/reference to any rating reviews carried out			

3.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)	No - detailed rating reviews are outwith the scope of this study. There is also insufficient high flows spot gaugings to update the current ratings at the present time. If more high flows gaugings become available, then a rating review for the Appleford (37010) and Guithavon (37009) gauges would be desirable.				
Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies (e.g. CFMPs, Strategies)					
Other data or information (e.g. groundwater, tides)					

3.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes - FEH Statistical method considered appropriate for both the River Blackwater and River Brain. As both catchments have gauges within the model extents (River Blackwater @ Appleford (37010) – and River Brain @ Guithavon Valley (37009)) both FEH Statistical

	ESS analysis and standard Pooling group analysis will be undertaken for the Appleford and Guithavon gauges. Since there remains some uncertainty with the robustness of the higher peak flow estimates at these gauges the steeper growth curve from the two methods (ESS and standard pooling group analyses) will be used to be precautionary. (The index flood, QMED to be based on the gauged data where there is more confidence in the flows). Corresponding peak flows will also be generated using ReFH2.3 method for comparison purposes. Neither the River Blackwater nor the River Brain are understood to have atypically large amounts of flood storage within their catchments. On this basis the peak flows will be established based upon the precautionary statistical values and the model will be run using ReFH1 inflow boundaries (as in the CH2M 2017 study) in which the storm duration is set to the default catchment descriptors value. These inflows will then be uniformly scaled to gain reconciliation with the statistically derived target flows. (This helps avoid the recognised[3] water balance problem associated with ReFH1 which can manifest itself when storm duration optimisation is used). Since the model being used does not need to represent all the sub-catchments upstream of where the A12 model extent starts, a new single catchment ReFH1 representation is used to supply the upstream inflow at the start of each model. For the Middle Blackwater model, the new single catchment ReFH1 representation is to the point on the River Blackwater just upstream (586750 218850) of the Domsey Brook confluence. The CH2M boundary units DB1, B45, R1 are then adopted below this location. For the Lower Blackwater model, the new single catchment ReFH1 representation for the River Blackwater is to the Appleford gauging station with B60 and B80 CH2M boundary units being adopted below this location. For the River Brain

[3] WHS, 2016. The Revitalised Flood Hydrograph Model ReFH2.2: Technical Guidance

	the single catchment ReFH1 representation is to the Guithavon Valley gauging station with the CH2M boundary unit BRA30 being adopted below this location.
Outline the conceptual model, addressing questions such as: • What is likely to cause flooding at the site (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might the site flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Flooding in the vicinity of the A12 and the Appleford and Guithavon gauging stations is judged to be due to peak flows and not sensitivity to flood volume issues. This was based upon a preliminary assessment of the results of the existing model, coupled with visual inspection of the EA flood maps for the catchments.
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, use permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	Both catchments are long thin and relatively flat catchments. The River Blackwater catchment to the Appleford gauge has a BFIHOST (0.475) and SPRHOST (40.08) values in the middle of the normal range and do not suggest the catchment to be particularly permeable or impermeable. The River Blackwater @ Appleford (37010) catchment is classified by FEH as 'essentially rural' with only limited urban areas. The FEH Web Service records a FARL of 0.991 for the catchment to the gauge. The geology of the catchment has been noted by the FEH Web Service as having high permeability bedrock in the upper catchment (estimated as making up approximately 25% of catchment), with mixed permeability bed rock making up the remaining 75% of the catchment. The FEH catchment descriptors FPEXT value to the Blackwater @ Appleford gauge is 0.0977. This suggests the Blackwater catchment has some but limited floodplain storage within the catchment. The River Brain catchment to the Guithavon gauge has a BFIHOST (0.534) and SPRHOST (36.43) values in the normal range and do not suggest the catchment to be overly permeable.

	The River Brain @ Guithavon Valley (37009) catchment is classified by FEH as 'moderately urbanised' with urban areas including Witham in the lower catchment and Braintree. The FEH Web Service records a FARL of 0.997 for the River Brain to the Guithavon Valley gauging station. There is therefore little attenuation due to reservoirs/lakes within the River Brain catchments. The geology of the catchment has been noted by the FEH Web Service as being composed of mixed permeability bedrock. The FEH catchment descriptors FPEXT value to the Brain @ Guithavon Valley gauge is 0.0839. This suggests the River Brain catchment has some but limited floodplain storage within the catchment.
Initial choice of method(s) and reasons	The FEH Statistical method (ESS and Pooling) has been undertaken for both the Blackwater @ Appleford (37010) and the Brain @ Guithavon Valley (37009) gauges. ReFH2.3 was also undertaken for these catchments for comparison purposes. Design peak flows were then adopted following a comparison of the peak flows from the FEH Statistical methods. The most conservative of the FEH Statistical peak flows was adopted as there is uncertainty associated with the higher flows for both gauges. The design hydrograph shape is based on the output from the ReFH1 IEDs which was uniformly scaled to agree with the finalised peak flow within the model at the Blackwater @ Appleford (37010) gauge for the River Blackwater model and the Brain @ Guithavon Valley (37009) gauge for the River Brain. Flows were then checked at the target locations within the model – in the vicinity of the A12 and reconciled where appropriate.
Software to be used (with version numbers if relevant) e.g. FEH Web Service (date accessed) WINFAPv4 WINFAP-FEH v5 data files (previously HiFlows) ReFH2.2	FEH Web Service – accessed March 2020 WINFAPv4 NRFA Peak Flow Dataset - downloaded March 2020 ReFH1 – IEDs only (for hydrograph shape) ReFH2.3 / FEH13 rainfall data

3.9 Site details

Watercourse	Site	Easting	Northing	AREA on FEH Web service(km^2)	Revised AREA if altered (km^2)
River Blackwater	@ A12	585550	217650	223.58	223.58
River Blackwater	@ Appleford Gauge	584450	215800	249.12	249.12
River Brain	@ Guithavon Valley	581800	214750	60.27	60.27
River Brain	@ A12	582750	213700	66.64	67.65

3.10 Catchment descriptors (incorporating any changes made)

Watercourse	Site	FARL	PROPWET	BFIHOST	BFIHOST 2019	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT 2000	FPEXT
River Blackwater	@ A12	0.993	0.26	0.484	0.469	27.86	26.50	573	39.67	0.0253	0.0933
River Blackwater	@ Appleford Gauge	0.991	0.26	0.475	0.461	28.01	25.90	572	40.08	0.026	0.0977
River Brain	@ Guithavon Valley	0.997	0.30	0.534	0.514*	14.02	24.2	572	36.43	0.0737	0.0839
River Brain	@ A12	0.997	0.29	0.531	0.514	14.63	23.60	570	36.69	0.0848	0.0852

**Assessed using River Brain @ Blackwater FEH catchment descriptors as a donor as values not provided in WINFAP.*

3.11 Checking catchment descriptors

Record how the catchment boundary was checked and describe any changes (refer to maps if needed)	The River Blackwater catchment at the Appleford gauging station (584450 215800) and at the River Blackwater catchment at the A12 (585550 217650) were checked using ordnance survey mapping data. The River Brain catchment at the Guithavon Valley gauge (581800 214750) and at the A12 (582750 213700) were also checked using ordnance survey mapping data and amended slightly for this assessment.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	The soils within the River Blackwater catchment have been assessed using Landis which reports soils are predominantly lime rich loamy and clayey soils with impeded drainage in the upper reaches and freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The soils within the River Brain catchment have also been assessed using Landis which reports soils are predominantly lime rich loamy and clayey soils with impeded drainage, freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. Based on this assessment the FEH SPRHOST / BFIHOST values were not amended.
Source of URBEXT	$URBEXT_{2000}$ has been extracted for the catchments from the FEH Web Service / from WINFAP (for the River Brain @ Guithavon Valley (37009)).
Method for updating of URBEXT	$URBEXT_{2000}$ amended within WINFAPv4 using urban adjustment procedures.

4 Statistical Method

4.1 Overview of estimation of QMED

Used	Condition	Approach followed
x	N >=30	Estimate QMED using annual maxima
	14=< N =<29	Estimate QMED from annual maxima & optionally adjust for climatic variation
	2=< N= <13	Estimate QMED from POT data & adjust for climatic variation
	N <2 & suitable donor site with 20 years or more of record	Ignore record at subject site; transfer QMED from donor site
	N <2 & suitable donor with 10 to 19 years of record & 12 month overlap between records	Estimate QMED using procedure based on flood peak regression
	N <2 & suitable donor with 10 to 19 years of record but no 12 month overlap	Ignore record at subject site; transfer QMED from donor site
	N <2 & no long-record site nearby	Estimate QMED from very short POT record
	N <2 & no long-record site nearby	Treat site as ungauged catchment
	N <2 & no long-record site nearby	Defer analysis until longer flow record available
	N <2 & no long-record site nearby	(Abstract flood event information and apply the UH rainfall-runoff model as an alternative, to the pooling group procedure. Particularly recommended when site is urbanised)
X	Ungauged catchment	Estimate QMED from catchment descriptors
X	Ungauged catchment	Estimate QMED by data transfer from donor catchment

	Ungauged catchment	Estimate QMED by data transfer from analogue catchment
	Ungauged catchment	Estimate QMED from channel dimensions
	Ungauged catchment	Compare to regional pattern of mapped QMED adjustment factors

4.2 Search for donor sites for QMED

Comment on potential donor sites

Mention:

- Number of potential donor sites available
- Distances from subject site
- Similarity in terms of AREA, URBEXT, FARL and other catchment descriptors
- Quality of flood peak data

Include a map if necessary. Note that donor catchments should usually be rural.

The River Blackwater has a number of flow gauges including a gauge at Stisted, Appleford and Langford. The Appleford gauging station (37010) is located within the River Blackwater model extent and has been assessed (by Peak Flows dataset) as suitable for QMED and Pooling. The AMAX data from this gauge will therefore be used in this analysis to derive QMED at the gauge and also will be used as a QMED donor for the target location the A12 (585550 217650).

The River Brain also has the Brain @ Guithavon Valley gauging station which is also assessed (by Peak Flows dataset) as suitable for QMED and Pooling. The Guithavon Valley gauging station is also located within the River Brain model extent and therefore the AMAX series from this gauge will be used for QMED estimation at the gauge and as a QMED donor at the target location the A12 (582750 213700). The 2018 water year AMAX value supplied by the Environment Agency for this study was added to the AMAX series.

The CDs and QMED at the target locations (the River Blackwater @ A12, the River Blackwater @ Appleford, the River Brain @ Guithavon Valley and the River Brain @ the A12) are presented below.

NAME	River Blackwater @ A12	River Blackwater @ Appleford	River Brain @ Guithavon Valley	River Brain @ A12
Gauge No.	-	37010	37009	-
AREA	223.58	249.12	60.27	67.65
SAAR	573	572	572	570
FARL	0.993	0.991	0.997	0.997
SPRHOST	39.67	40.08	36.4	36.69
BFIHOST	0.484	0.475	0.534	0.531
Years	-	56*	57**	-
QMEDcd's	15.4	17.1	4.6	5.2
QMED data	-	13.2	3.9	-
Ratio(R) $QMED_{data}$/ $QMED_{cds}$	-	0.767	0.838	-

**AMAX data from 2018 water year not added as noted as suspect and comment reported drift. **AMAX data from 2018 water year added.*

Method	Location	Initial estimate of QMED (m^3/s)	Data transfer				Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	
AM	River Blackwater @ A12	15.41	37010	1.31	0.736	0.823	12. 68*
AM	River Blackwater @ Appleford	-	37010	-	-	-	13.15
AM	River Brain @ Guithavon Valley	-	37009	-	-	-	3.89
AM	River Brain @ the A12	5.18	37009	1.00	0.786	0.871	4.51**
		Notes Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone. When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added below. When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why. The data transfer procedure is the revised one from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors. If more than one donor has been used, give the weights used in the averaging.				

**Final estimate of QMED extracted from WINFAP using Appleford (37010) as donor.*

***Final estimate of QMED extracted from WINFAP using Guithavon (37009) as donor.*

Final estimate of Qmed from FEH Statistical method

QMED (River Blackwater @ A12) = 12.68 m^3/s

QMED (River Blackwater @ Appleford) = 13.15m^3/s

QMED (River Brain @ Guithavon) = 3.89m^3/s

QMED (River Brain @ A12) = 4.51 m^3/s

4.3 Derivation of pooling group – River Blackwater

Name of saved .feh group file: Appleford (37010)
Target return period (years) for 5T rule *: T = 100 years

*(Updated recommendations advise that a standard 500 years is used regardless of target return period i.e.T=100yr)

Target site: River Blackwater @ Appleford (gauge 37010)

Total number of sites	12
Total number of years	583
Total number of initial highly discordant sites	0
Discordant sites	
Total number of short records (< 7 years) removed	0
List sites removed with reasons	High SPRHOST (>0.65) – 30003, 43005
Number of pooled years after sites removed	498

Is subject site included in pooled group: yes / ~~no~~

If no state reason why: _____

Test statistics on validity of pooling group for flood frequency analysis

Heterogeneity test H2 value = 2.3468

Status	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable	X	2 < H2 < 4
	Review essential		H2 > 4

				Value
Goodness-of-fit test	Z values	GL	acceptable /~~not acceptable~~	0.8731
		GEV	acceptable /~~not acceptable~~	-1.5780
		PT3	~~acceptable~~ /not acceptable	-2.4514
	other			

(Note: in the FEH the GL is the generally favored distribution for use)

ACTION is construction of flood frequency curve valid?

YES: / NO: review the pooling group further

Comment?	No; need to add sites to increase pooling group above 500 years

4.4 Revision of Pooling Group

Revision No. 1

Station Number	Reason for changes in pooling group
20001	Added to increase pooling group above 500 years.

Number of sites 11 Years 520

Heterogeneity test H2 value = 2.0376

Status			
	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable	/	2 < H2 < 4
	Review essential		H2 > 4

Note: FEH Vol.3, chapter 16.3.2: “The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review.”

				Value
Goodness-of-fit test	Z values	GL	acceptable /~~not acceptable~~	0.0746
		GEV	~~acceptable~~ /not acceptable	-2.2328
		PT3	~~acceptable~~ /not acceptable	-2.9849
	other			

ACTION	Is construction of flood frequency curve valid?	
~~No~~	Yes	
	Comment ?	Pooled group has been reviewed, therefore proceed to construction of FFC.

Derivation of flood growth curves

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 1% AEP**
ESS	GL – L-median GEV – L-median		2.930 2.786
Pooling	GL – L-median GEV – L-median	-	2.797 2.662
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

Refer to Annex A.3 to A.5 for final pooling group details.

4.5 Flood estimates from the statistical method – Enhanced Single Site

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.438	1.745	2.066	2.174	2.265	2.533	2.760	2.930	3.372	4.037	4.610
River Blackwater @ A12*	12.7	18.2	22.1	26.2	27.6	28.7	32.1	35.0	37.2	42.8	51.2	58.5
River Blackwater @ Appleford**	13.2	18.9	22.9	27.2	28.6	29.8	33.3	36.3	38.5	44.3	53.1	60.6

**used River Blackwater @ Appleford growth curve as donor. **used urbanised flood frequency results.*

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.485	1.804	2.107	2.203	2.281	2.497	2.667	2.786	3.073	3.448	3.73
River Blackwater @ Appleford	13.2	19.5	23.7	27.7	29.0	30.0	32.8	35.1	36.6	40.4	45.3	49.0

4.6 Flood estimates from the statistical method – Pooling

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.408	1.693	1.992	2.093	2.178	2.427	2.638	2.797	3.208	3.827	4.360
River Blackwater @ A12*	12.7	17.9	21.5	25.3	26.5	27.6	30.8	33.4	35.5	40.7	48.5	55.3
River Blackwater @ Appleford	13.2	18.5	22.3	26.2	27.5	28.6	31.9	34.7	36.8	42.2	50.3	57.3

**used River Blackwater @ Appleford growth curve as donor.*

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.452	1.748	2.030	2.119	2.192	2.392	2.550	2.662	2.929	3.278	3.540
River Blackwater @ Appleford	13.2	19.1	23.0	26.7	27.9	28.8	31.5	33.5	35.0	38.5	43.1	46.6

Refer to Annex A.3 to A.5 for the details on pooling group analysis at River Blackwater at Appleford gauge.

4.7 Derivation of pooling group – River Brain @ Guithavon

Name of saved .feh group file: Guithavon (37009)
Target return period (years) for 5T rule *: T = 100 years

*(Updated recommendations advise that a standard 500 years is used regardless of target return period i.e.T=100yr)

Target site: River Brain @ Guithavon

Total number of sites	10
Total number of years	516
Total number of initial highly discordant sites	1
Discordant sites	26013
Total number of short records (< 7 years) removed	0
List sites removed with reasons	26013, 26003, 33032 – removed as permeable catchments (BFIHOST>0.65)
Number of pooled years after sites removed	401

Is subject site included in pooled group: ~~yes~~ / no

If no state reason why: Target site not included in standard pooling as moderately urbanized catchment. Subject site included in ESS analysis.

Test statistics on validity of pooling group for flood frequency analysis

Heterogeneity test H2 value = -0.0464

Status	Review not necessary	X	H2 < 1
	Review optional		1 < H2 < 2
	Review desirable		2 < H2 < 4
	Review essential		H2 > 4

					Value
Goodness-of-fit test	Z values	GL	~~acceptable~~ / not acceptable		2.7097
		GEV	acceptable / ~~not acceptable~~		-0.6755
		PT3	acceptable / ~~not acceptable~~		-0.4797
	other				

(Note: in the FEH the GL is the generally favored distribution for use)

ACTION is construction of flood frequency curve valid?

YES: / NO: review the pooling group further

Comment ?	No; need to add sites to increase pooling group above 500 years.

4.8 Revision of Pooling Group – River Brain @ Guithavon

Revision No. 1

Station Number	Reason for changes in pooling group
37014 / 54036	To increase pooling group above 500 years.

Number of sites 9 Years 501

Heterogeneity test H2 value = 1.8506

Status			
	Review not necessary		H2 < 1
	Review optional	X	1 < H2 < 2
	Review desirable		2 < H2 < 4
	Review essential		H2 > 4

Note: FEH Vol.3, chapter 16.3.2: “The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review.”

				Value
Goodness-of-fit test	Z values	GL	acceptable /~~not acceptable~~	0.1297
		GEV	~~acceptable~~ /not acceptable	-2.9096
		PT3	~~acceptable~~ /not acceptable	-2.7412
	other			

ACTION	Is construction of flood frequency curve valid?	
~~No~~	Yes	
	Comment ?	Pooled group has been reviewed, therefore proceed to construction of FFC.

Derivation of flood growth curves

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 100-year return period**
ESS	GL – L-median GEV – L-median		3.674 3.501
P	GL – L-median GEV – L-median	-	2.715 2.544
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

Refer to Annex A.6 to A.8 for final pooling group details.

4.9 Flood estimates from the statistical method - Enhanced Single Site – River Brain @ Guithavon

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	5	10	20	25	30	50	75	100	200	500	1000
Growth Factor	1.000	1.587	2.005	2.450	2.602	2.729	3.107	3.431	3.674	4.314	5.290	6.142
River Brain @ Guithavon Valley	3.9	6.2	7.8	9.5	10.1	10.6	12.1	13.3	14.3	16.8	20.6	23.9
River Brain @ the A12*	4.5	7.2	9.0	11.1	11.7	12.3	14.0	15.5	16.6	19.5	23.9	27.7

**The River Brain @ Guithavon growth curve will be used as a donor for the target site the River Brain @ A12 which is located a short distance downstream of the gauge.*

For the ESS analysis the River Brain @ Guithavon (37009) was included in the statistical analysis. This gauge is not included in the pooling group analysis due to the catchment being moderately urbanised. In order to mitigate this the L-Moments were deurbanised.

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	5	10	20	25	30	50	75	100	200	500	1000
Growth Factor	1.000	1.650	2.089	2.515	2.651	2.763	3.075	3.324	3.501	3.932	4.508	4.950
River Brain @ Guithavon Valley	3.9	6.4	8.1	9.8	10.3	10.7	12.0	12.9	13.6	15.3	17.5	19.3

Refer to Annex A.6 to A.8 for the details on pooling group analysis at River Brain at Guithavon Valley gauge.

4.10 Flood estimates from the statistical method – Pooling – River Brain @ Guithavon

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.436	1.721	2.006	2.099	2.176	2.398	2.581	2.715	3.053	3.536	3.932
River Brain @ Guithavon Valley	3.9	5.6	6.7	7.8	8.2	8.5	9.3	10.0	10.6	11.9	13.8	15.3
River Brain @ the A12	4.5	6.5	7.8	9.0	9.5	9.8	10.8	11.6	12.2	13.8	15.9	17.7

**The River Brain @ Guithavon growth curve will be used as a donor for the target site the River Brain @ A12 which is located a short distance downstream of the gauge.*

The River Brain @ Guithavon (37009) is not included in the FEH Statistical pooling analysis due to the moderately urbanised catchment.

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.486	1.776	2.033	2.110	2.171	2.336	2.460	2.544	2.736	2.968	3.128
River Brain @ Guithavon Valley	3.9	5.8	6.9	7.9	8.2	8.4	9.1	9.6	9.9	10.6	11.5	12.2

5 Revitalised flood hydrograph (ReFH2.3) method

5.1 Parameters for ReFH2.3 model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible, so it is not essential to enter them in the table.

Location	**Method:** OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	**Tp (hours)** Time to peak	**C_{max} (mm)** Maximum storage capacity	**BL (hours)** Baseflow lag	**BR** Baseflow recharge
River Blackwater @ A12	CD	21.1	399	74.9	2.37
River Blackwater @ Appleford	CD	21.3	391	74.2	2.27
River Brain @ Guithavon Valley	CD	12.4	435	63.9	2.76
River Brain @ the A12	CD	13.4	438	65.5	2.74
Brief description of any flood event analysis carried out (further details should be given below or in a project report)		No flood event analysis carried out.			

5.2 Design events for calculating peak flow using ReFH2.3 method

Location	**Urban or rural**	**Season of design event (summer or winter)**	**Storm duration (hours)**	**Storm area for ARF** (if not catchment area)
River Blackwater @ A12	Rural	Winter	33.0	Catchment area
River Blackwater @ Appleford	Rural	Winter	33.0	Catchment area

Location	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
River Brain @ Guithavon Valley	Rural	Winter	18.0	Catchment area
River Brain @ the A12	Rural	Winter	22.0	Catchment area

5.3 Peak flood estimates from the ReFH2.3 method (based upon the sub-catchment specific critical storm durations)

Using ReFH2.3 Duration	Flood peak (m^3/s) for the following return periods (in years)											
	2	5	10	20	25	30	50	75	100	200	500	1000
River Blackwater @ A12	17.9	23.1	26.5	30.0	31.2	32.1	35.2	38.0	40.4	48.5	63.1	74.8
River Blackwater @ Appleford	20.2	26.0	29.9	33.8	35.1	36.2	39.6	42.8	45.5	54.6	71.1	84.3
River Brain @ Guithavon Valley	5.7	7.6	8.9	10.2	10.6	11.0	12.2	13.3	14.2	17.5	23.5	28.3
River Brain @ the A12	6.2	8.2	9.6	11.0	11.4	11.8	13.1	14.2	15.2	18.7	25.0	30.1

6 Discussion and summary of results

6.1 Comparison of results from different methods

This table compares peak flows from the two methods for two key return periods.

	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	FEH Statistical ESS	FEH Statistical Pooling	ReFH2.3	FEH Statistical ESS	FEH Statistical Pooling	ReFH2.3
River Blackwater @ A12	12.7	12.7	17.9	**37.2**	35.5	40.4
River Blackwater @ Appleford	13.2	13.2	20.2	**38.5**	36.8	45.5
River Brain @ Guithavon Valley	3.9	3.9	5.7	**14.3**	10.6	14.2
River Brain @ the A12	4.5	4.5	6.2	**16.6**	12.2	15.2

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	ReFH2.3 generally produces higher peak flows than the FEH Statistical methods (ESS and Pooling) for the River Blackwater and River Brain catchments. The River Blackwater @ Appleford gauge and the Brain @ Guithavon gauges are however assessed as suitable for estimating QMED. Both gauges have been assessed as also suitable for pooling however there is less confidence in higher flows estimates at the gauges. Therefore, it has been decided to take forward the higher FEH Statistical ESS peak flow estimated up to and including the 1% AEP event (100-year return period) and then use ReFH ratios for all flow estimates above the 1% AEP event. Refer to Annex A.9 for the peak flow comparison from these methods. To be consistent with the current ReFH1 method based IED boundary units used within the Blackwater hydraulic model, the hydrograph shapes are derived from the application of ReFH1. As there is limited storage within the River Blackwater and River Brain catchments this is assessed as acceptable. A sensitivity analysis was further carried out within the

	Middle Blackwater model to assess the impact of using the updated ReFH2.3 hydrograph shape (scaled to the target statistical peak flow estimate at the Appleford gauge) for the 1% AEP event (100-year return period). The results of the sensitivity analysis indicated that the ReFH1 hydrograph shape (scaled to the final peak flow estimate) produces a conservative flood extent outline at the A12 crossing location of the River Blackwater in comparison to that produced when using the ReFH2.3 flood hydrograph shape. The typical hydrograph for the 1% AEP event (100-year return period), at the two gauges, are presented in Annex A.10. Further details of the sensitivity analysis are presented in Annex A.11, which also address the comments by the EA (refer to Annex A.12). For the Middle Blackwater model one new ReFH1 boundary unit was created at the top of the Jacobs model extent just upstream of the Domsey Brook confluence with the River Blackwater (586750, 218850), as modelling is only required in the vicinity of the A12. The IEDs for the lateral inflows (DB1, B45, R1) from the original River Blackwater model were retained. For the Lower Blackwater and Brain model a new ReFH1 boundary unit was created at the top of the Jacobs model extent on the River Blackwater at the Appleford gauging station. The IEDs for the River Blackwater lateral inflows B60 and B80 were retained. A new ReFH1 boundary unit was further created for the River Brain to the Guithavon Valley gauging station with the CH2M BRA30 ReFH1 lateral inflow retained. The final design peak flows at the River Blackwater @ Appleford (37010) and River Brain @ Guithavon (37009) gauges are suitable to use as a target flow to reconcile the modelled flow at the gauges. A check on the peak flow at the target locations; the River Blackwater @ A12 and the River Brain @ A12 was also carried out and adjustments considered where necessary.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions used in this study are as follows: • ReFH1 hydrograph shapes are suitable to use in this investigation. • ReFH2.3 is likely to be overestimating lower return periods design peak flows for the target catchments (based on gauged data for the River Blackwater @ Appleford and River Brain @ Guithavon gauges). • The River Blackwater @ Appleford and River Brain @ Guithavon gauges are suitable to estimate design

	peak flows up to the 1% AEP (100-year return period) event. • ReFH2.3 ratios are more appropriate from the 0.5% AEP (200-year return period) event.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH **3** 12.5 or the factorial standard error from Science Report SC050050 (2008).	QMED for the River Blackwater @ Appleford (37010) gauge and River Brain @ Guithavon (37009) gauge has been estimated using AMAX data from the gauges. It is assumed that the AMAX data at both gauges is suitable for QMED.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	The design peak flow estimates have been derived specifically for the River Blackwater and River Brain for modelling of potential impacts of A12 road improvements scheme. Before the flows are used in future studies there should be an assessment of their suitability for the issue in question. Flow derivation methods change in time and this should be taken into consideration.
Give any other comments on the study, for example suggestions for additional work.	If additional high flows spot gaugings become available, the ratings for the River Blackwater @ Appleford (37010) and River Brain @ Guithavon (37009) gauges should be reviewed. There is some uncertainty regarding the higher flow estimates at these gauges.

6.4 Checks

What do the results imply regarding the return periods of floods during the period of record?	River Blackwater @ Appleford (37010) gauge: the biggest recorded flow (22 Oct 2001) maps on to 0.2% AEP (500-year return period) event with an estimated flow of approximately 61m^3/s. Though the NFRA website says, "*Exceptionally high flow recorded in Oct 2001, magnitude suspect as likely to have been affected by drowning*". There are 56-years of AMAX data for this gauge. The second AMAX value (27.9 m^3/s recorded on 11th February 2009) has a rarity of approximately a 5% AEP (20-year return period) event. River Brain @ Guithavon (37009) gauge; the biggest recorded flow (21 Oct 2001) maps onto a 0.2% AEP (500-year return period) event with an estimated flow of

	25.6m^3/s. There are 57-year's worth of AMAX data for this gauge. The second AMAX value (12.5 m^3/s recorded on 10th February 2009) has a rarity of approximately a 2% AEP (50-year return period) event. Caution: the findings of this study suggests that both the gauges struggle to provide robust flow estimates at the highest flows in the record. This suggests that the above rarities are treated with considerable caution.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	The 1% AEP (100-year return period) growth factor for the River Blackwater @ Appleford gauge is 2.93. The growth factor is quite high but within the expected range. The 1% AEP (100-year return period) growth factor for the River Brain @ Guithavon gauge is 3.67. This growth factor is very high and is considered to be towards the upper end of the possible range.
If 1000-year flows have been derived, what is the ratio for 1000-year flow over 100-year flow?	The ratio of the 0.1% AEP (1000-year return period) flow to the 1% AEP (100-year return period) flow for the River Blackwater @ Appleford gauge is 1.85. The ratio of the 0.1% AEP (1000-year return period) flow to the 1% AEP (100-year return period) flow for the River Brain @ Guithavon gauge is 1.99.
What specific runoff (l/s/ha) does the design flow equate to?	The corresponding 1% AEP (100-year) and 50% (2-year) specific runoffs for the River Blackwater @ Appleford catchment are 1.551 l/s/ha and 0.53 l/s/ha respectively. The corresponding 1% AEP (100-year) and 50% (2-year) specific runoffs for the River Brain @ Guithavon catchment are 2.37 l/s/ha and 0.65 l/s/ha respectively.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	Peak flow estimates are available from a previous study of the River Blackwater and River Brain catchments undertaken by CH2M (2017). The CH2M study derived a 1% AEP (100-year return period) flow of 32.9 m^3/s for the River Blackwater @ Appleford gauge. (The current study 38.5 m^3/s). The CH2M study derived a 1% AEP (100-year return period) flow of 20.2 m^3/s for the River Brain @ Guithavon gauge. (The current study 14.3 m^3/s). There is some uncertainty regarding the higher flow estimates at the River Blackwater @ Appleford and River Brain @ Guithavon gauges. In the CH2M (2017) study the rating for the River Brain @ Guithavon gauge was derived from their hydraulic model due to concerns regarding the rating. The resulting CH2M growth curve for the River

	Brain @ Guithavon gauge was however unconvincingly steep.
Are the results compatible with the longer-term flood history?	The modelled flows can be validated within the hydraulic model using readily available flood extents for the River Blackwater and River Brain where available.
Describe any other checks on the results	

6.5 Final results

	Flood peak (m^3/s) for the following return periods (in years) – FEH Statistical ESS and ReFH2.3 Ratios above 1% AEP event												
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**100 + 38%CC Uplift**	**200**	**500**	**1000**
River Blackwater @ A12	12.7	18.2	22.1	26.2	27.6	28.7	32.1	35.0	37.2	51.3	44.5	58.0	68.8
River Blackwater @ Appleford	13.2	18.9	22.9	27.2	28.6	29.8	33.3	36.3	38.5	53.2	46.2	60.1	71.4
River Brain @ Guithavon Valley	3.9	6.2	7.8	9.5	10.1	10.6	12.1	13.3	14.3	19.7	17.6	23.6	28.5
River Brain @ the A12	4.5	7.2	9.0	11.1	11.7	12.3	14.0	15.5	16.6	22.9	20.4	27.4	33.0

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	Design peak flows have been derived for the target locations for the River Blackwater and River Brain hydraulic models. The hydrographs were created using ReFH1 inflow boundary units within the hydraulic models. Modelled flows have been reconciled to agree with the final peak flow estimates presented within this report at the River Blackwater @ Appleford and River Brain @ Guithavon gauges. The modelled peak flows at the target locations the River Blackwater @ A12 and River Brain @ A12 were further checked to ensure flows are appropriate and adjusted where necessary. It should be noted that the ReFH1 boundary units should be run using the design storm duration to the target locations (the A12 scheme). This results in a duration of 29.7 hours for the River Blackwater model and a design duration of 15.9 hours for the River Brain.

Glossary

Term	Definition
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
ESS	Enhanced Single Site flood frequency analysis
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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Centre for Ecology & Hydrology. Flood Estimation Handbook Web Service. Available at: [redacted].

Environment Agency (2021). Flood Risk Assessments: Climate Change Allowances guidance.

Environment Agency (2020). Flood Estimation Guidelines, Technical guidance 197_08.

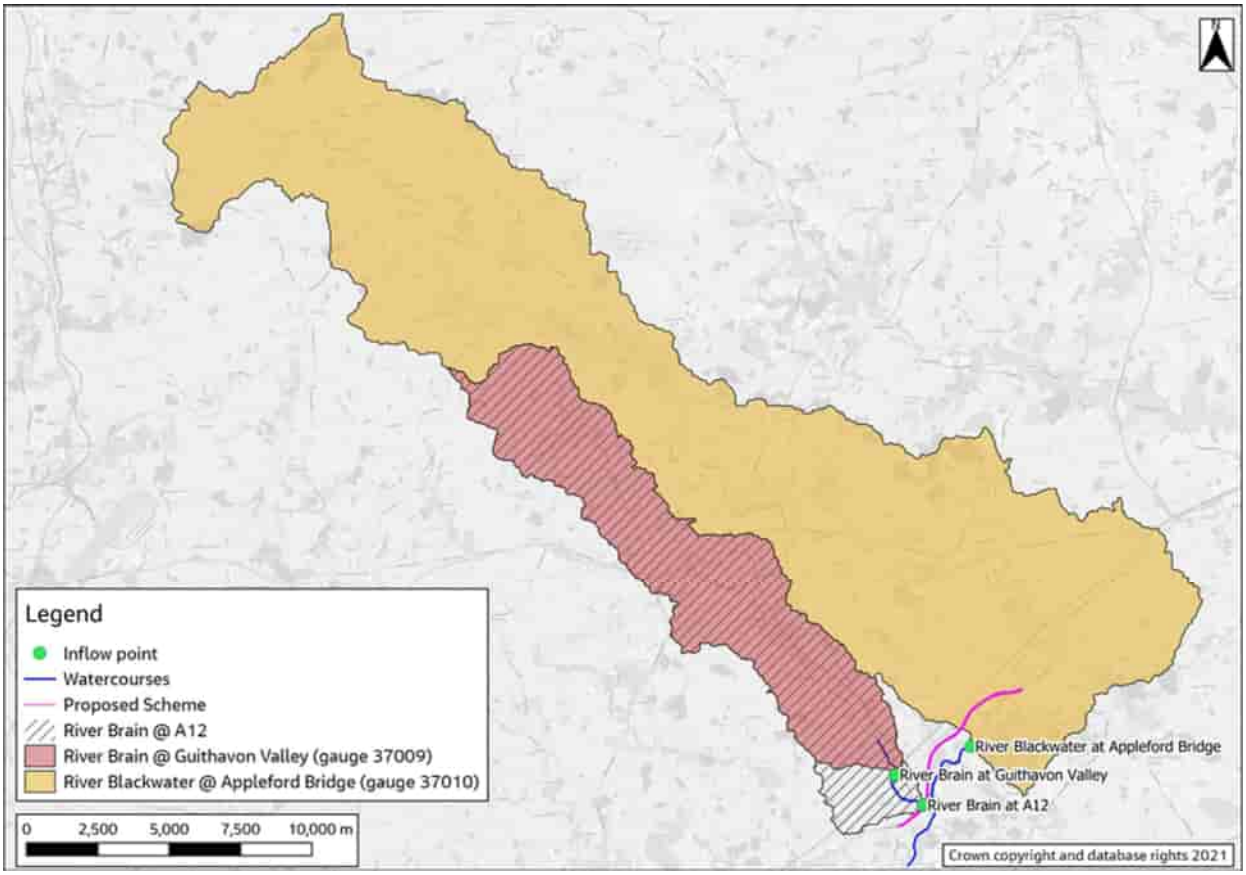
JBA (2010). River Blackwater and Brain study reference not available.

Landis (2020). Available at: http://www.landis.org.uk/soilscapes/.

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Annex A Supporting Information

A.1 Location of catchments



A.2 River Blackwater and River Brain Hydrology Update - A12 Road Widening Scheme (issued to the Environment Agency April 2020)

A review of the hydrology for the River Blackwater Middle hydraulic model and the River Blackwater Lower (including River Brain) hydraulic model has been undertaken to assess whether the CH2M (2017) design flows and hydrographs are suitable to use as input into the Jacobs hydraulic models for the A12 Road Widening Scheme DMRB Stage 3 assessment.

The rivers Blackwater and Brain both have well positioned flow gauges that appear within the Peak Flows dataset. The Appleford gauge on the Blackwater is located within the extent of the hydraulic model, as is the Guithavon gauge on the Brain. The use of these two gauges was central within the CH2M analysis and their use has been the focus of this review which has considered the following:

- the availability of additional new data to refine the understanding (both new spot gaugings and additional annual maximum data),
- the appropriateness of the estimated flows within the annual maximum (Amax) series,
- the methods used to estimate design peak flows,
- improvements in recent updates to software (WINFAP and ReFH) since the 2017 assessment.

The key findings of the review are:

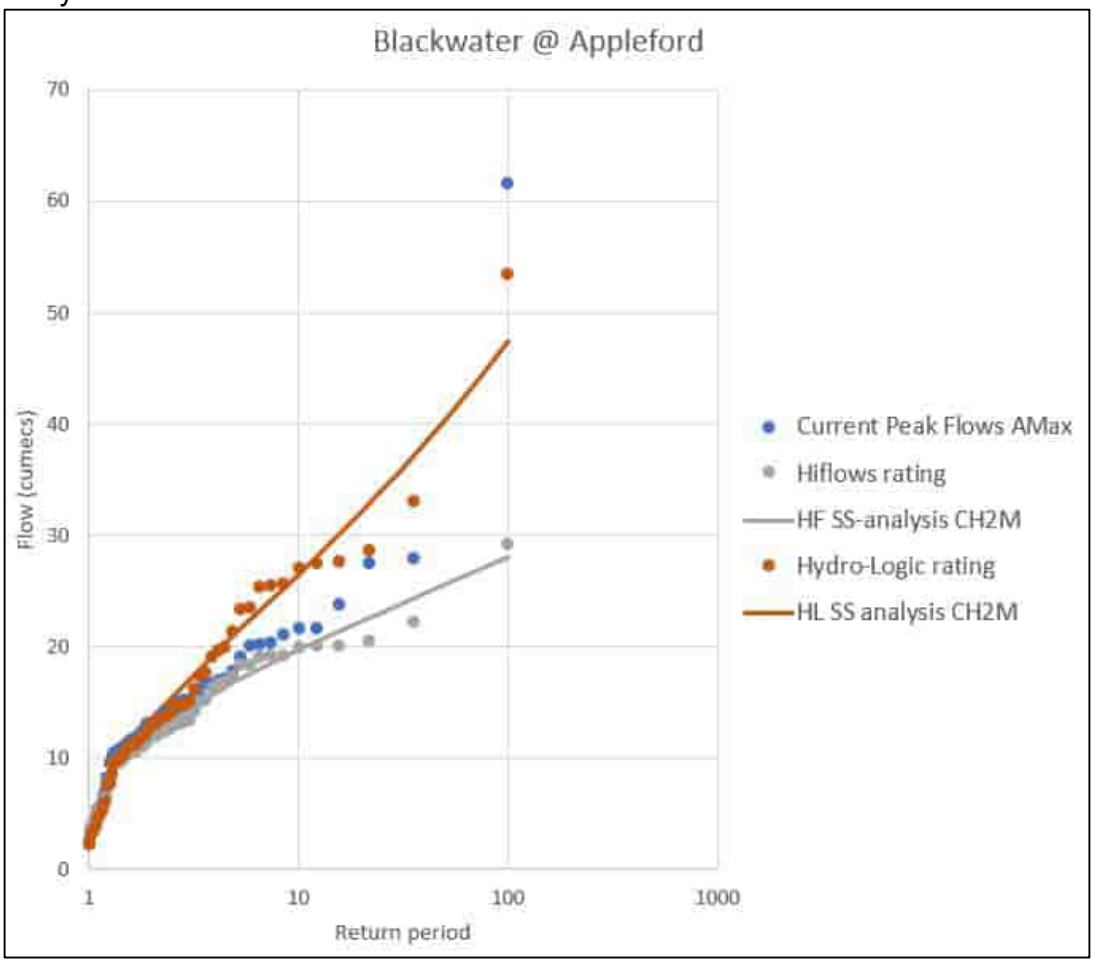
i) No new high flow spot gaugings have been taken since the CH2M (2017) hydrological analysis that can help refine the understanding of the high flows performance of either the Blackwater @ Appleford or the Brain @ Guithavon gauges.

ii) Six additional years of annual maximum data now exist for both gauges (not available to the CH2M (2017) study). It is recommended that these be included in any analysis updating the design flows. However, this by itself is thought unlikely to greatly change the resulting design flows since none of the recent flows are particularly large.

iii) The CH2M study used an annual maximum series for the Appleford gauge that differs to the most recent version of the annual maximum series that now appears on the Peak Flows database. In the CH2M study two versions of the Amax series were actually considered: called the “Hiflows” and the “Hydro-Logic” Amax series. The Hiflows series is understood to have been the favored series at the time that appeared in the Hiflows dataset which was the predecessor to the Peak Flows dataset. The Hydro-Logic series was one based on a revised provisional rating developed by Hydro-Logic following their rating

review (Hydrologic, 2014)[4]. Plate A.1 compares the Gringorten plotting positions of these Amax series, together with the single site statistical analysis undertaken by CH2M and reported in 2017.

Plate A.1 - Comparison of the Gringorten plotting positions of the current Peak Flows Amax series to those of the Amax series considered by CH2M in the 2017 study.



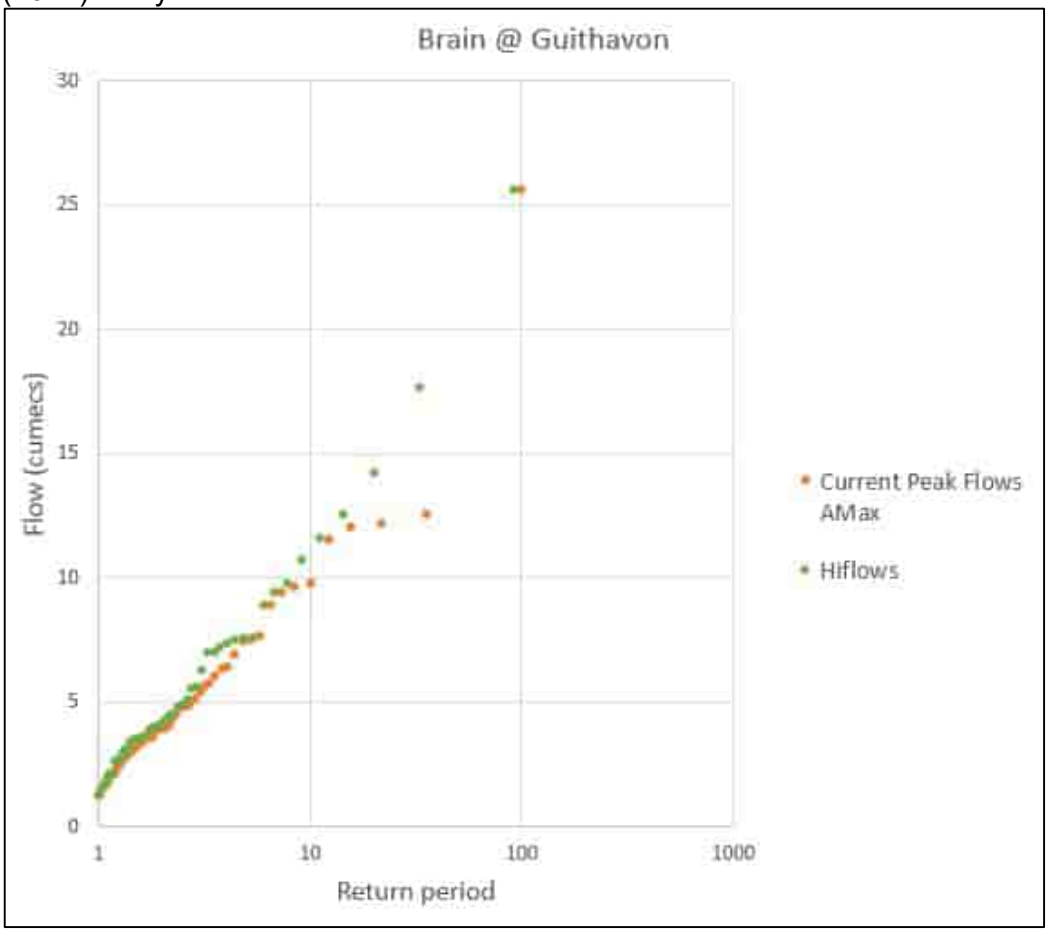
The Hydro-Logic rating review suggested that the gauge should provide reasonable estimates of Qmed, and that crest-tapping measurements were used to adjust for non-modular flow conditions, plus that an additional overflow spillway contribution should also be included at the highest of flows. However Hydro-Logic were at the time unable to review these aspects as the background information was unavailable. At the time they suggested a provisional revision to the general rating though this was not particularly convincing at higher flows when compared to the available spot gaugings. They recommended further

[4] Hydro-Logic, 2014. Review of performance of gauging Station 2013-14 River Blackwater at Appleford Bridge. Report for Environment Agency, Anglian Region.

spot gauging of high flows to help refine the understanding. However, the CH2M study rejected the Hydro-Logic rating as it resulted in a flood growth curve that they considered to be excessively steep (refer to Plate A.1).

iv) The Guitavon Amax values that now appear in the Peak Flows database also differ to those that were available on the Hiflows database at the time of the CH2M (2017) study (Plate A.2). The full period of record Amax series has recently been reviewed by the EA (released September 2019). The difference is in part thought to be based on the use of crest tappings to improve the estimate when the flow becomes non-modular. These amendments seem not to have been applied in the earlier Hiflows series.

Plate A.2 - Comparison of the Gringorten plotting positions of the current Peak Flows Amax series to those of the Hiflows Amax series available to the CH2M (2017) study.



The Hydro-Logic (2014) rating review[5] concluded that there were “insufficient high flow gaugings with which to draw any meaningful conclusions about the

[5] Review of the performance of gauging stations 2013-14 Guithavon Valley. Report for Environment Agency, Anglian Region

high flow rating i.e. greater than stage of 1 m". However, there are a reasonable number of spot gaugings for smaller floods and the gauge is therefore likely to provide a reasonable estimate of Qmed.

In the CH2M study it was noted that there was uncertainty about the adequacy of the station's rating curve (and hence the Amax series). They derived an alternative rating from their hydraulic model. They did however note: "the hydraulic model is a broadscale catchment model and therefore not ideally suited to deriving detailed rating curves". This lowered the highest Amax values by up to 15% whilst leaving the Qmed little changed and was preferred in the study to the Hiflows Amax series.

It is beyond the resources of the current project to undertake a further detailed gauge review. However, the indications are that the gauge will supply a reasonable estimate of Qmed. There was uncertainty at the time of the CH2M analysis as to what represented the most accurate Amax series (particularly for the higher flows). Since then the Amax series appearing in Peak Flows has been revised by the EA which makes allowances for non-modular flow. It is therefore recommended that this Peak Flows Amax series be used in preference to that relied upon in the CH2M study. However, we consider that residual uncertainty remains, and recommend that the analysis needs to be wary of placing too much reliance on the accuracy of the gauge's higher flows.

v) In the CH2M (2017) study the design peak flows for the Blackwater at Appleford gauge were based upon Enhanced Single Site (ESS) analysis (using the Hiflows rating) augmented above the 1% AEP by application of ReFH ratios.

Based on ii and iii above this review recommends that the estimate of design peak flows at the Blackwater at Appleford gauge be estimated by both up-dated and revised ESS and pooling group in which the Qmed is taken directly from the gauge. Given there remains a degree of uncertainty concerning the higher Amax values it is considered prudent to take the higher of the two sets of estimates to be the design flows. Extension of the flood frequency curve above the 1% AEP using the ratios of ReFH is considered acceptable. We will in addition to the statistical estimates also provide single catchment estimates from the latest ReFH version (2.3) for comparison.

vi) The CH2M (2017) flood growth curve used for the River Brain was remarkably steep. For example: the1% AEP growth factor was marginally over 5 (yet the normal range tends to be between 2 and 3.5). This is an exceptionally steep relationship and one that merits further consideration. Based on ii and iv above this review recommends that the design peak flows at the Brain at Guithavon station be recalculated using the up-dated and revised annual maximum series via both ESS and pooling group analysis in which the Qmed is taken directly from the gauge. Given there remains a degree of uncertainty concerning the higher Amax values it is considered prudent to take the higher of the two sets of estimates to be the design flows. Extension of the flood frequency curve above the 1% AEP using the ratios of ReFH is considered acceptable. We will

in addition to the statistical estimates also provide single catchment estimates from the latest ReFH version (2.3) for comparison.

Proposed analysis\approach:

Based upon the above key findings the following steps are proposed for revising the CH2M (2017) design flood hydrology at those locations where the A12 crosses or potentially lies on the flood plain of the rivers Blackwater and Brain.

Step 1: Revise the statistical estimates of the peak flows.

Following the recommendations given in v and vi above the design flows at the two gauges will be re-estimated. Design flows for target locations upstream or downstream of the gauges will be calculated using standard donor principals.

Step 2: Compare the runoff rates per unit area.

The resulting runoff rates (l/s/ha) over a range of return periods will be compared to each other and to those of adjacent similar catchments also subject to analysis on the A12 project. Any differences will be interpreted.

Step 3: Inflows to the hydraulic flood model.

The ReFH1 IEDs from the 2017 study will be retained in the flood model and will be the means of supplying the necessary inflow design hydrographs, with the exception that new ReFH1 IEDs will be created at the top of the Jacobs model extent to represent as a single catchment the upstream area. (Since the full extent of the River Blackwater and River Brain models do not require to be run for the specific locations of interest to the A12 Road Widening assessment).

Step 4: Design event runs

Critical storm durations will be established from the ReFH default storm duration equation for the location of the target site. The same design storm will be applied to all the catchments modelled. The modelled flows will be reconciled at the target locations of interest to the preferred statistical flows established in Step 1. This will be done by uniformly scaling the inflows such that the routed flow at the target location equals the design flow obtained in Step 1. The design flows and hence storm on the Brain above where the two rivers interact will be the subject of a separate storm event appraisal and flood model analysis.

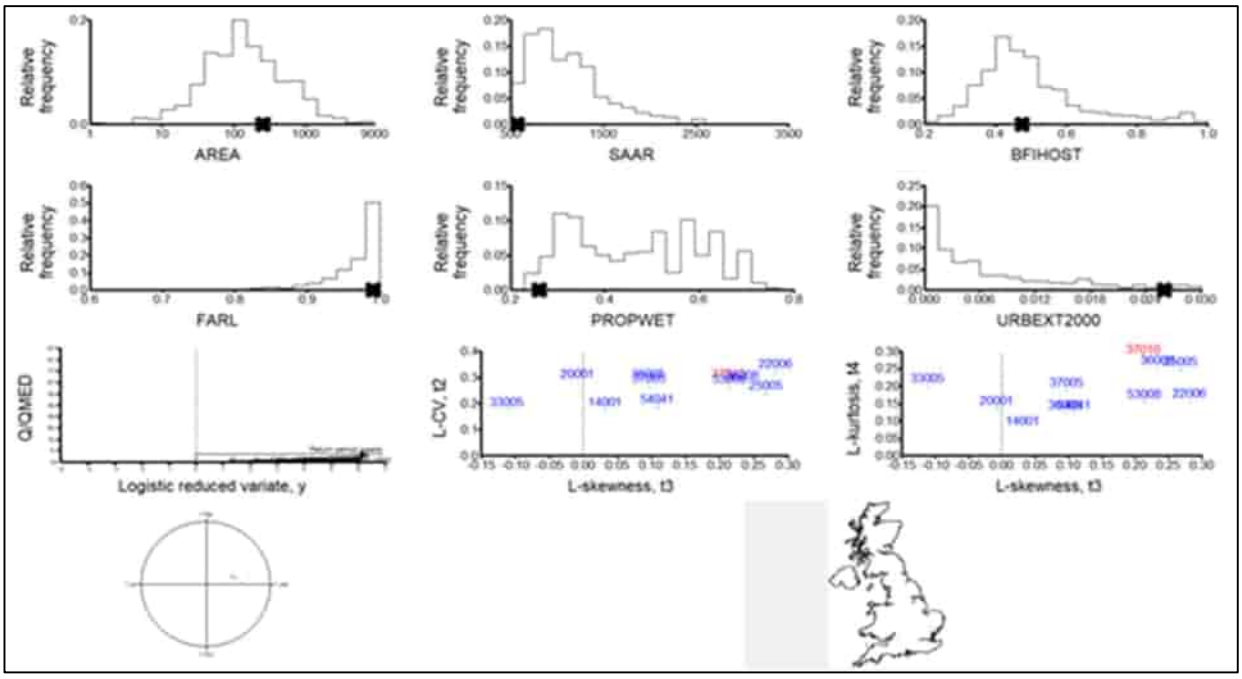
Step 5: Validation of the modelled design floods

Design levels and flood extents will be compared to observed flood levels and extents where readily available as a check upon the acceptability of the design runs.

Requested action:

The Environment Agency is requested to confirm they agree with the proposed hydrological approach for the River Blackwater and River Brain for the A12 Road Widening Scheme.

A.3 Pooling Group Details – Graphs – River Blackwater @ Appleford



A.4 Pooling Group Details – Table- River Blackwater @ Appleford

A.4.1 Initial Pooling Group Details: (show default selection) at River Blackwater @ Appleford

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37010 (Blackwater @ Appleford Bridge)	0	56	13.15	0.285	0.213	1.138	0	249.12	572	0.098	0.991	0.026
37005 (Colne @ Lexden)	0.287	58	12.4	0.268	0.097	0.351	0.287	235.8	566	0.076	0.97	0.022
36008 (Stour @ Westmill)	0.368	58	17.395	0.28	0.233	0.647	0.368	222.8	589	0.068	0.994	0.023
22006 (Blyth @ Hartford Bridge)	0.441	58	52.591	0.322	0.283	0.659	0.441	273.67	696	0.115	0.99	0.009
30003 (Bain @ Fulsby Lock)	0.499	56	16.198	0.287	0.083	0.992	0.499	199.48	667	0.081	0.963	0.009
25005 (Leven @ Leven Bridge)	0.583	48	43.54	0.241	0.269	1.318	0.583	194.15	726	0.107	0.994	0.014
54041 (Tern @ Eaton Upon Tern)	0.656	42	12.444	0.187	0.109	1.203	0.656	193.49	719	0.119	0.954	0.015
33005 (Bedford Ouse @ Thornborough Mill)	0.688	28	21.8	0.178	-0.112	2.253	0.688	387.67	655	0.111	0.983	0.014
36005 (Brett @ Hadleigh)	0.698	56	11	0.284	0.095	0.581	0.698	155.8	580	0.076	0.994	0.009
43005 (Avon @	0.698	29	10.264	0.322	0.255	1.119	0.698	326.4	744	0.071	1	0.013
14001 (Eden @ Kemback)	0.711	39	40.417	0.176	0.032	1.513	0.711	308.7	800	0.104	0.992	0.011

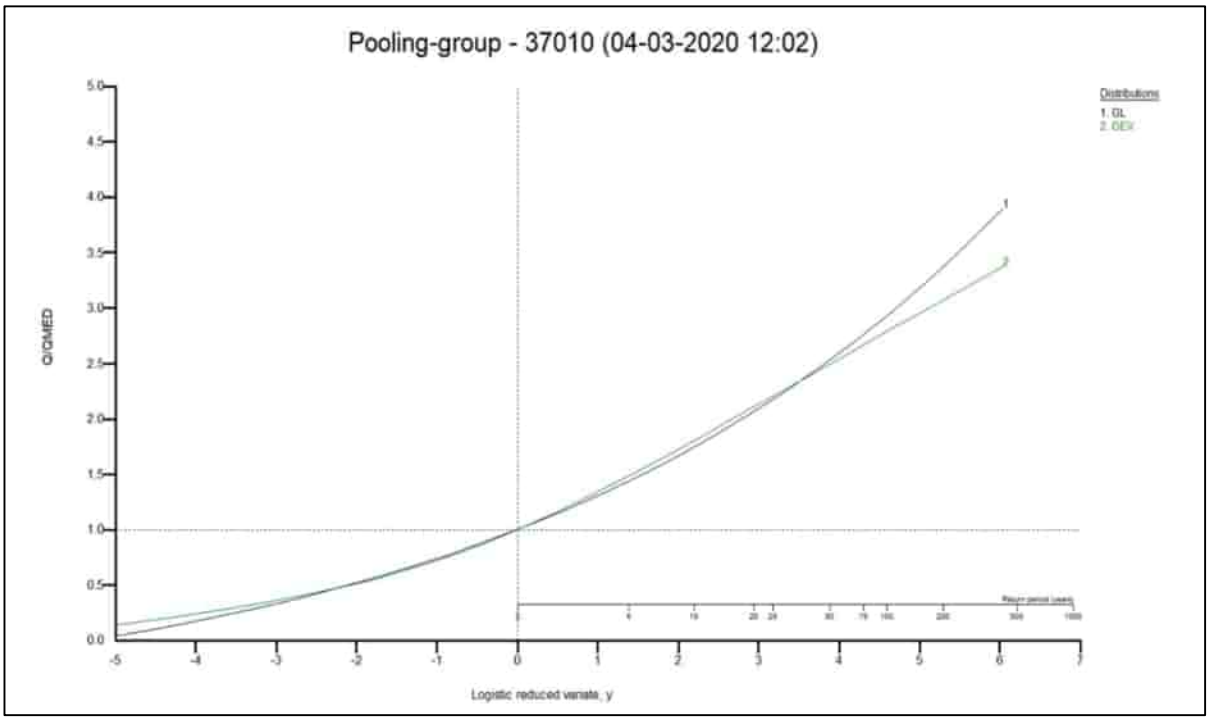
53008 (Avon @ Great Somerford)	0.712	55	37.132	0.264	0.214	0.227	0.712	305.17	804	0.093	0.988	0.009
Total		583										
Weighted means				0.279	0.167							

A.4.2 Final Pooling Group Details at River Blackwater @ Appleford

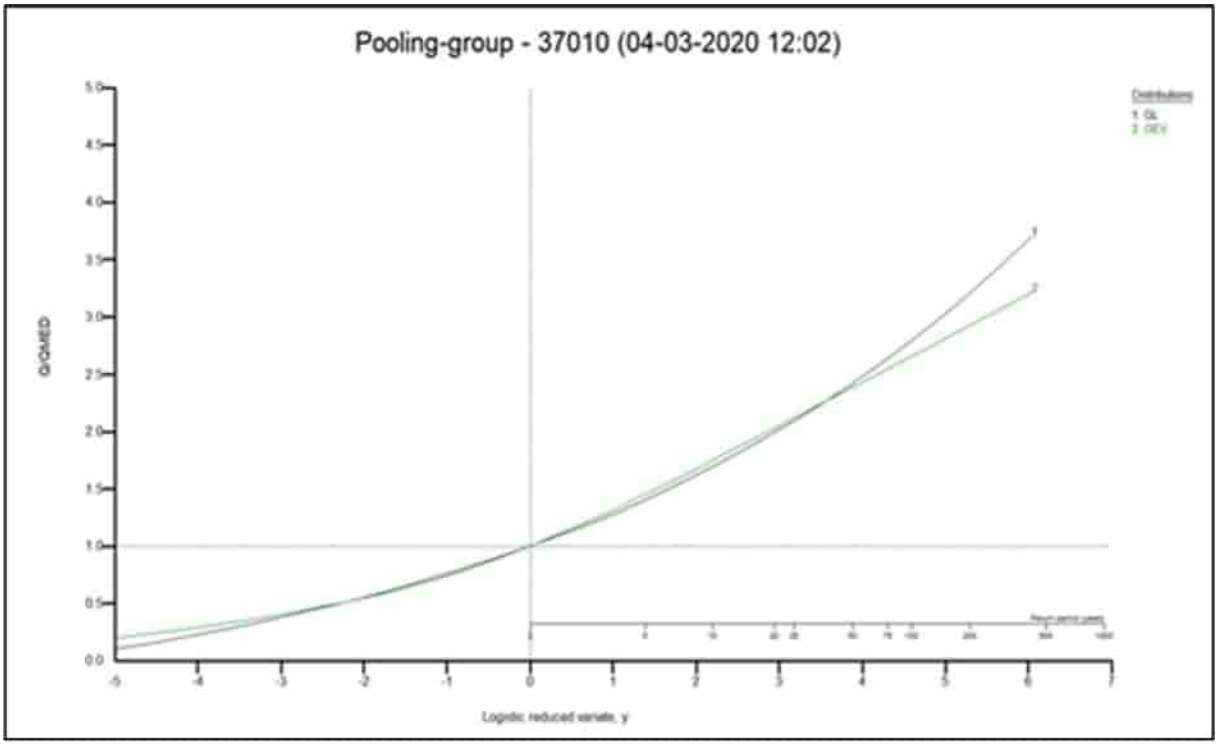
Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37010 (Blackwater @ Appleford Bridge)	0	56	13.15	0.285	0.213	1.046	0	249.12	572	0.098	0.991	0.026
37005 (Colne @ Lexden)	0.287	58	12.4	0.268	0.097	0.17	0.287	235.88	566	0.076	0.97	0.022
36008 (Stour @ Westmill)	0.368	58	17.395	0.28	0.233	0.577	0.368	222.82	589	0.068	0.994	0.023
22006 (Blyth @ Hartford Bridge)	0.441	58	52.591	0.322	0.283	1.176	0.441	273.67	696	0.115	0.99	0.009
25005 (Leven @ Leven Bridge)	0.583	48	43.54	0.241	0.269	1.09	0.583	194.15	726	0.107	0.994	0.014
54041 (Tern @ Eaton Upon Tern)	0.656	42	12.444	0.187	0.109	1.01	0.656	193.49	719	0.119	0.954	0.015
33005 (Bedford Ouse @ Thornborough Mill)	0.688	28	21.8	0.178	-0.112	2.087	0.688	387.67	655	0.111	0.983	0.014
36005 (Brett @ Hadleigh)	0.698	56	11	0.284	0.095	0.681	0.698	155.85	580	0.076	0.994	0.009
14001 (Eden @ Kemback)	0.711	39	40.417	0.176	0.032	1.428	0.711	308.74	800	0.104	0.992	0.011
53008 (Avon @ Great Somerford)	0.712	55	37.132	0.264	0.214	0.348	0.712	305.17	804	0.093	0.988	0.009
20001 (Tyne @ East Linton)	0.738	22	51.91	0.284	-0.008	1.388	0.738	307.26	713	0.05	0.987	0.007
Total		520										
Weighted means				0.277	0.159							

A.5 River Blackwater Growth Curves

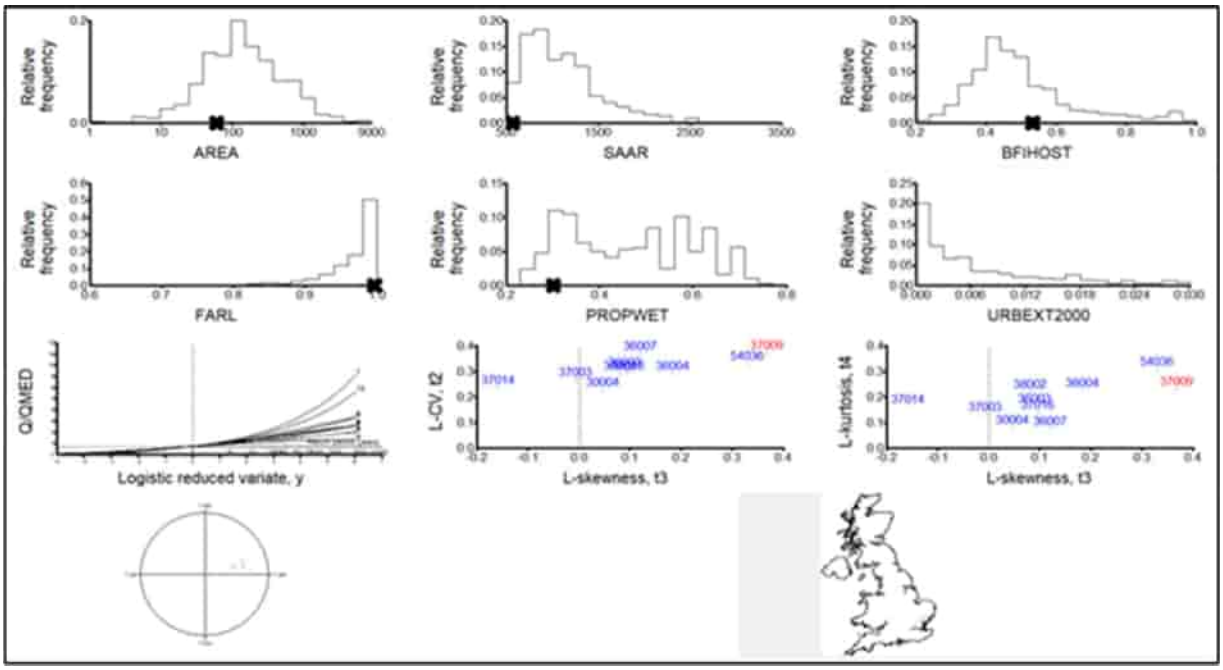
A.5.1 River Blackwater @ Appleford Bridge ESS Analysis



A.5.2 River Blackwater @ Appleford Bridge Pooling Analysis



A.6 Pooling Group Details – Graphs – River Brain @ Guithavon



A.7 Pooling Group Details – Table- River Brain @ Guithavon

A.7.1 Initial Pooling Group Details: (show default selection) at River Brain @ Guithavon

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
36007 (Belchamp Brook @ Bardfield Bridge)	0.085	53	4.63	0.371	0.119	2.12	58.16	560	0.079	0.996	0.004
36003 (Box @ Polstead)	0.141	57	3.91	0.305	0.089	0.172	56.72	566	0.093	0.993	0.012
37016 (Pant @ Copford Hall)	0.19	54	7.24	0.293	0.096	0.053	63.8	588	0.069	0.997	0.009
36004 (Chad Brook @ Long Melford)	0.334	51	5.186	0.294	0.182	1.811	50.33	589	0.065	1	0.006
26013 (Driffield Trout Stream @ Driffield)	0.411	8	2.78	0.29	0.218	2.603	53.33	690	0.093	0.997	0.006
30004 (Lymn @ Partney Mill)	0.45	56	6.983	0.229	0.046	0.953	60.09	686	0.06	0.979	0.006
26003 (Foston Beck @ Foston Mill)	0.46	57	1.76	0.248	-0.009	1.029	59.59	698	0.106	0.987	0.004
37003 (Ter @ Crabbs Bridge)	0.5	53	5.43	0.269	-0.007	0.739	77.76	570	0.115	0.994	0.012
33032 (Heacham @ Heacham)	0.52	50	0.442	0.304	0.124	0.084	56.16	688	0.116	0.983	0.006
38002 (Ash @ Mardock)	0.553	77	6.76	0.292	0.08	0.434	78	619	0.049	1	0.014
Total		516									
Weighted means		401		0.295	0.09						

*River Brain @ Guithavon not included in initial pooling group until ESS analysis selected as moderately urbanised catchment.

A.7.2 Final Pooling Group Details at River Brain @ Guithavon – ESS Analysis

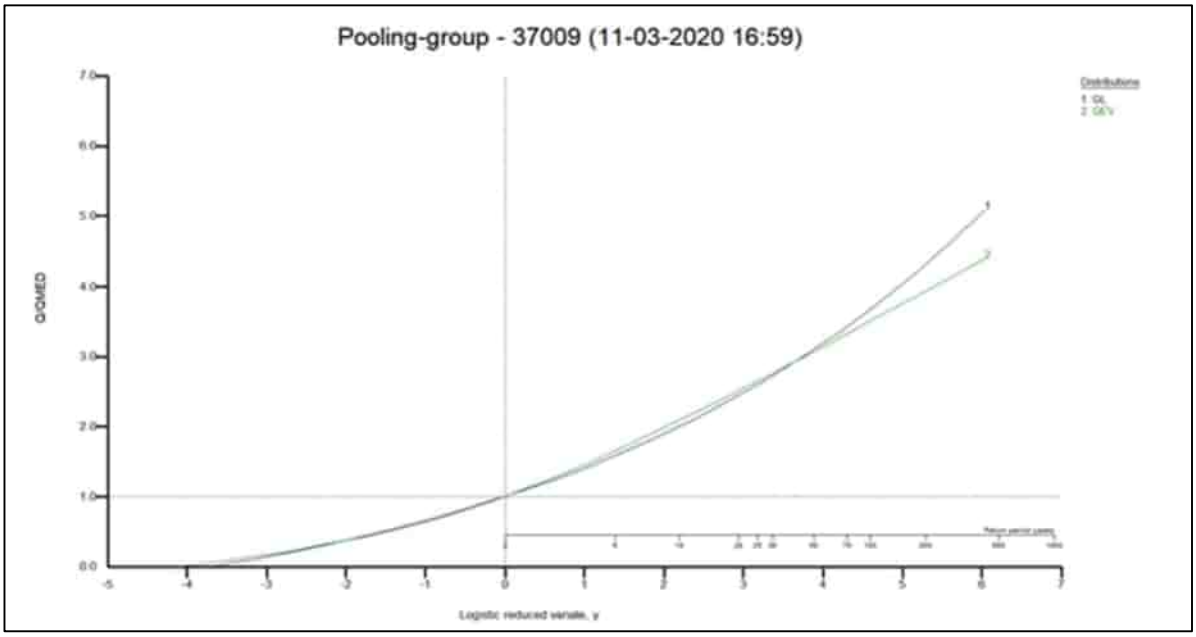
Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37009 (Brain @ Guithavon Valley)	0	56	3.895	0.389	0.354	1.082	60.27	572	0.084	0.997	0.074
36007 (Belchamp Brook @ Bardfield Bridge)	0.085	53	4.63	0.371	0.119	2.26	58.16	560	0.079	0.996	0.004
36003 (Box @ Polstead)	0.141	57	3.91	0.305	0.089	0.059	56.72	566	0.093	0.993	0.012
37016 (Pant @ Copford Hall)	0.19	54	7.24	0.293	0.096	0.114	63.8	588	0.069	0.997	0.009
36004 (Chad Brook @ Long Melford)	0.334	51	5.186	0.294	0.182	0.298	50.33	589	0.065	1	0.006
30004 (Lymn @ Partney Mill)	0.45	56	6.983	0.229	0.046	2.435	60.09	686	0.06	0.979	0.006
37003 (Ter @ Crabbs Bridge)	0.5	53	5.43	0.269	-0.007	0.244	77.76	570	0.115	0.994	0.012
38002 (Ash @ Mardock)	0.553	77	6.76	0.292	0.08	0.413	78	619	0.049	1	0.014
37014 (Roding @ High Ongar)	0.663	54	10.928	0.239	-0.159	1.747	92.65	597	0.107	0.986	0.008
54036 (Isbourne @ Hinton on the Green)	0.737	46	13.578	0.331	0.329	1.348	92.83	701	0.07	0.99	0.012
Total		557									
Weighted means				0.368	0.17						

A.7.3 Final Pooling Group Details at River Brain @ Guithavon – Pooling Analysis

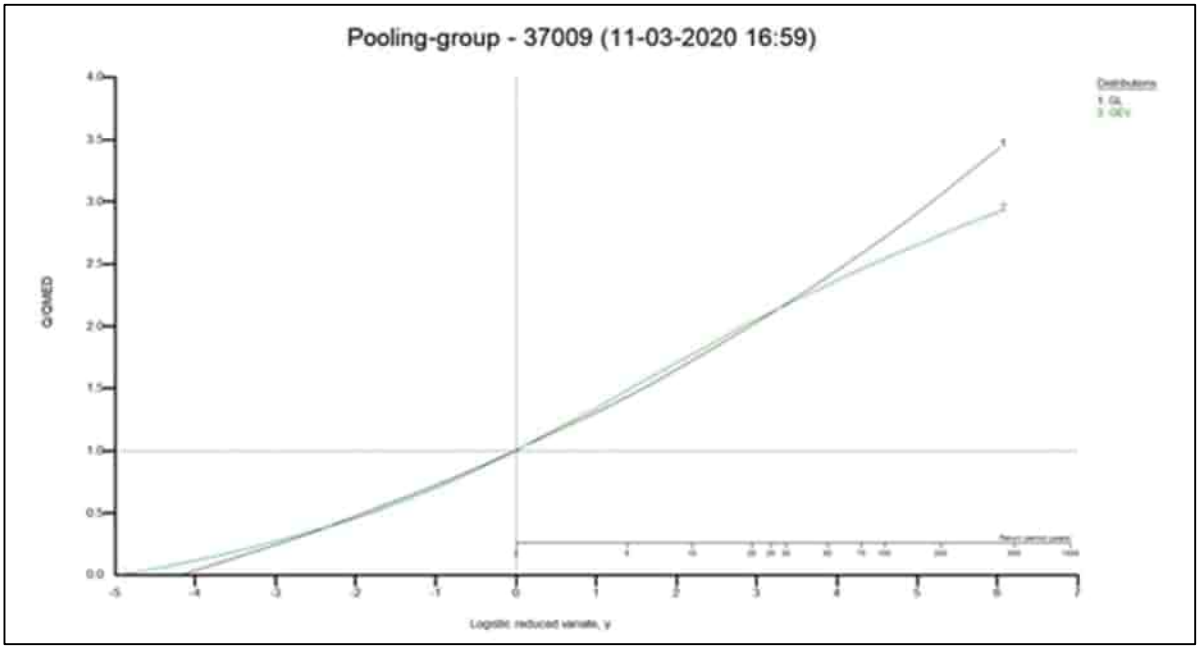
Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
36007 (Belchamp Brook @ Bardfield Bridge)	0.085	53	4.63	0.371	0.119	2.261	58.16	560	0.079	0.996	0.004
36003 (Box @ Polstead)	0.141	57	3.91	0.305	0.089	0.056	56.72	566	0.093	0.993	0.012
37016 (Pant @ Copford Hall)	0.19	54	7.24	0.293	0.096	0.107	63.8	588	0.069	0.997	0.009
36004 (Chad Brook @ Long Melford)	0.334	51	5.186	0.294	0.182	0.344	50.33	589	0.065	1	0.006
30004 (Lymn @ Partney Mill)	0.45	56	6.983	0.229	0.046	2.16	60.09	686	0.06	0.979	0.006
37003 (Ter @ Crabbs Bridge)	0.5	53	5.43	0.269	-0.007	0.196	77.76	570	0.115	0.994	0.012
38002 (Ash @ Mardock)	0.553	77	6.76	0.292	0.08	0.346	78	619	0.049	1	0.014
37014 (Roding @ High Ongar)	0.663	54	10.928	0.239	-0.159	1.857	92.65	597	0.107	0.986	0.008
54036 (Isbourne @ Hinton on the Green)	0.737	46	13.578	0.331	0.329	1.673	92.83	701	0.07	0.99	0.012
Total		501									
Weighted means				0.297	0.09						
Weighted means				0.279	0.051						

A.8 River Brain Growth Curves

A.8.1 River Brain @ Guithavon ESS Analysis

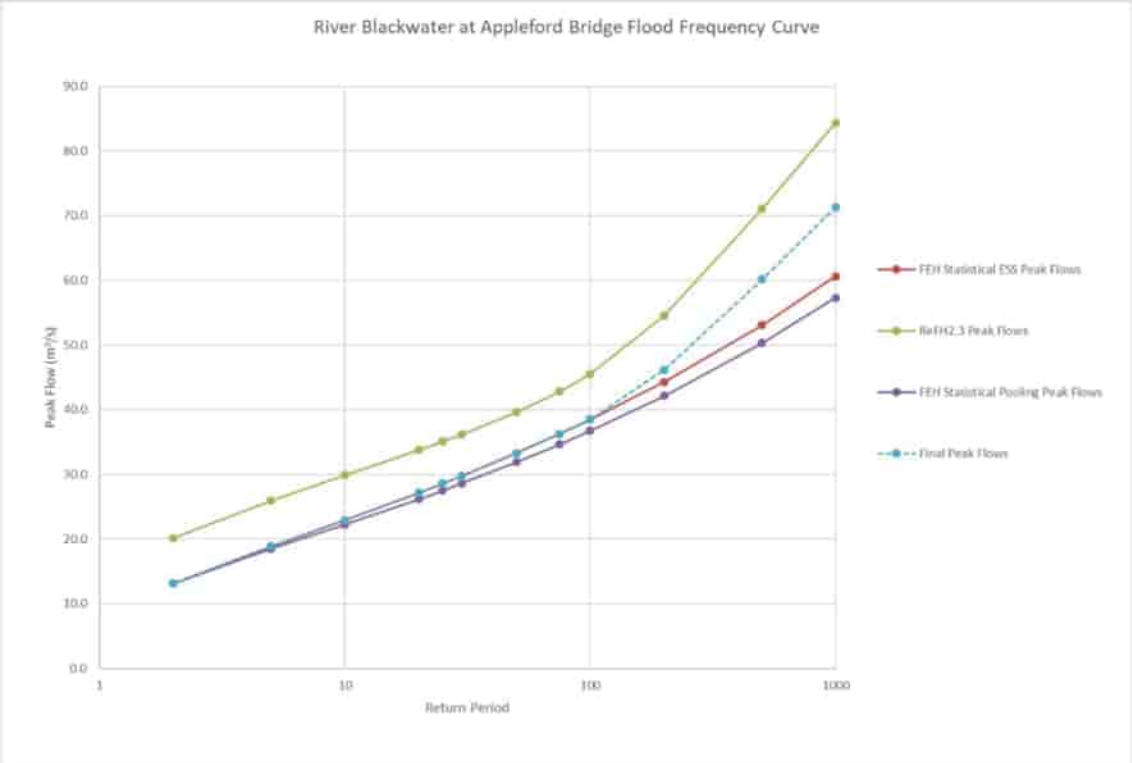


A.8.2 River Brain @ Guithavon Pooling Analysis

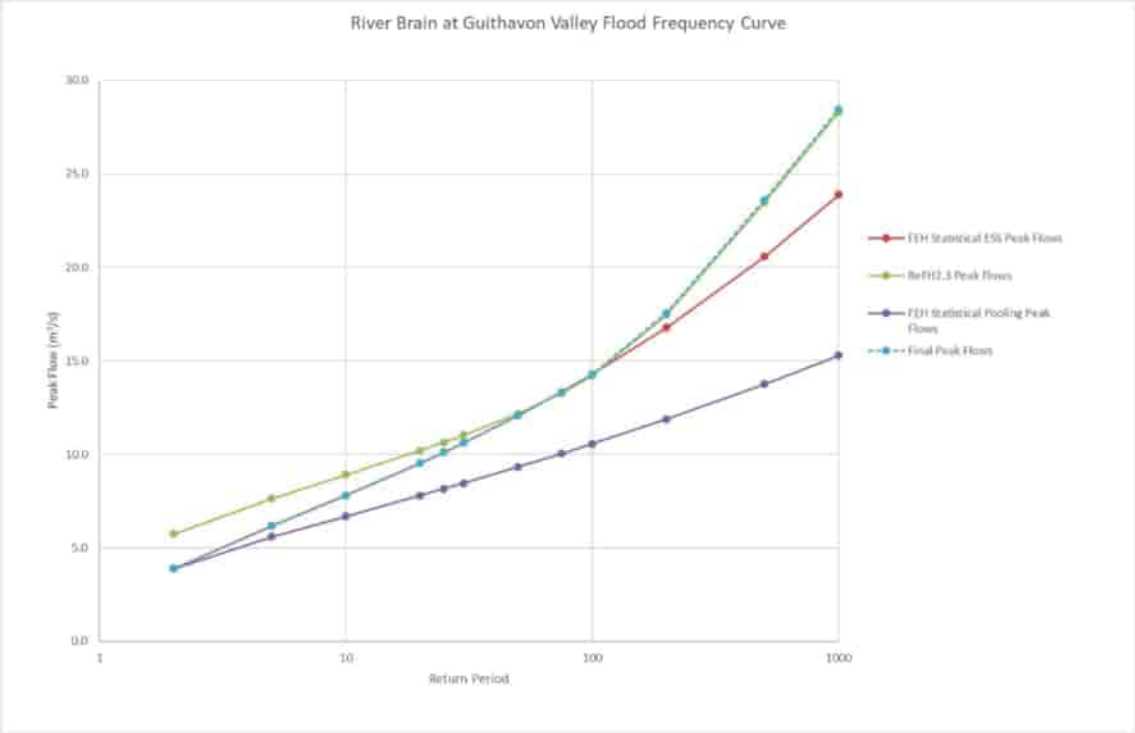


A.9 Flood Frequency Curve Comparison

A.9.1 River Blackwater at Appleford Bridge

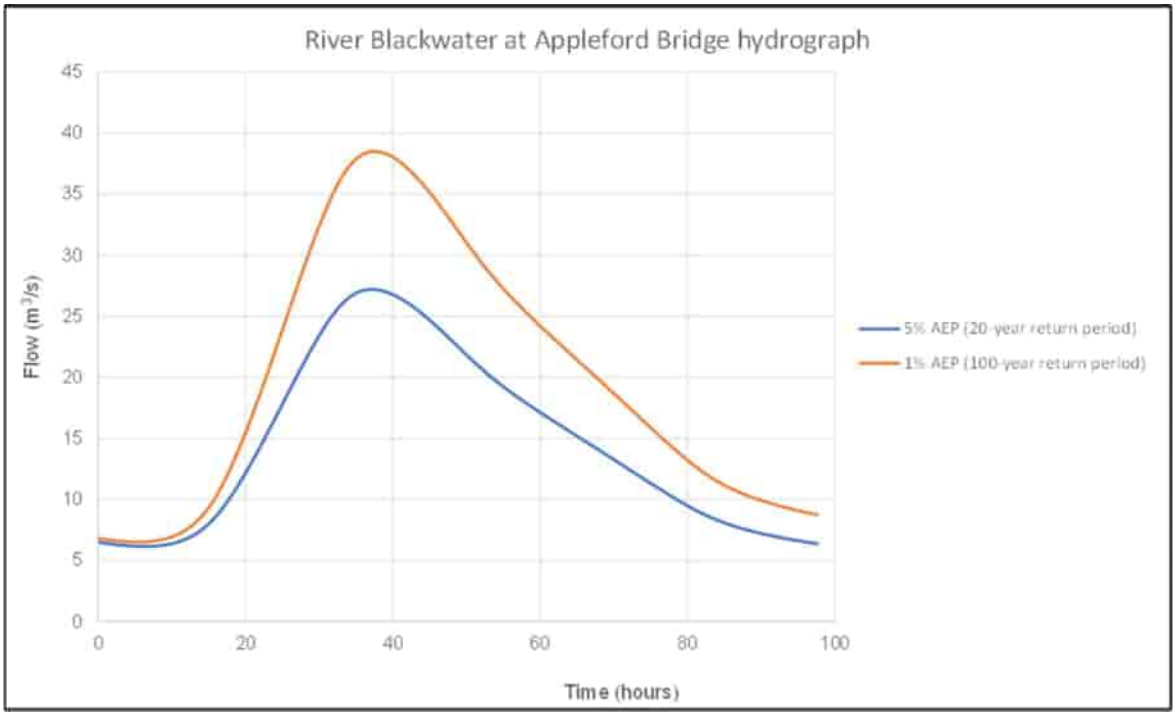


A.9.2 River Brain at Guithavon Valley



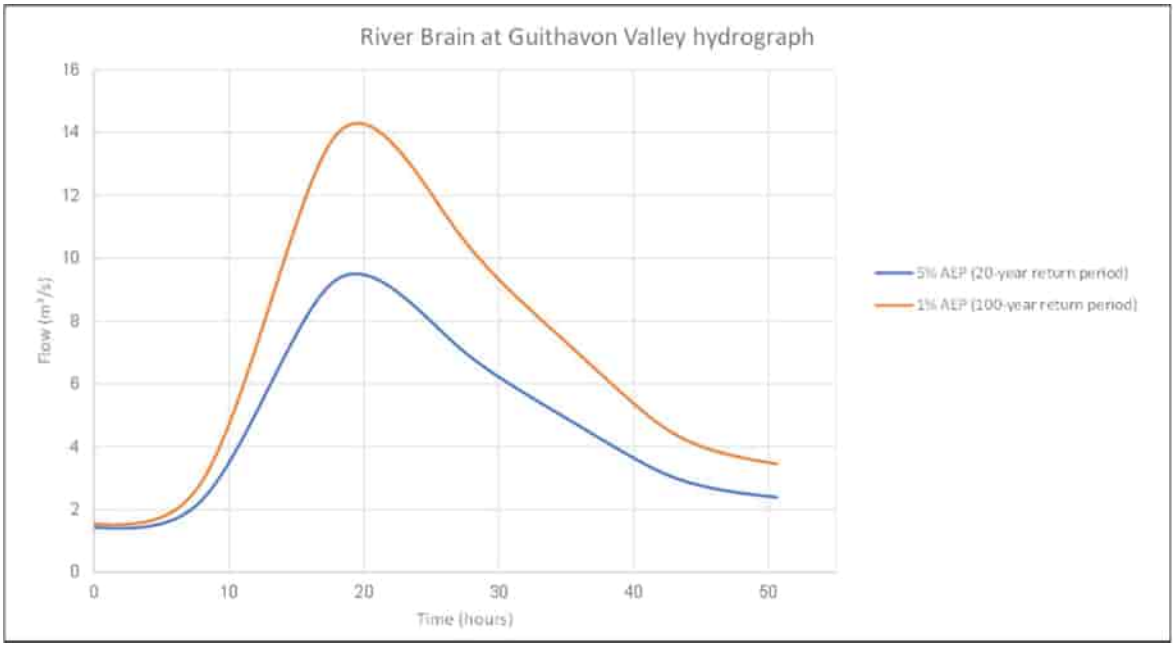
A.10 1% AEP (100-year return period) Hydrographs

A.10.1 River Blackwater @ Appleford Bridge 5% and 1% AEP (20-year and 100-year return period respectively) Hydrographs



ReFH1 hydrograph scalled to the final peak flow estimate. Duration 29.7 hours

A.10.2 River Brain @ Guithavon Valley 5% and 1% AEP (20-year and 100-year return period respectively) Hydrographs



ReFH1 hydrograph scalled to the final peak flow estimate. Duration 15.9 hours.

A.11 ReFH2.3 Hydrograph Shape Sensitivity Analysis

A.11.1 A sensitivity analysis was undertaken for the Middle Blackwater model for the 1% AEP (100-year return period) event to assess whether there is an impact on flood extent and depths by adoption the ReFH2.3 hydrograph shape opposed to the originally adopted ReFH1 hydrograph shape. For this purpose, the Middle Blackwater model ReFH1 IEDs for the baseline scenario were replaced with ReFH2.3 flows for the 1% AEP (100-year return period) event. The modelled flow using ReFH2.3 inflows were reconciled with the final peak flow estimate at the River Blackwater at Appleford Bridge (37010) gauging station, similar to that was undertaken for the ReFH1 modelled flows. The results from this sensitivity analysis indicated that the ReFH1 hydrograph shape produced more conservative flood extents (red outline in figure below) within the vicinity of the A12 crossing of the River Blackwater than the corresponding ReFH2.3 inflows scaled to the target peak flow estimate (blue shading in figure).

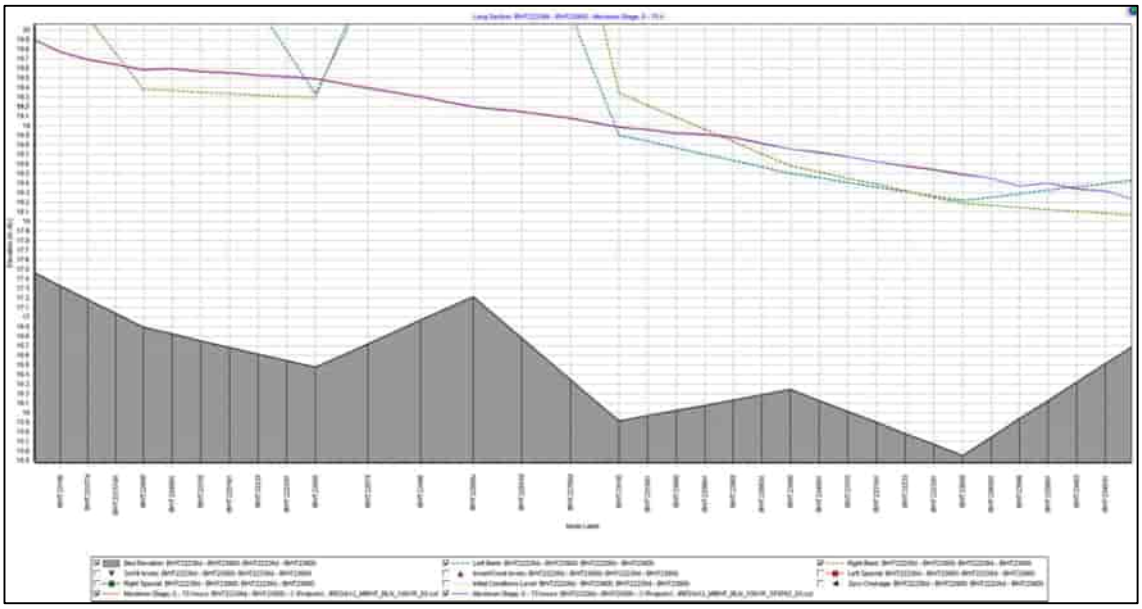
Figure showing 1% AEP (100-year return period) flood extents at the A12 crossing of the River Blackwater for the Baseline Scenario (ReFH1 – red - scaling factor 0.7, ReFH2.3 – blue - scaling factor 0.82)



A.11.2 The water level in the River Blackwater in the vicinity of the A12 crossing was also assessed and estimated to be marginally higher when the ReFH1

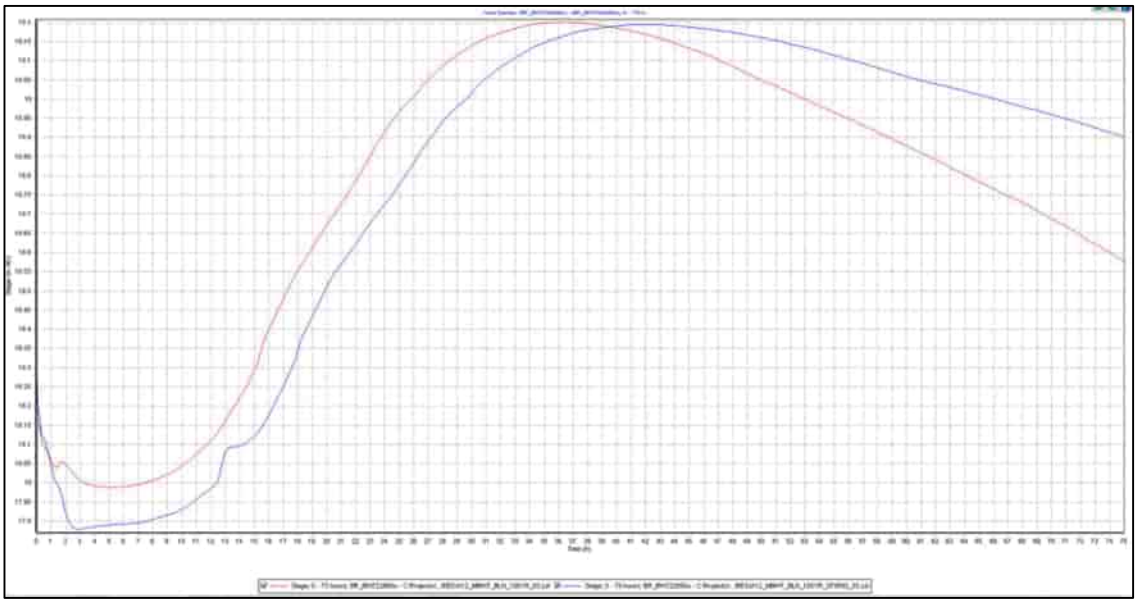
hydrograph shape (red) is adopted opposed to the ReFH2.3 hydrograph shape (blue).

Longprofile in vicinity of A12 for baseline scenario (ReFH1 – red – scaling factor 0.7, ReFH2.3 – blue – scaling factor 0.82)



A.11.3 When comparing the hydrograph shapes produced using ReFH1 and ReFH2.3 inflows scaled to the final peak flow estimate at the A12 crossing (baseline scenario) the ReFH1 shape is relatively comparable with the ReFH2.3 shape, with peak from one method (ReFH1) occurring earlier than the other (ReFH2.3).

Stage at A12 for baseline scenario (ReFH1 – red – 0.7 scaling factor, ReFH2.3 – blue – 0.82 scaling factor)



A.11.4 In conclusion it has been assessed that the application of the ReFH1 hydrograph shape for model inflows results in slightly more conservative flood extents and depths within the vicinity of the proposed scheme for the tested baseline scenario and therefore the ReFH1 hydrograph shape has been accepted as appropriate to take forward for the Middle Blackwater and Lower Blackwater and Brain hydraulic models for the purposes of assessing any potential impacts of the A12 road improvements scheme.

A.12 Environment Agency Response to Consultation

Environment Agency

Mr. Michael Symons
Jacobs
1180 Eskdale Road
Winnersh
WOKINGHAM
RG41 5TU

Our ref: AE/2020/125129/01-L01
Your ref: B229H130-TRN-00590

Date: 11 June 2020

Dear Mr. Symons

A12 CHELMSFORD TO A120 WIDENING SCHEME - JUNCTION 23 (KELVEDON SOUTH) TO JUNCTION 25 (MARKS TEY INTERCHANGE).
RIVER BLACKWATER & RIVER BRAIN FLOOD MODEL HYDROLOGY REVIEW

Thank you for sending over the technical note regarding the review of the hydrology for the River Blackwater Middle hydraulic model and the River Blackwater Lower (including River Brain) hydraulic model (Doc No. HE551497-JAC-EWE-3_S0-TN-LE-0001).

Following a review of that note we would confirm that we are largely in agreement with the approaches proposed. Carrying out a revised ESS and Pooling group approaches seems appropriate given the gauging uncertainty, despite the updates to the Peak Flow data. It would be useful to have rating reviews carried out on the two gauges, but we accept that it's outside of the resources of the project. It would be useful perhaps to add a note on this in the reporting, so as to highlight this requirement in future studies.

The only further point we would like to raise relates to Step 3: Inflows to the hydraulic flood model. For the new IED files being generated, will there be any consideration of applying the ReFH2 methods? The National River Flow Archive states that the Appleford gauge (37010) is groundwater influenced, ReFH2 should offer a major improvement on these types of catchments – so ideally a comparison of the hydrographs should be carried out.

Yours sincerely

[redacted]

MR MARTIN BARRELL
Sustainable Places - Planning Specialist

[redacted]

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Weekday Daytime calls cost 8p plus up to 6p per minute from BT Weekend Unlimited. Mobile and other providers' charges may vary
Email: enquiries@environment-agency.gov.uk
Website: www.environment-agency.gov.uk

A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVENHALL BROOK MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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0 Executive Summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at Rivenhall Brook (NGR 584207 216765). The modelling and hydrology have been carried out in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1D only type using Flood Modeller software (Jacobs 2021) and was built from scratch.

0.1.2 Three events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) with 38% climate change. Flow reconciliation was carried out to match the design hydrology. No data was available for calibration, but the model was verified using Environment Agency (EA) flood maps. The model was then used to simulate baseline and with-scheme scenarios for the three events mentioned. Sensitivity analysis of roughness, inflows and the downstream boundary was carried out to further validate the results.

0.1.3 The with-scheme scenario involved the implementation of a new culvert and realignment of the existing channel to represent the crossing of Rivenhall Brook by the new offline road alignment.

0.1.4 The analysis demonstrates that the proposed-scheme results in a significant reduction (>0.10m) in flooding upstream of the existing A12. The channel realignment requires a raised bank-top (bund), on the right-hand side immediately downstream from the new A12 culvert outfall in order to contain the design flows.

0.1.5 There is no significant impact on flooding from Rivenhall Brook downstream of the proposed-scheme. Therefore, no flood risk mitigation is required.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening the A12 to three lanes throughout the length of the proposed-scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic models with associated catchment hydrology. Water levels for the baseline and with-scheme scenarios both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events at each watercourse crossing.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) were:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook (this report)
- Middle Blackwater
- Domsey Brook
- Roman River
- Five minor watercourses were also modelled for the proposed-scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for Rivenhall Brook, to assess the baseline and with-scheme scenarios. This is a technical report, focused on the hydraulic modelling and catchment hydrology, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 For the A12 crossing of Rivenhall Brook, the highway will be upgraded with the addition of a new offline road alignment situated approximately 120m to the south of existing carriageway. The new road will be embanked across the Rivenhall valley and will require a culvert structure

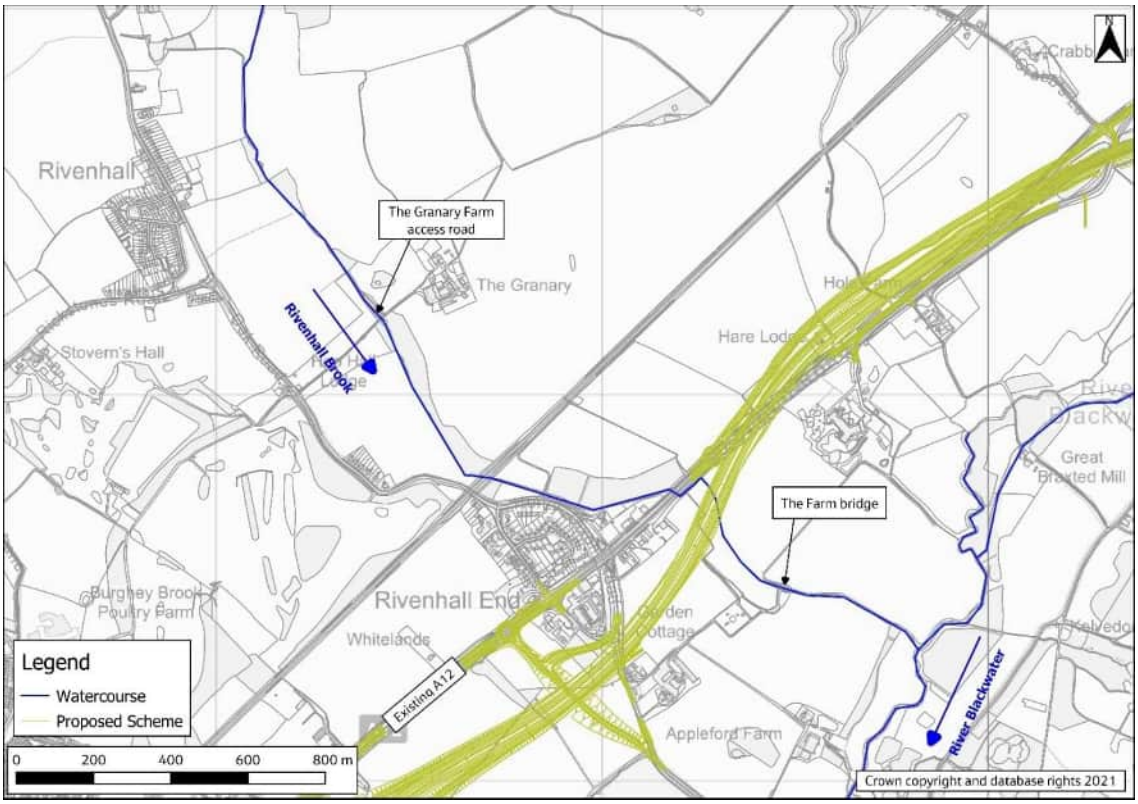
for the watercourse, plus minor re-alignment of the watercourse local to the new crossing. Full details of the with-scheme arrangements are given in Section 7.

1.1.6 This report should be read in conjunction with the FRA.

1.2 Study area

1.2.1 Rivenhall Brook flows to the north-east of Rivenhall End village in Essex, from north to south. It passes under the A12 to meet River Blackwater (as shown in Plate 1.1). The model domain includes a reach of 1.5km of Rivenhall Brook upstream of the A12, and 850m of downstream channel between the A12 and the River Blackwater.

Plate 1.1 Rivenhall Brook study area



2 Input data

2.1.1 The data used to construct the hydraulic model for Rivenhall Brook is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR DTM	2m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) data. It was used to extend cross sections into the floodplain.	Environment Agency, 2018
OS MasterMap	Ordnance Survey MasterMap data	National Highways, 2017
Channel survey	In-channel cross sections and hydraulic structures	Jacobs, January 2020

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the hydraulic model of the Rivenhall Brook for the 5% (1 in 20) and 1% (1 in 100) Annual Exceedance Probability (AEP) events. The 1% (1 in 100) AEP event is also required including an allowance for climate change (plus CC) (Environment Agency 2021). For full details of the flow derivation methods and background hydrology refer to the Rivenhall Brook Flood Estimation Calculation Record (Appendix A).

3.2 Hydrological considerations

3.2.1 The Rivenhall Brook catchment is a lowland catchment in the south-east of England. It covers an area of 17.3km^2 to the downstream model extent at its confluence with the River Blackwater. The catchment is classified as essentially rural with small urban areas around Silver End and Rivenhall End. There are small offline lakes and an online reservoir south of Silver End at Park House; but the attenuation of river flows is expected to be low across the catchment.

3.2.2 The catchment lies on a bedrock of London Clay Formation - Clay, Silt Sand and Gravel. In the headwaters, soils are described as being slowly permeable, seasonally wet and slightly acid but with base-rich loamy and clayey soils. The majority of the catchment has loamy and clayey soils with impeded drainage. The baseflow index is 0.391 with a standard percentage runoff of 44.6%. Downstream of the existing A12, soils are freely draining slightly acid loamy soils. These do not indicate a highly permeable catchment. The catchment receives a low average annual rainfall of 565mm.

3.3 Peak flow derivation

3.3.1 The Rivenhall Brook catchment is classified as a small (<25km^2) ungauged catchment. For such catchments where uncertainties may be large it is considered sensible to compare peak flows derived from two independent methods. Therefore, both the Flood Estimation Handbook (FEH) Statistical and the Revitalised Flood Estimation Handbook 2.3 (ReFH2.3) methods were applied to estimate the design return period peak flows.

3.3.2 Inflow hydrographs were required as inputs to the hydraulic model at the upstream model extent and as 3 lateral inflows. The FEH statistical method is not recommended for lateral sub-catchments, but for the upstream catchment (RIV1), it was used to be able to compare flows from the two methods. An estimate was also made at the downstream model extent (target location) to provide comparisons of runoff from the entire catchment and to enable flow reconciliation analysis. The ReFH2.3 resulted in higher design peak flows for the Rivenhall Brook than the FEH

Statistical method at both the upstream and downstream extents of the model.

3.3.3 The peak flow estimates at RIV1 (at the upstream model extent) and at the downstream modelling extent (total catchment runoff to the confluence (TOT)) are presented in Table 3.1. Plate 3.1 shows these two peak flow estimation locations. The peak flows from the ReFH2.3 model show a greater estimated peak flow at the upstream model extent (RIV1) than at the downstream total catchment (TOT). This is considered to be due to the model being more sensitive to small changes in some catchment descriptors than others as well as the downstream catchment having more permeability (higher value of the BFIHOST Baseflow Index) in comparison to the upper catchment.

Table 3.1 Rivenhall Brook peak flows as a function of event rarity

Inflow	Peak Flows (m^3/s)										
	AEP 50%	AEP 20%	AEP 10%	AEP 5%	AEP 3.33%	AEP 2%	AEP 1.33%	AEP 1%	AEP 1% + 38% CC uplift	AEP 0.5%	AEP 0.1%
	1 in 2	1 in 5	1 in 10	1 in 20	1 in 30	1 in 50	1 in 75	1 in 100	1 in 100+CC	1 in 500	1 in 1000
Statistical method											
RIV1	1.83	2.63	3.19	3.79	4.16	4.66	5.09	5.41	7.47	6.25	10.66
TOT	1.89	2.72	3.31	3.92	4.31	4.83	5.27	5.60	7.73	6.47	11.03
ReFH2.3											
RIV1	2.86	3.79	4.40	5.03	5.42	5.97	6.49	6.93	9.56	8.47	13.69
TOT*	2.69	3.54	4.11	4.69	5.05	5.56	6.04	6.45	8.90	7.87	12.69

*The AEP flows at TOT are for the full catchment runoff at the target location which are used to reconcile the modelled flow for the corresponding return period at this location.

3.4 Inflow hydrograph shape

3.4.1 Given the Rivenhall Brook catchment is ungauged it was assessed as appropriate to derive hydrograph shapes for input into the hydraulic model using ReFH2.3. The ReFH2.3 hydrograph shapes were derived for all sub-catchments and are all based on an 18-hour winter storm duration, which is equivalent to the design storm duration at the downstream modelling extent (TOT).

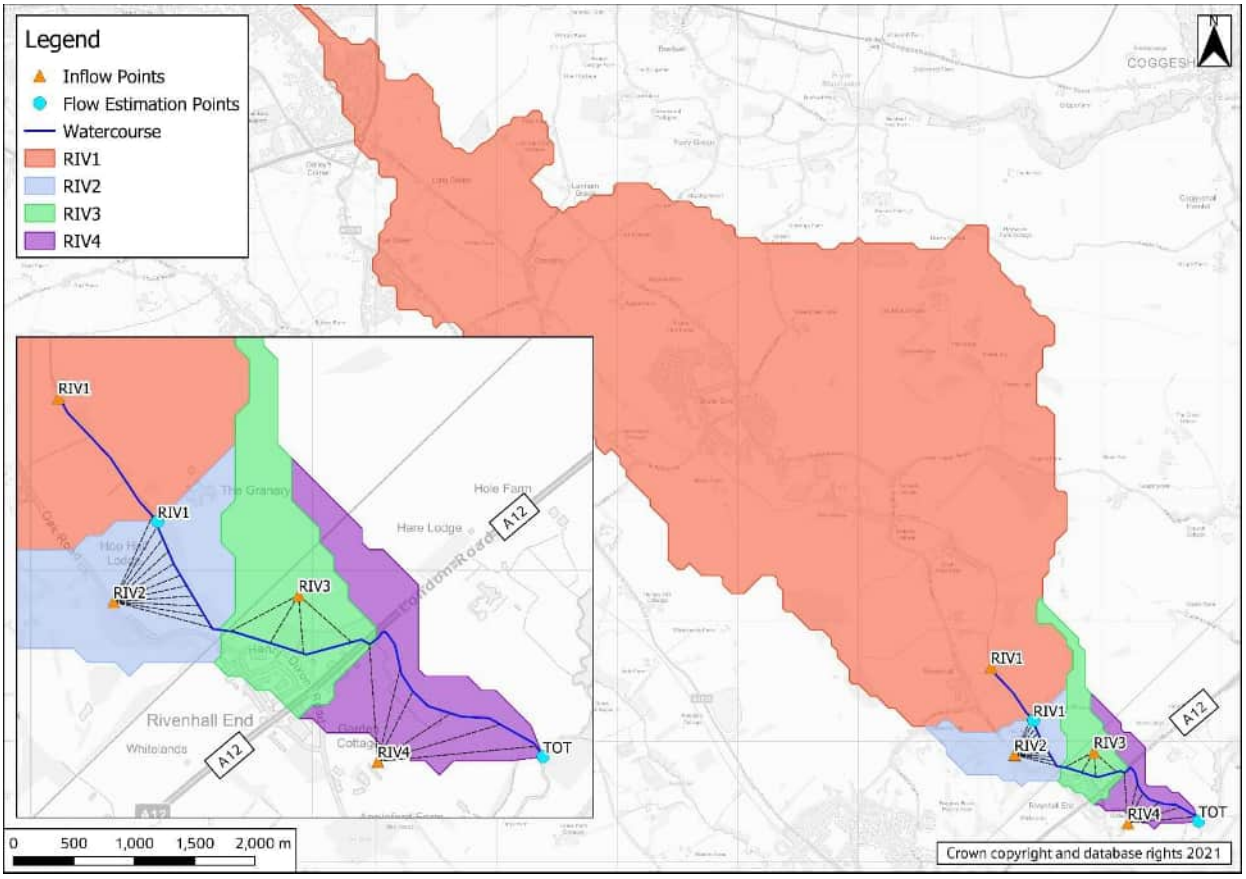
3.5 Inflows into the Rivenhall Brook hydraulic model

3.5.1 The Rivenhall Brook hydraulic model requires design hydrographs at four locations as described below:

- RIV1 – upstream model extent to the south-east of Rivenhall;
- RIV2 – lateral inflow between the upstream extent and the railway crossing;
- RIV3 – lateral inflow between the railway crossing and the A12 crossing; and
- RIV4 – lateral inflow between the A12 and the downstream model extent at the confluence with the River Blackwater.

3.5.2 A map showing the Rivenhall Brook hydraulic model extent and inflow locations is provided in Plate 3.1.

Plate 3.1 Model extent and inflow locations

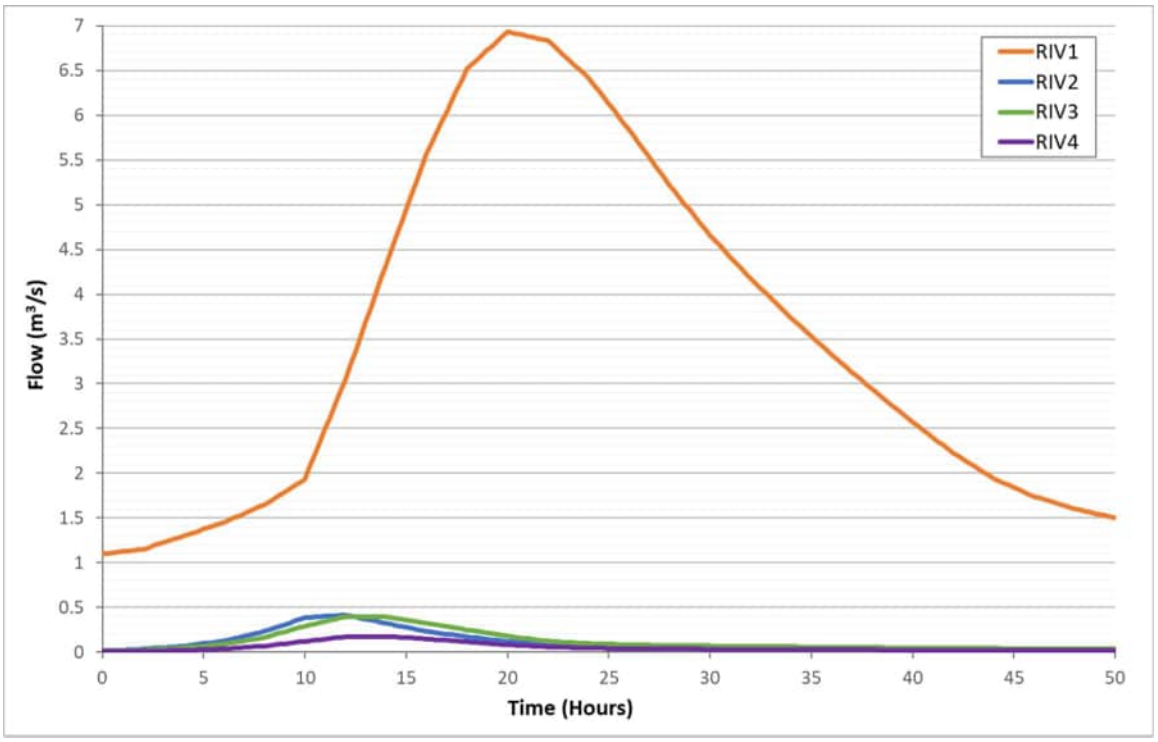


3.5.3 Peak inflows before flow reconciliation are given in Table 3.2 and inflow hydrographs for the 1% (1 in 100) AEP design event are presented in Plate 3.2.

Table 3.2 Inflow peaks as a function of event rarity

Inflow	Peak Flows (m^3/s)										
	AEP 50%	AEP 20%	AEP 10%	AEP 5%	AEP 3.33%	AEP 2%	AEP 1.33%	AEP 1%	AEP 1% + 38% CC	AEP 0.5%	AEP 0.1%
	1 in 2	1 in 5	1 in 10	1 in 20	1 in 30	1 in 50	1 in 75	1 in 100	1 in 100+CC	1 in 500	1 in 1000
RIV1	2.86	3.79	4.40	5.03	5.42	5.97	6.49	6.93	9.57	8.47	13.69
RIV2	0.16	0.22	0.26	0.30	0.32	0.35	0.39	0.41	0.57	0.51	0.84
RIV3	0.16	0.21	0.25	0.29	0.31	0.34	0.37	0.40	0.55	0.49	0.80
RIV4	0.07	0.09	0.11	0.13	0.14	0.15	0.17	0.18	0.24	0.22	0.36

Plate 3.2 1% (1 in 100) AEP event inflow hydrographs



4 Baseline modelling

4.1.1 The model was built using a one-dimensional (1D) schematisation, using the Flood Modeller version 4.6 (Jacobs, 2021) river modelling software. A 1D representation of the floodplain using extended cross-sections was considered appropriate as the Environment Agency (EA) flood maps show the flood extent is aligned to the river channel. Also, Rivenhall Brook runs within a steep and narrow floodplain that remains consistent across the model domain, and where the predominant flow path from upstream to downstream is maintained for all flows that are considered.

4.2 Model schematisation

4.2.1 Surveyed cross section data has been used to inform the in-channel geometry of the modelled watercourses. The locations of the surveyed cross-sections are shown in Plate 4.1.

4.2.2 To aid model performance on all modelled watercourses, interpolated cross sections were added between the surveyed cross sections. The spacing of the interpolated cross sections ranged from 20m to 50m.

4.2.3 The floodplain has been represented in the model using extended cross sections. Selected cross sections have been extended using LiDAR data (2018).

4.2.4 Due to the limited capacity of the twin culvert under Granary Farm's access road, some additional cross sections were added using the Digital Terrain Model (DTM) to represent the conveyance and storage upstream of that control structure.

4.2.5 On the downstream side of Granary Farm's access road, 4m away from the twin culvert outlets, the bed of the watercourse drops abruptly by 1m, as shown in Plate 4.2. A spill unit (labelled "RB_1903wu") was used to represent this in the model as an in-channel weir.

4.2.6 Downstream of the A12, the right bank floodplain topography shows a localised depression. A reservoir unit was therefore added, covering the area between the A12 and the farm access crossing located 465m downstream of the A12. The reservoir unit has a maximum plan area of 28,800m² and elevation ranging from 17.51mAOD to 24.28mAOD. The watercourse is connected to the reservoir with a series of lateral spill units along the (raised) right bank alignment. Water in this floodplain storage returns to Rivenhall Brook from the reservoir's south (downstream) edge at the farm bridge location. However, at this point the channel is particularly narrow, constricting flows and holding water back in the floodplain reservoir at high flows.

4.2.7 Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using the photographs taken during the topographic survey. Generally, gravel can be seen in the bed of Rivenhall Brook, with long grass and heavy vegetation on the floodplain. The Manning's 'n'

coefficients used in the model are shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow 1959).

Plate 4.1 Rivenhall Brook Baseline Model Schematisation

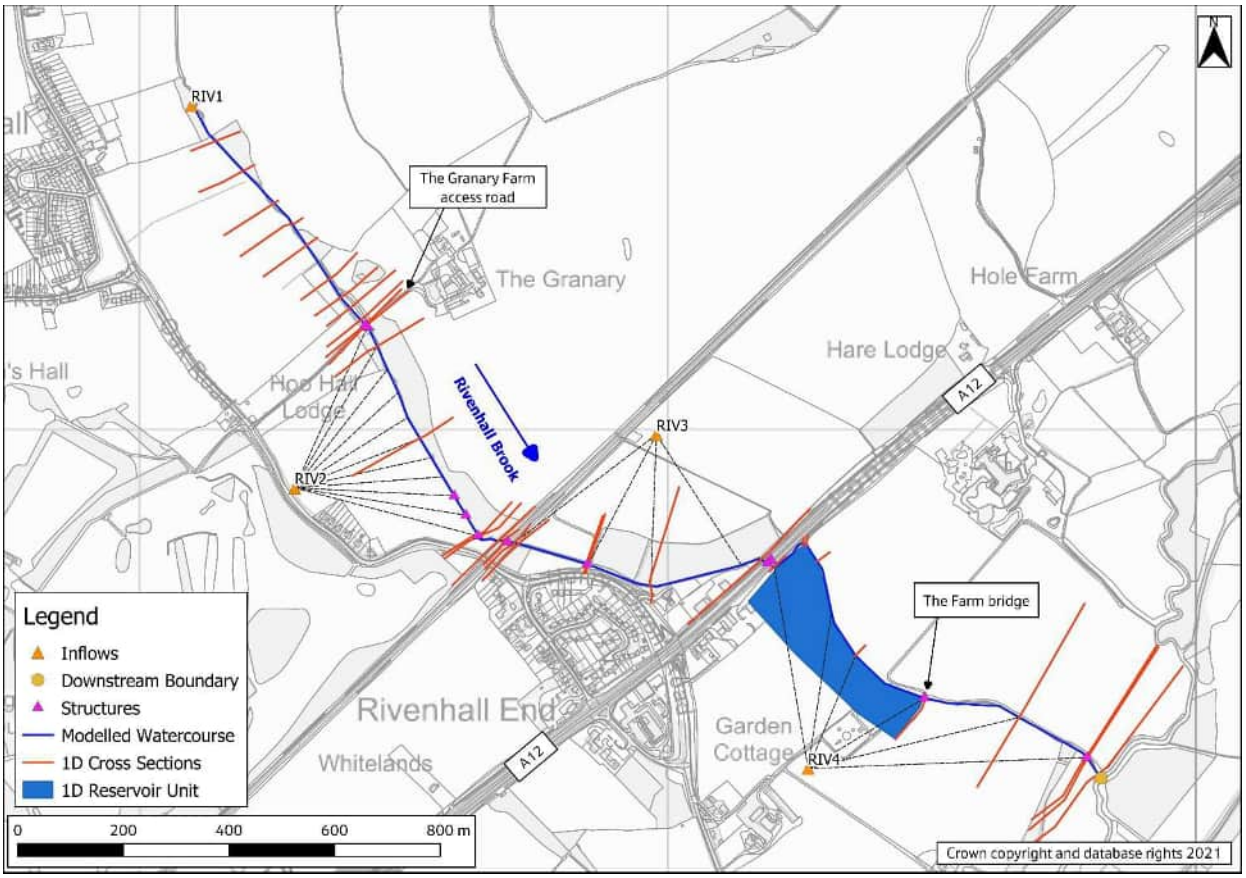


Plate 4.2 Edge of concrete slab at The Granary farm access road (downstream side)



Table 4.1 Manning's 'n' coefficients – 1D domain

Flood Modellers Nodes	Watercourse	Bed Manning's 'n'	Bed Material	Floodplain Manning's 'n'	Floodplain Material
RB_2362 to RB_0000	Rivenhall Brook	0.035	Clean, straight channel with Gravels and Weeds	0.060	Long grass, Light brush and trees

4.2.8 Nine hydraulic structures are included in the model. Their locations are shown in Plate 4.1. A structure schedule is presented in Appendix B. A footbridge 8m downstream of the railway Bridge (originally located at "RB_1364") is not represented as it would not affect the channel flow.

4.2.9 The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4.2. Inflow locations are shown in Plate 4.1.

Table 4.2 Boundary conditions – 1D domain

Type of Boundary	Flood Modeller Node Label	Description
QTBDY (Flow-Time Boundary)	RIV1	ReFH2 boundary. Applied at the upstream end of Rivenhall Brook.
QTBDY	RIV2	ReFH2 boundary. Applied laterally between RB_1903i2 (downstream of the inline weir) and RB_1446 (upstream of the railway) using a reach distribution method.
QTBDY	RIV3	ReFH2 boundary. Applied laterally between downstream of the railway to upstream of the A12 Road with a custom weighted flow distribution.
QTBDY	RIV4	ReFH2 boundary. Applied laterally between downstream of the A12 Road to the downstream end of the model, with custom weighted flow distribution.
Normal depth downstream boundary	RB_0000	Normal depth boundary condition applied to the downstream end of Rivenhall Brook. Sensitivity testing has shown that this boundary assumption does not affect model results in the area of interest (See Section 6.4).

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 38% uplift is in accordance with guidance (Environment Agency 2021) for climate change uplifts for this area. For further details of the climate change uplift, see the FRA (Jacobs 2022).

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline scenario 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope (see Section 6.4).

Table 5.1 Modelled events

Scenario	AEP Event		
	5% (1 in 20)	1% (1 in 100)	1% (1 in 100) +38%CC
Baseline	X	X	X
With-scheme	X	X	X
Roughness Sensitivity (1D)		X	
Hydrological Inflow Sensitivity		X	
Downstream Boundary Sensitivity (1D)		X	

6 Model proving

6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution, within a specified number of iterations, for each model time step. The convergence of the 1D model was checked and is within tolerance, as shown in Plate 6.1 and Plate 6.2 below, for the baseline and with-scheme scenario 1% (1 in 100) AEP +38%CC event respectively. This convergence plot is typical for all events modelled.

Plate 6.1 1D model convergence for baseline scenario – 1% (1 in 100) AEP +38%CC event

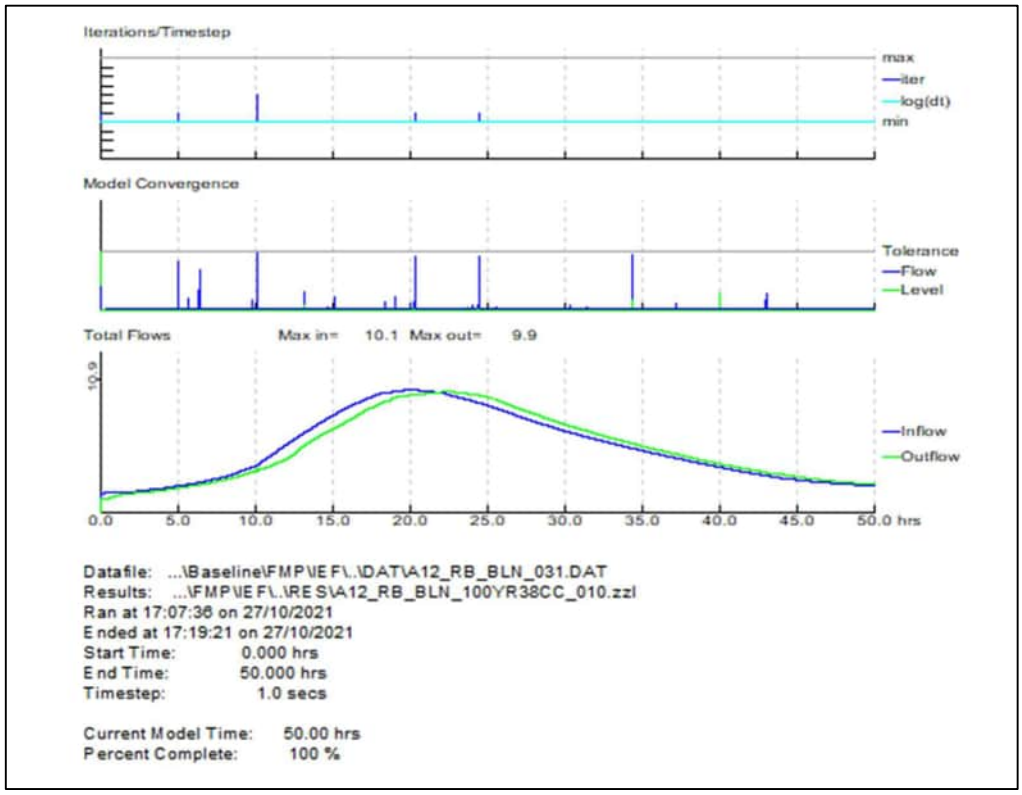
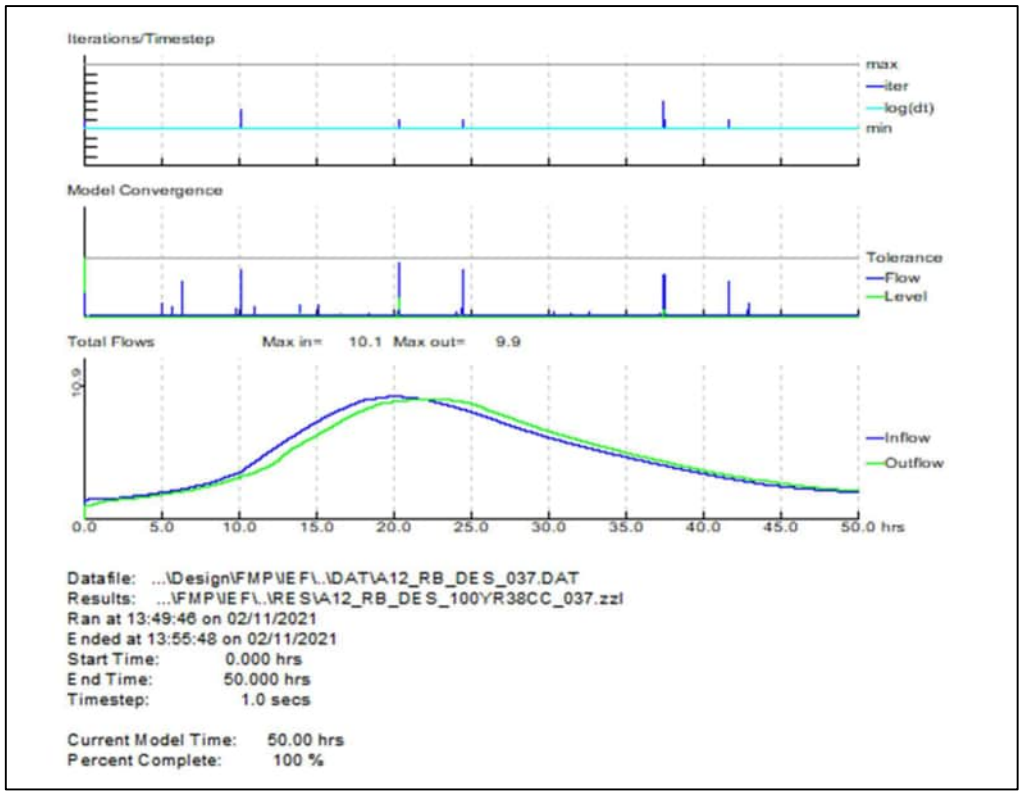


Plate 6.2 1D model convergence for with-scheme scenario – 1% (1 in 100) AEP +38%CC event



6.3 Calibration and verification

Flow reconciliation

6.3.1 The modelled flows at the downstream end of the model domain were checked against the hydrological target flows for the entire Rivenhall Brook catchment (TOT). Peak flows obtained from the model were found to slightly exceed the target flow estimates for the entire catchment for all AEP events simulated. However, the difference was less than 5% and therefore, as a conservative approach, no flow reconciliation was carried out.

Calibration

6.3.2 There are no gauges on Rivenhall Brook within the extent of the model. Therefore, it was not possible to calibrate the model. However, some data is available for a high-level verification of the model results.

Verification using historic data

6.3.3 The Environment Agency (EA) publishes historic flood maps for UK watercourses, which show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. However, a review of this data showed that there is no historic flood data for Rivenhall Brook.

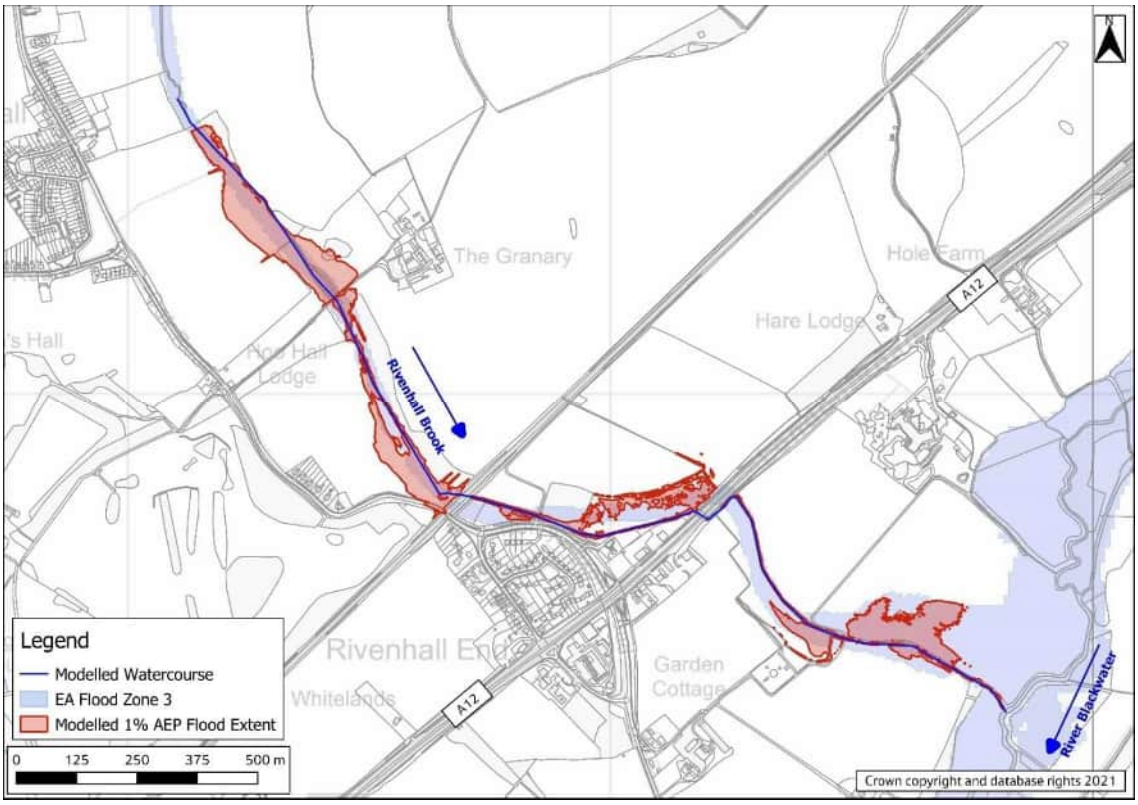
Verification using EA flood maps

6.3.4 Plate 6.3 shows the modelled 1% (1 in 100) AEP existing situation flood extent and the EA published Flood Zone 3. The comparison shows that the Environment Agency flood map predicts the flood extent to be of a simple form aligned to the centre line of the channel. Whilst the present study shows defined cells of flooding caused by a lack of capacity at several in-channel structures. This difference is expected as the EA map is generated based on the high-level J-Flow modelling system which utilises a relatively coarse 5m DTM grid and does not explicitly represent the in-channel capacity or the bridges and culverts on the watercourse.

Verification conclusion

6.3.5 In conclusion, there is limited data available for verification of the Rivenhall Brook model. The EA flood map does not compare well to the present study. However, this is due to the present modelling being a considerably more detailed assessment of flooding than the methodology utilised for the EA flood map.

Plate 6.3 Modelled 1% (1 in 100) AEP event flood extent vs. Environment Agency flood zone 3



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness, hydrological inflows and downstream boundary conditions. These analyses were performed on the penultimate version of the baseline model. However, the changes made in the final version of the baseline model were minor. Therefore, the changes will not affect the general outcomes of the sensitivity tests. All sensitivity tests were performed using the 1% (1 in 100) AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by ±20%. Table 6.1 shows the impact of changing the model roughness on predicted peak water levels. The results show that the in-channel water levels are not very sensitive to changes in roughness.

Table 6.1 Roughness sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	0.11	0.00	0.05
-20% Roughness	-0.16	0.02	-0.06

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by ±20%. Table 6.2 shows the impact of changing the model inflows on predicted peak water levels. The results show that the water levels are moderately sensitive to changes in flow.

Table 6.2 Flow sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.20	0.02	0.09
-20% Flow	-0.28	-0.03	-0.12

Downstream boundary condition sensitivity

6.4.4 The slope in the normal depth downstream boundary was adjusted by ±20%. Also, the 1% (1 in 100) AEP +65%CC (Climate Change) maximum levels of the receiving River Blackwater were used as an additional sensitivity test on the downstream boundary. Table 6.3 shows the response at the downstream end of the model (Flood Modeller node

RB_0000). The location at which there is no change in water level as a result of changing the downstream boundary has been identified. Distances from this location, in relation to the downstream end of the model (tailwater distance) and in relation to the proposed-scheme are also shown.

6.4.5 The results show that the effect of the downstream boundary does not reach the proposed-scheme location.

Table 6.3 Downstream boundary condition sensitivity results

Sensitivity	Water Level Difference at RB_0000 (m)	Tailwater Distance (m)	Distance to the proposed-scheme (m)
+20% Downstream boundary slope	0.02	40	595
-20% Downstream boundary slope	-0.02	40	595
R. Blackwater Peak 1% (1 in 100) AEP +65%CC water level	0.76	190	445

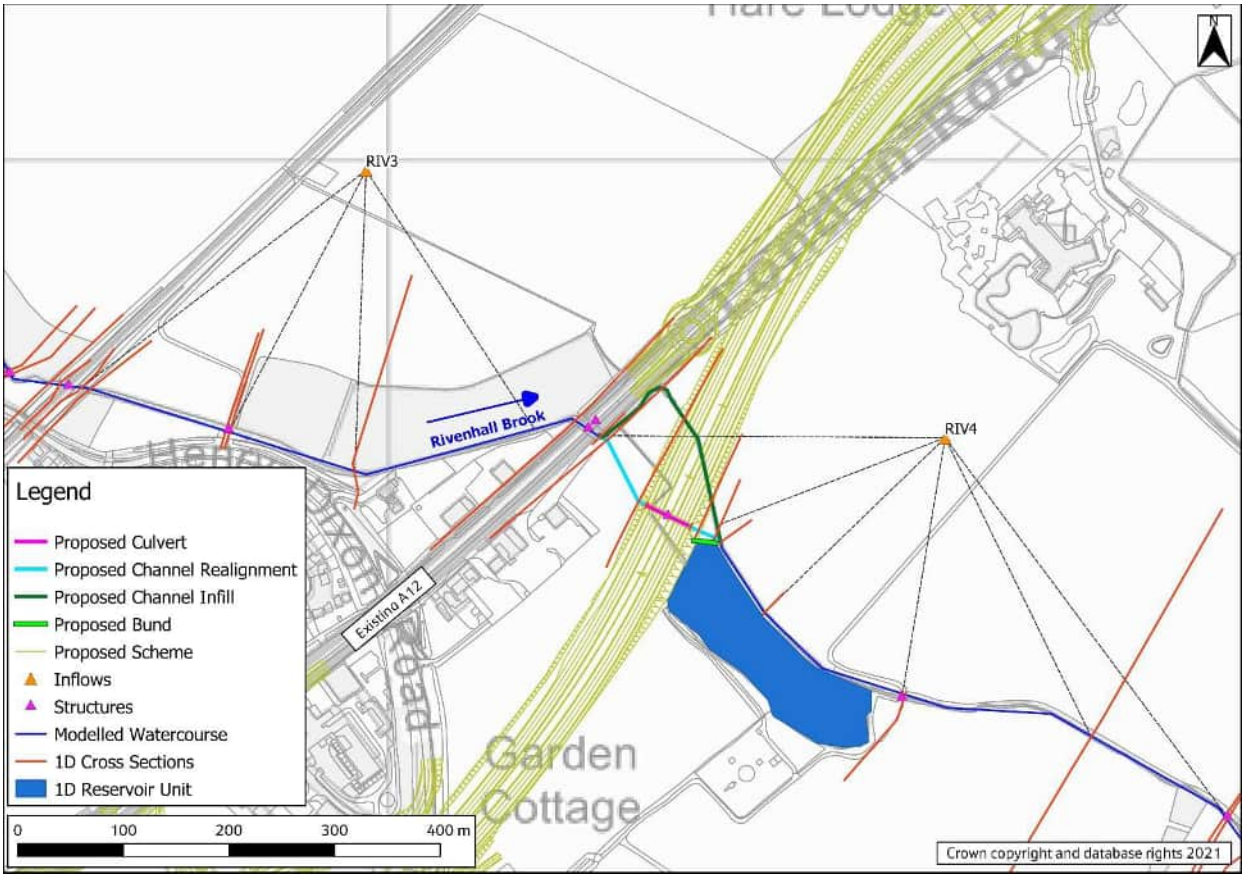
7 With-scheme model build

7.1.1 The proposed-scheme consists of a new offline road adjacent to the current A12 which branches off the existing A12 at approximately 200m north-east from the existing Rivenhall Brook A12 crossing. A new culvert of 46m length and 4.5m wide by 3.6m high is required to convey Rivenhall Brook below the new road crossing. The new embankment cuts through the floodplain as shown in Plate 7.1.

7.1.2 In addition to the new culvert, the proposed-scheme requires the realignment of Rivenhall Brook between the existing and the new A12 crossing and between the new A12 crossing and the existing Rivenhall Brook. Downstream of the new culvert a raised right bank (bund), which ties in with the existing raised right bank of Rivenhall Brook, is required to ensure there is no increased flooding to the floodplain. This raised right bank has been represented in the model by allowing the right bank of the realigned channel to artificially 'glass wall' at this location, therefore preventing any spillage into the right bank floodplain from the new channel section. The location of the raised bund is shown in Plate 7.1.

7.1.3 Because the floodplain is represented as a reservoir unit in the baseline scenario, connected to the river sections adjacent to it via spill units, the realignment of the watercourse required the removal of 6 cross sections and associated spills initially connected to the floodplain reservoir unit. The realigned channel length is shortened, from 315m to 238m compared with the existing watercourse. Also, the realigned channel bed slope has increased in order to tie into the existing bed levels upstream and downstream from the proposed-scheme. The reservoir unit itself has been clipped to account for the new A12 crossing the floodplain.

Plate 7.1 Proposed-scheme Layout at Rivenhall Brook Crossing



8 Model results

8.1 Baseline model results

8.1.1 In-channel maximum water levels have been inspected at key locations in relation to the proposed-scheme. Table 8.1 shows in-channel maximum water levels for the 1% (1 in 100) AEP +38%CC event at these locations. The in-channel water levels at these locations for all modelled AEP events are shown in Appendix C.

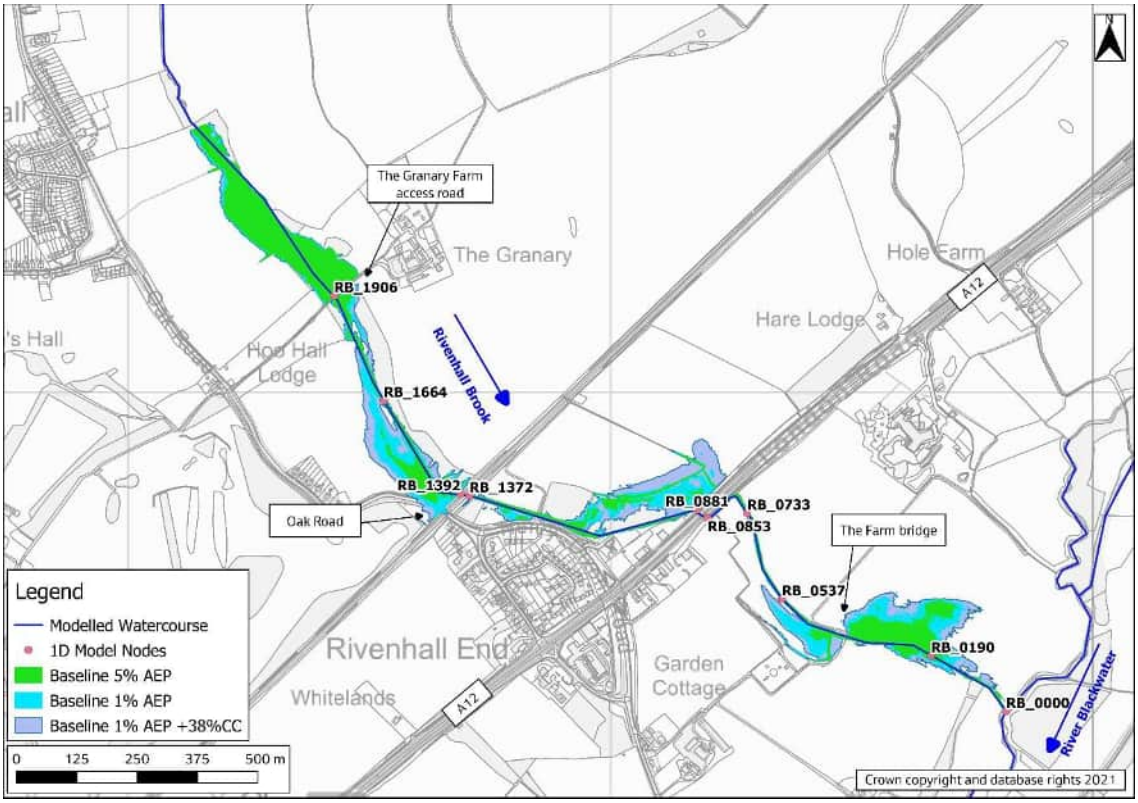
8.1.2 Plate 8.1 shows the maximum flood extent for all AEP events modelled for the baseline scenario. Floodplain storage is noted alongside Rivenhall Brook upstream of The Granary Farm access road, where flows are locally constricted for all AEP events.

8.1.3 Around Rivenhall End village and near the existing A12, Rivenhall Brook remains largely in-bank for the 5% (1 in 20) AEP event, but there is some out of channel flow for the 1% (1 in 100) AEP event due to the surcharge of watercourse crossing structures. The 1% (1 in 100) AEP +38%CC event exhibits out of channel flow for almost all of the modelled reach. However, the results show that no properties are flooded within the model domain for any of the AEP events simulated and the A12 itself is free from flooding.

8.1.4 It is evident when inspecting the full suite of flood event simulations (Appendix C) that the limited capacity of several crossing structures is a key factor contributing to flood risk on Rivenhall Brook. These are summarised below:

- Backing up upstream of the “The Granary” Road Bridge (model node RB_1906) is observed for all the modelled events.
- Backing up upstream of both the railway culvert, and the A12 Road crossing for events of 1% (1 in 100) AEP and above, without overtopping either embankment.
- Flooding is observed on Oak Road which crosses under the railway to the right of Rivenhall Brook for events of 1% (1 in 100) AEP and above.
- The right bank flood basin downstream of the existing A12 Road floods increasingly from the 5% (1 in 20) AEP to the 1% (1 in 100) AEP +38%CC flood event, receiving flows from all Rivenhall Brook lateral spill sections downstream from the A12 culvert exit.
- For the 1% (1 in 100) AEP and 1% (1 in 100) AEP +38%CC events, the farm bridge downstream of A12 causes backing up within the right bank flood basin, up to 150m upstream of the structure.
- Downstream of the farm bridge, significant inundation of the left bank floodplain occurs during all modelled AEP events.

Plate 8.1 Baseline Modelled Flood Extents for All Events



8.2 With-scheme model results

8.2.1 Table 8.1 shows In-channel maximum water levels for the baseline and with-scheme scenarios at key locations, for the 1% (1 in 100) AEP +38%CC event. The in-channel water levels at these locations for all modelled AEP events are shown in Appendix C. Table 8.1 shows that peak water levels upstream of the new crossing are reduced due to implementation of the proposed-scheme. Such reduction is expected since the change in slope of the realigned channel (compared with the existing watercourse) improves the conveyance of the existing A12 crossing whilst the new culvert does not restrict flow conveyed downstream.

8.2.2 For all modelled events, flow is contained within the realigned channel between the existing A12 and the new culvert. Downstream of the new culvert, flow is contained within channel throughout the 33m realigned reach due to the raised bank (bund) imposed on the right bank of the channel. To contain the 1% (1 in 100) AEP + 38%CC event, the maximum required height of the bund is 0.62m (without the inclusion of freeboard) with a length of 22m. The bund will be required to tie-in with the culvert wingwall structure at the upstream extent and the existing raised right-hand side banktop of Rivenhall Brook on the downstream extent.

8.2.3 Due to flow being contained within channel throughout the realigned reach and subsequently less flow transferred to the right bank floodplain, water level in the channel downstream of the new culvert is increased compared to baseline. This results in a negligible (< 0.01m) reduction in maximum water level within the right bank floodplain during all modelled events.

8.2.4 Plate 8.2 shows the comparison of maximum modelled flood extents for the with-scheme and baseline scenarios for the 1% (1 in 100) AEP + 38%CC event. It is evident that there is a minor reduction in flood extent upstream from the A12. Immediately downstream of the new culvert, flow exiting the culvert is managed by the raised right-hand side banktop which directs flow back into the original Rivenhall Brook channel. Elsewhere the modelled extent is unchanged.

8.2.5 Plate 8.3 shows the water level difference between the with-scheme and baseline scenarios for the 1% (1 in 100) AEP + 38%CC event, whilst water level difference maps for all other AEP events can be found in Appendix D. Both the existing and the proposed A12 roads remain free from flooding. The difference map shows there to be some reduction in maximum flood levels upstream from the A12 and an increase in water level immediately downstream of the new culvert due to the raised right bank along the realigned channel, but elsewhere only negligible effect on modelled flood risk.

8.2.6 Plate 8.4 compares Rivenhall Brook flows between the baseline and with-scheme scenarios downstream of the proposed-scheme for the 1% (1 in 100) AEP + 38%CC event and shows the two scenarios have identical

pass forward flow for the entire model simulation. There is also no change in pass forward flow for the two lower magnitude flood events simulations.

Table 8.1 In-channel water levels at key locations for the 1% (1 in 100) AEP +38%CC event

Nodes	Description	Water Level (mAOD)		
		1% (1 in 100) AEP +38%CC		
		Baseline	With-scheme	Difference (m)
RB_1664	272m upstream of the railway culvert	23.18	23.18	+0.00
RB_1392	Upstream of the railway culvert	22.95	22.95	+0.00
RB_1372	Downstream of the railway culvert	22.07	22.07	+0.00
RB_0881	Upstream of the existing A12 Road culvert	20.61	20.47	-0.14
RB_0853	Downstream of the existing A12 Road culvert	20.28	19.92	-0.36
RB_0190	445m downstream of the proposed carriageway culvert	17.41	17.41	+0.00

Plate 8.2 Flood extent comparison between baseline and with-scheme scenarios for the 1% (1 in 100) AEP +38%CC

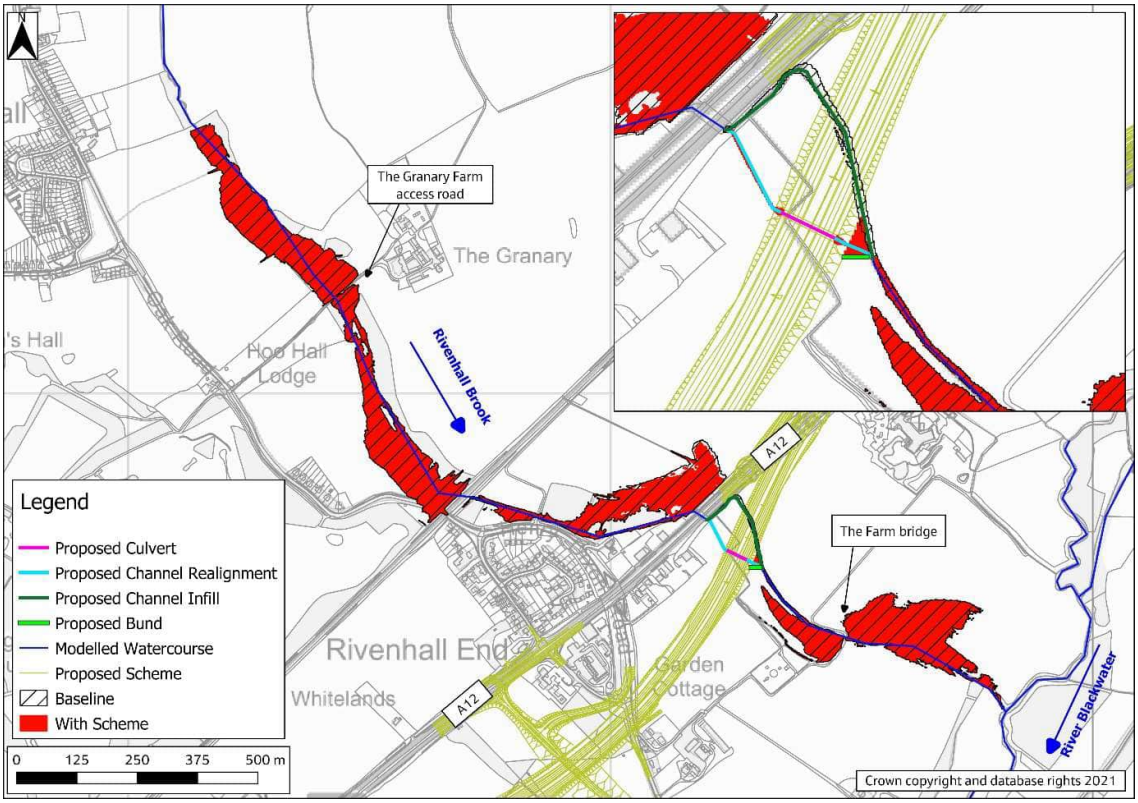


Plate 8.3 Water Level Difference between With-scheme and Baseline for the 1% (1 in 100) AEP + 38%CC event

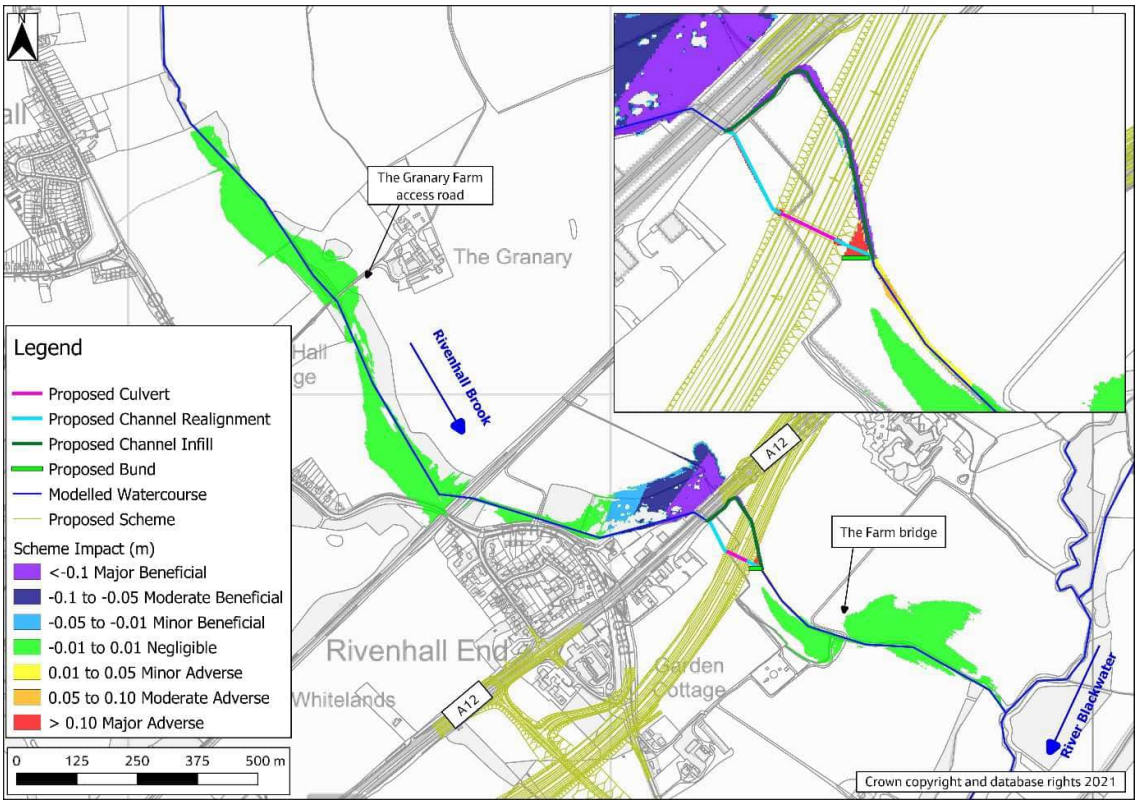
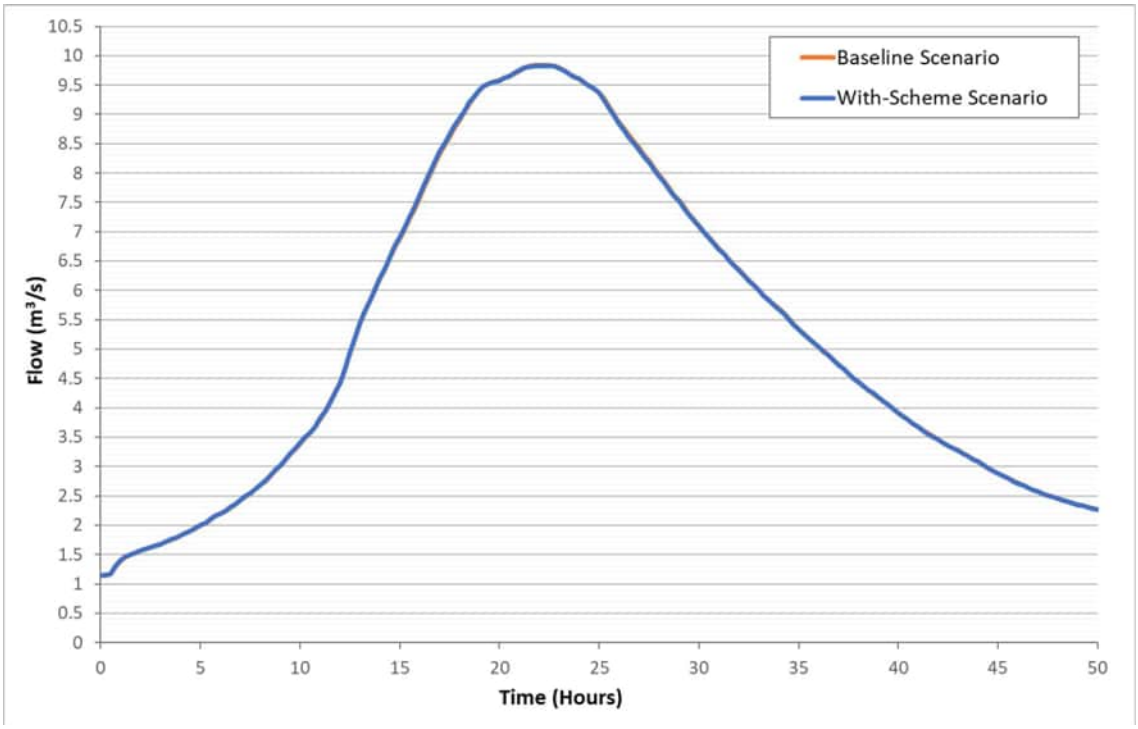


Plate 8.4 Flow comparison between the With-scheme and Baseline scenarios for the 1% (1 in 100) AEP +38%CC event, at the Farm Bridge (downstream of the proposed-scheme)



9 Model assumptions and limitations

9.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological and topographic data included in the model. Therefore, the most appropriate available information has been used to construct the model.

9.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are considered to be generally conservative for modelled water levels at the proposed-scheme and are therefore appropriate for the flood risk assessment.

9.2 Hydrology

9.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- The River Ter at Crabbs Bridge has been selected as a donor catchment in the statistical method with the assumption that it is hydrologically similar to the Rivenhall Brook and will respond to rainfall in the same manner.
- The 68% and 95% confidence limits for the QMED calculation at the downstream model extent, using the statistical method, are 1.4 – 3.1m^3/s and 0.9 – 4.0m^3/s respectively.

9.3 Hydraulic modelling

9.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for Rivenhall Brook are as follows:

- Surveyed cross sections have been extended into the floodplain using elevations informed by LiDAR DTM data (2018).
- Channel roughness has been assigned using the best available information (survey data and aerial photographs). The roughness estimates used are based on available guidance (Chow 1959).
- Selected structures have been simplified to aid the representation in the model (see Section 4).
- Hydraulic coefficients for structures have been applied using available guidance within the Flood Modeller software. The dimensions for structures have been based on detailed survey measurements.
- Implementing River Blackwater maximum levels to the downstream boundary for the 1% (1 in 100) AEP +65%CC events, impacts on water levels up to 190m upstream of the downstream model extent, which is 445m downstream from the proposed-scheme. Therefore, it was deemed appropriate to use a normal depth boundary.

- No calibration was carried out as the Rivenhall Brook catchment is ungauged.
- The channel realignment utilised in the with-scheme scenario has been schematised by connecting to the existing downstream channel at model node RB_0733In2. This node is an interpolate unit within the baseline scenario, updated to river cross-section using the HEC-RAS interpolation tool within the with-scheme scenario. As there is no surveyed cross-section within the channel at this location, the precise elevations and geometry of the channel are uncertain.

10 Conclusion

10.1.1 This report has detailed the modelling carried out to assess the baseline flood risk for Rivenhall Brook with respect to the location of the proposed-scheme.

10.1.2 The results of the baseline modelling have shown that there are no properties at risk of flooding and the A12 remains free from flooding up to for the 1% (1 in 100) AEP +38% Climate Change event.

10.1.3 The proposed-scheme was modelled, and it was found that the proposed-scheme has no impact on any properties and the A12 still remains free from flooding. The implementation of the proposed-scheme leads to a significant reduction in modelled water levels immediately upstream of the existing A12, but otherwise no significant effect on flood risk. Pass forward flows predicted by the model are unchanged between the baseline and with-scheme scenarios.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

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Soil Map of England and Wales - Soil Survey of England and Wales (1983). Soil Maps of England and Wales Scale 1:250,000.

Appendix A Rivenhall Brook Flood Estimation Calculation Record

A.1.1 See document HE551497-JAC-EWE_SCHW-RP-LE-0076.docx

Appendix B Structure Schedule

Table B.1 Model Structure Schedule

Structure Name	Node Label(s)	XY Coordinates	Modelling Approach
Culvert under Road Bridge, 05.022	RB_1906ci1	583429.1, 217199.8	Represented as twin circular culvert with a spill unit in parallel. Inverts of the culvert are adopted from the survey drawing. Manning's 'n' roughness of 0.015 is used to represent the brick culvert. Weir coefficient of 1.2 is used for the spill to represent the structure deck. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all parameters.
Weir downstream of the culvert 05.022	RB_1903wu	583434.70, 217199.45	Spill unit used to represent the drop of 1m, based on survey plus interpretation from site photographs. Weir coefficient of 1.0 is used for the online spill.
Road Bridge, 05.019	RB_1450b	583643.1, 216800.9	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m applied. Data from detailed survey. Spill has weir coefficient of 1.0 for the concrete slab. FM default coefficients applied for all other parameters.
Railway Bridge, 05.017	RB_1392ci	583698.7, 216789.5	Represented as symmetrical culvert with spill unit in parallel. Data from detailed survey. Colebrook coefficient of 0.008m is used for the sides and soffit, while 0.614 is used for the bed.

			Spill unit has weir coefficient of 1.2 as it considers Henry Dixon Road underpass in its section (as a slot) beneath the railway embankment. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all parameters.
Road Bridge, 05.013	RB_1229b	583848.0, 216745.2	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill has weir coefficient of 1.0 for the concrete slab. FM default coefficients applied for all other parameters.
A12 Road Bridge, 05.010	RB_0881ci	584180, 216757	Represented as asymmetrical culvert without any spill unit in parallel. Data from detailed survey. Manning's 'n' roughness of 0.015 is used for the sides and soffit, while 0.035 is used for the bed. Spill was ignored as road embankment is never overtopped. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all parameters.
A12 Road side drain culvert, 05.010	RB_0881o	584225, 216800	Represented as a circular orifice of 450mm diameter added in parallel to the A12 Road Bridge culvert to represent the crossing of a side drain (60m away from Rivenhall Brook on the left floodplain). FM default coefficients applied for all parameters.
Road Bridge, 05.006	RB_0381b	584486.0, 216492.5	Represented as an arch bridge with a spill unit in parallel.

			Lower and upper transition distance of 0.1m were ignored as the bridge never surcharges. Data from detailed survey. Spill has weir coefficient of 1.2 to represent the structure deck. FM default coefficients applied for all other parameters.
Footbridge, 05.003	RB_0041b	584793.3, 216379.1	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill has weir coefficient of 0.8 to represent the grass surface. FM default coefficients applied for all other parameters.

Appendix C Maximum Water Level Tables and Long Sections

Table C.1 Maximum water level of Baseline and With-scheme models for all events at key nodes

Model Node	Description	Water Level (mAOD)								
		5% (1 in 20) AEP			1% (1 in 100) AEP			1% (1 in 100) AEP +38%CC		
		Baseline	With-Scheme	Difference	Baseline	With-Scheme	Difference	Baseline	With-Scheme	Difference
RB_1664	272m upstream of the railway culvert	22.74	22.74	+0.00	22.96	22.96	+0.00	23.18	23.18	+0.00
RB_1392	Upstream of the railway culvert	22.12	22.12	+0.00	22.49	22.49	+0.00	22.95	22.95	+0.00
RB_1372	Downstream of the railway culvert	21.55	21.55	+0.00	21.72	21.72	+0.00	22.07	22.07	+0.00
RB_0881	Upstream of the existing A12 Road culvert	20.04	19.99	-0.05	20.29	20.19	-0.10	20.61	20.47	-0.14
RB_0853	Downstream of the existing A12 Road culvert	19.88	19.62	-0.26	20.09	19.76	-0.33	20.28	19.92	-0.36
RB_0190	445m downstream of the proposed carriageway culvert	17.26	17.26	+0.00	17.35	17.35	+0.00	17.41	17.41	+0.00

Plate C.1 Long profile for baseline and with-scheme scenarios showing 5% (1 in 20) AEP peak water level

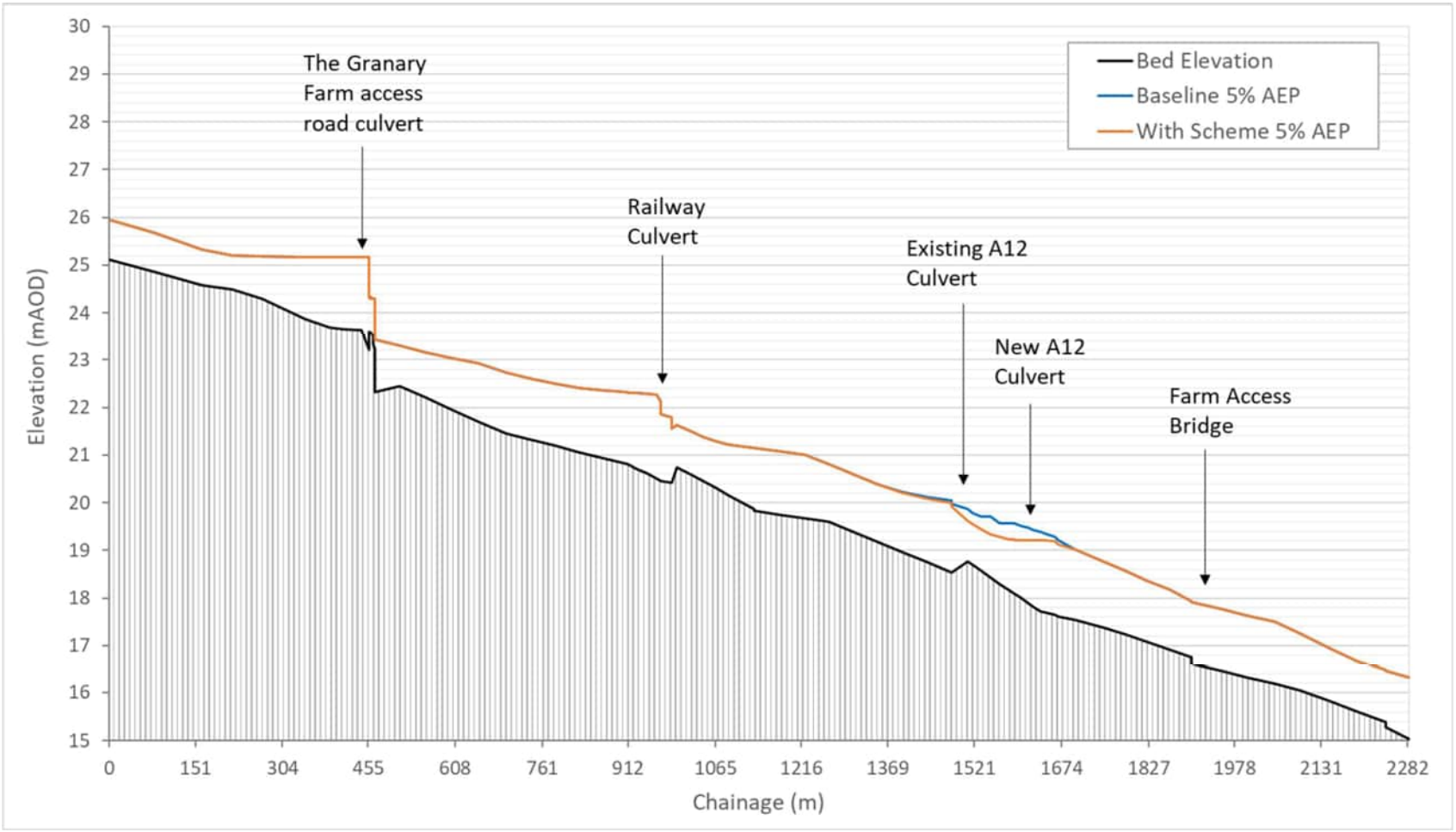


Plate C.2 Long profile for baseline and with-scheme scenarios showing 1% (1 in 100) AEP peak water level

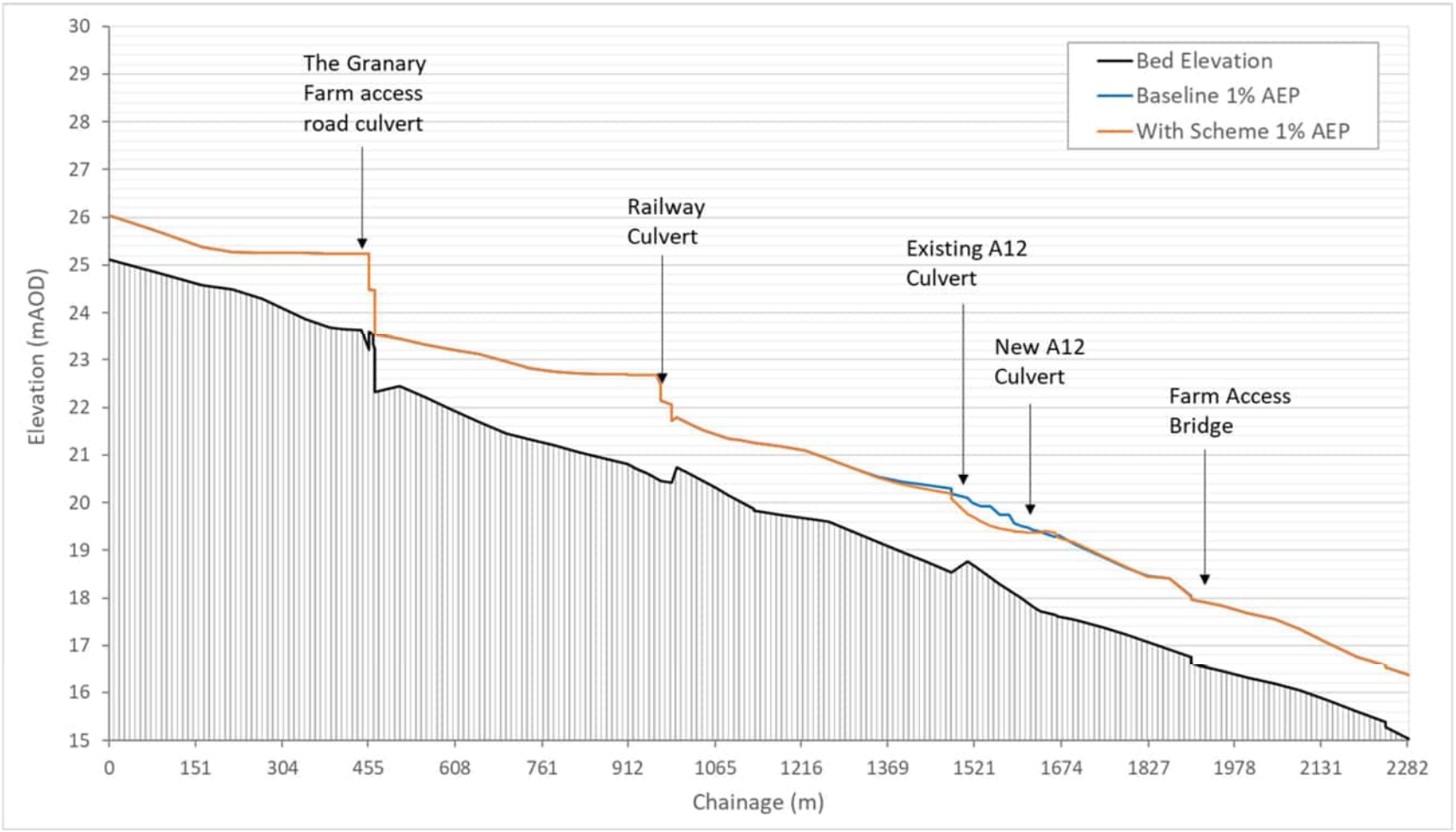
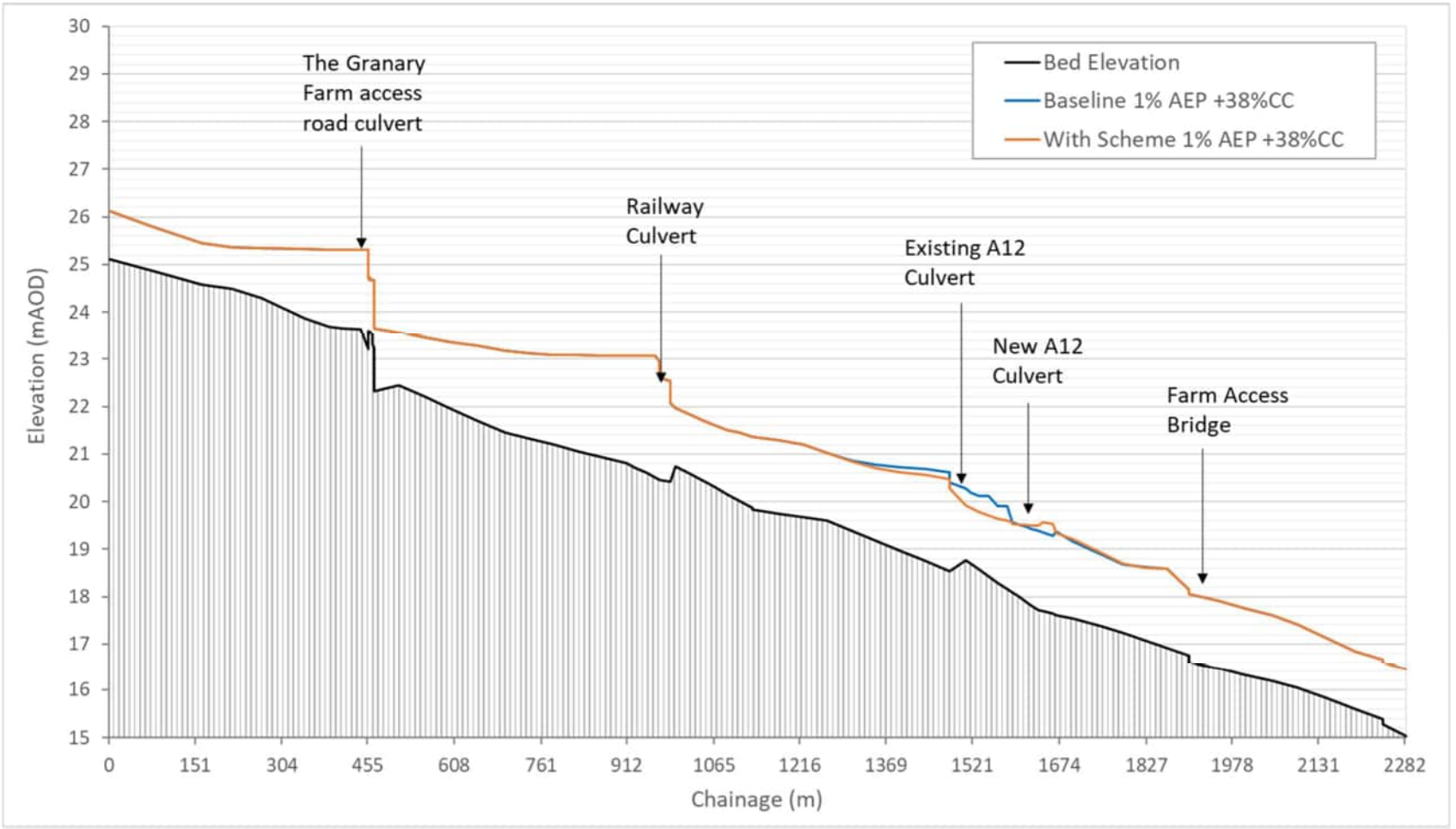


Plate C.3 Long profile for baseline and with-scheme scenarios showing 1% (1 in 100) AEP +38%CC peak water level



Appendix D Flood Maps

Plate D.1 5% (1 in 20) AEP maximum flood extent for baseline and with-scheme scenarios

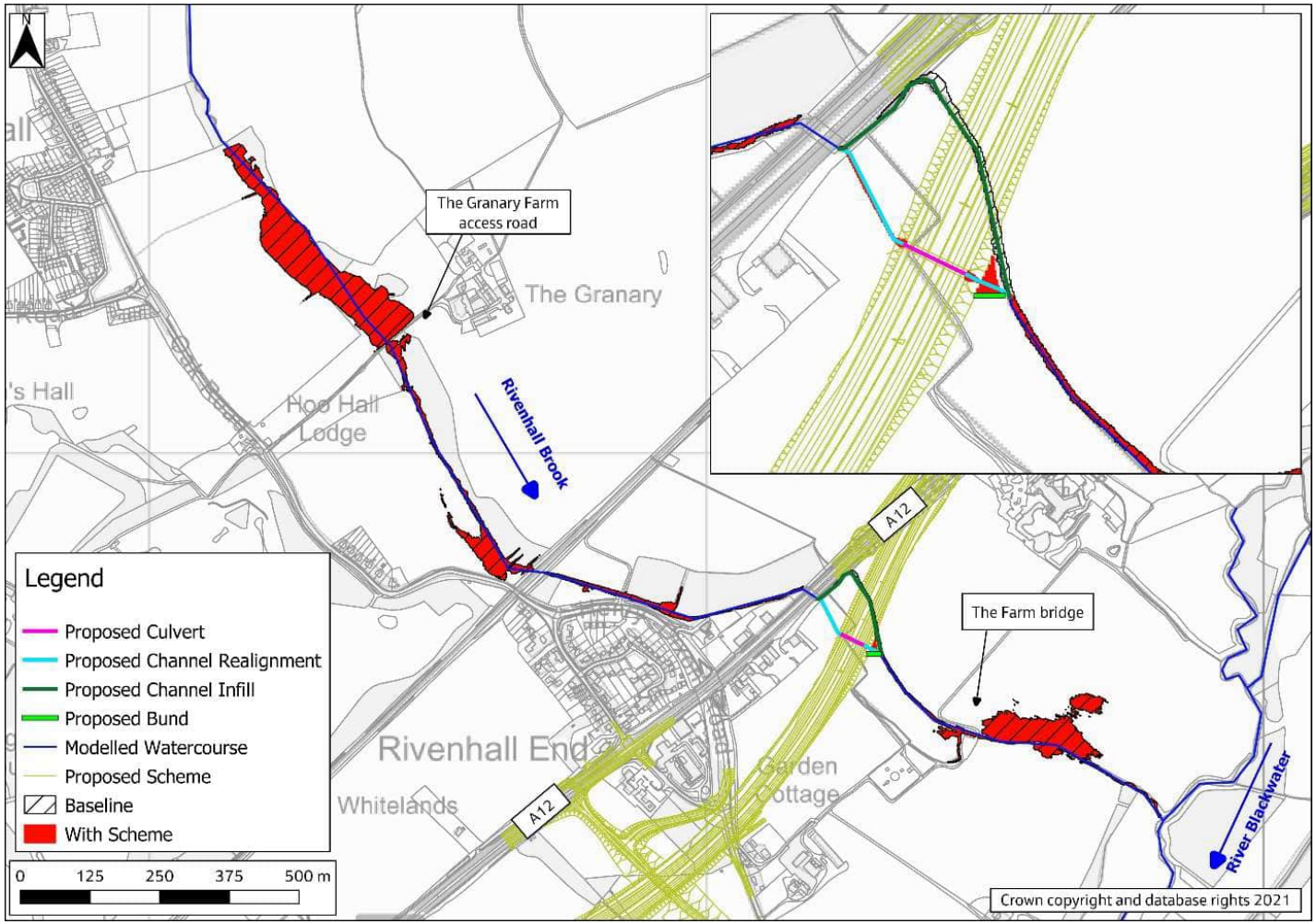


Plate D.2 1% (1 in 100) AEP maximum flood extent for baseline and with-scheme scenarios

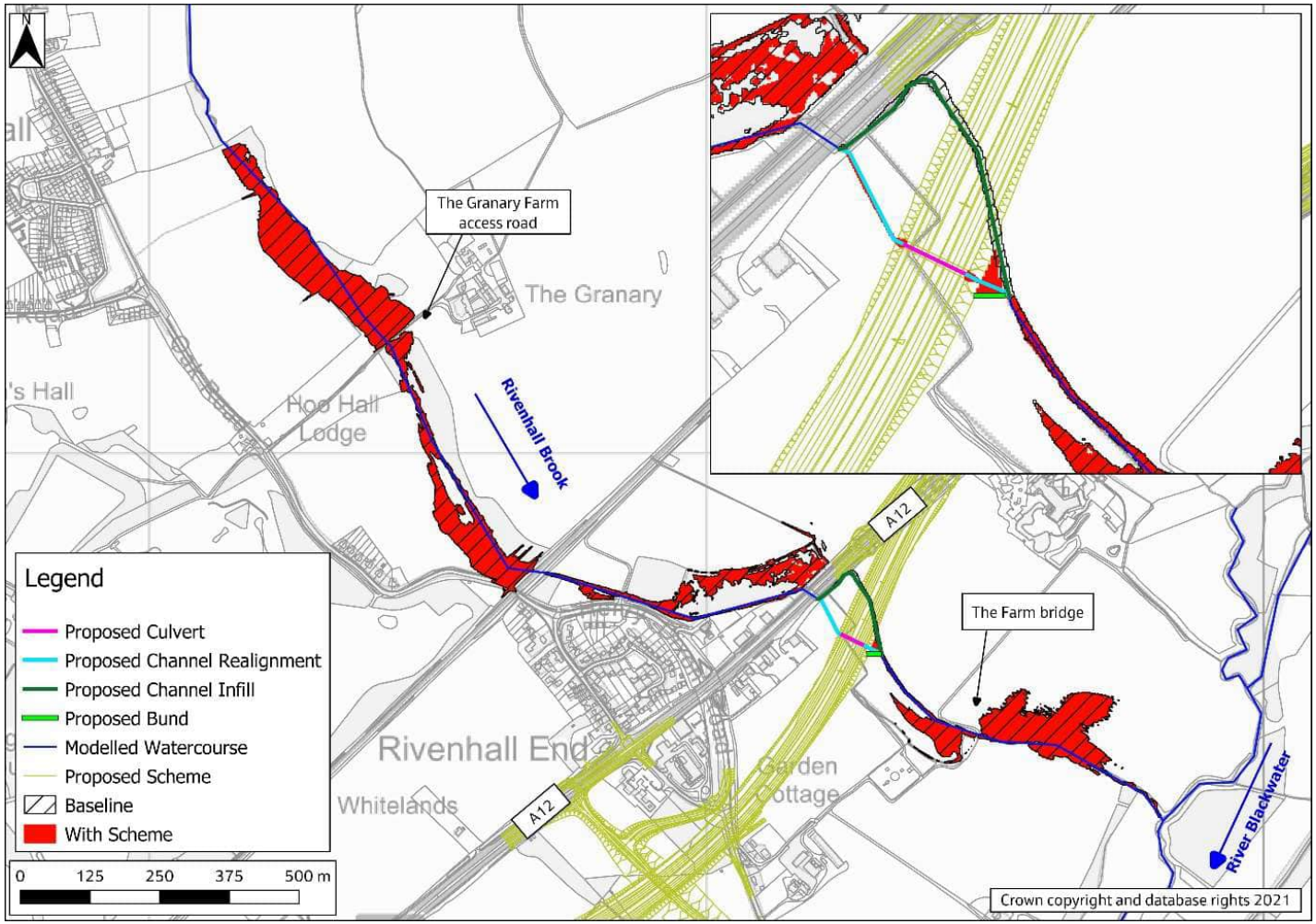


Plate D.3 1% (1 in 100) AEP +38%CC maximum flood extent for baseline and with-scheme scenarios

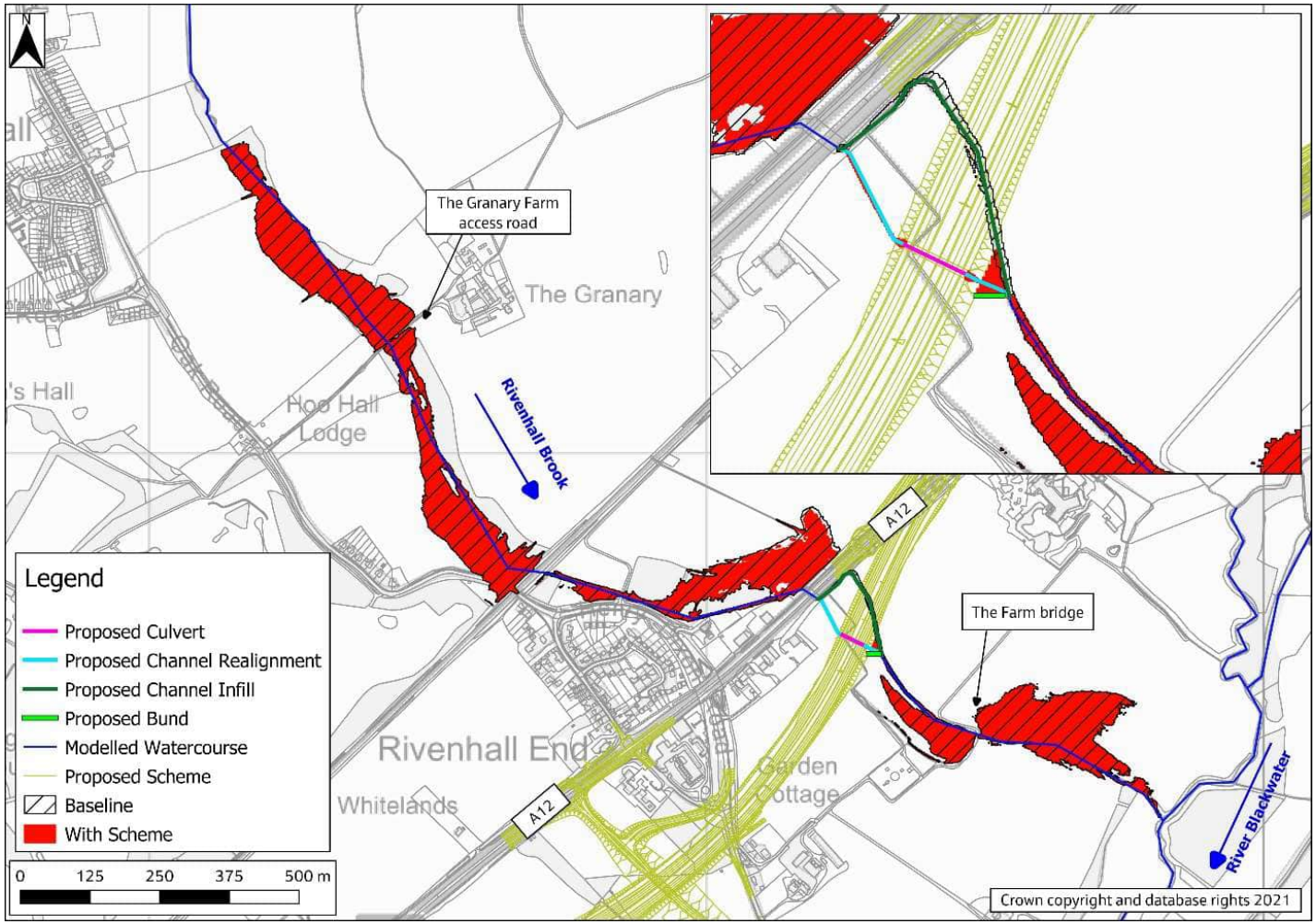


Plate D.4 Water level difference (with-scheme - baseline) for the 5% (1 in 20) event AEP

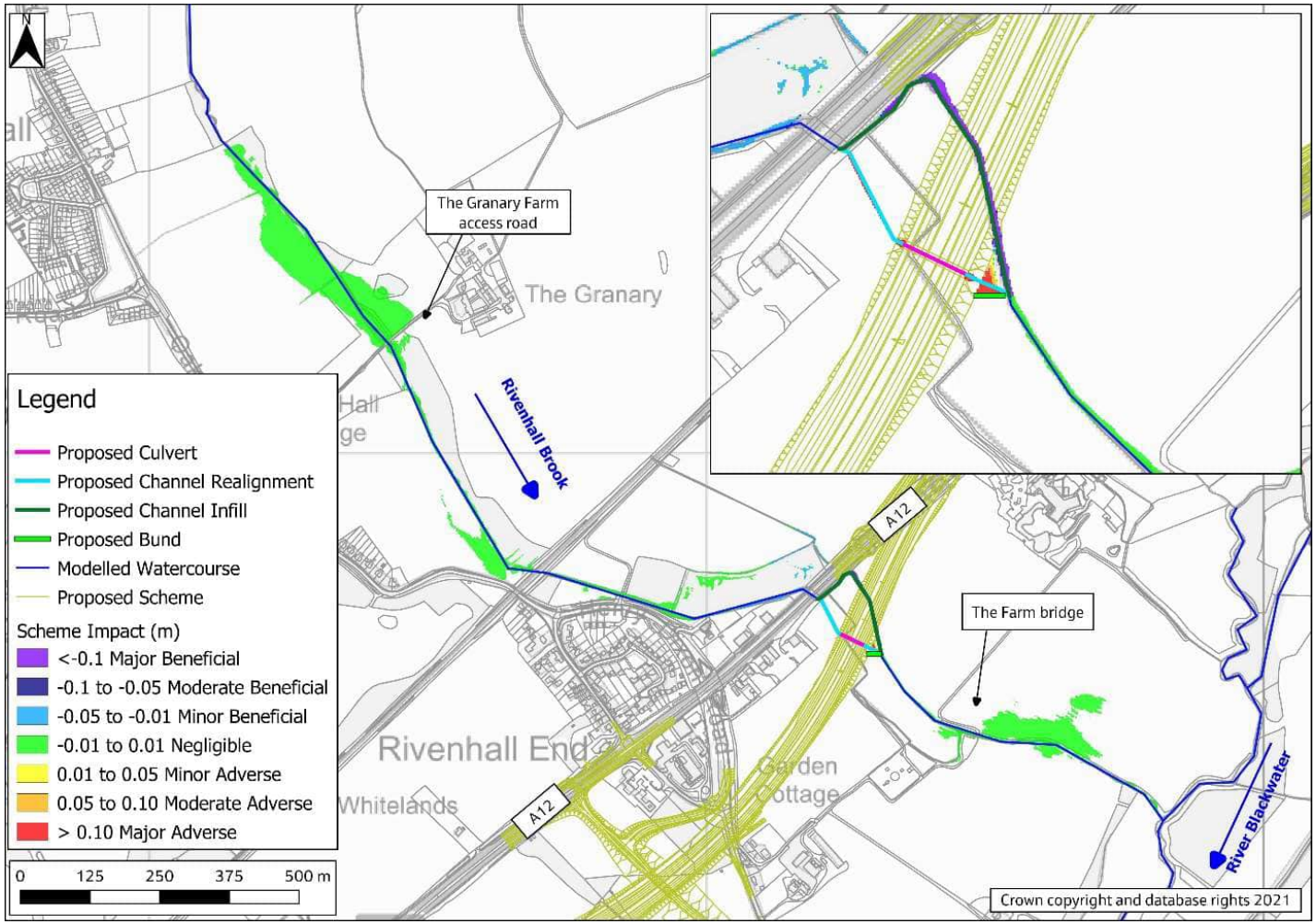


Plate D.5 Water level difference (with-scheme - baseline) for the 1% (1 in 100) AEP event

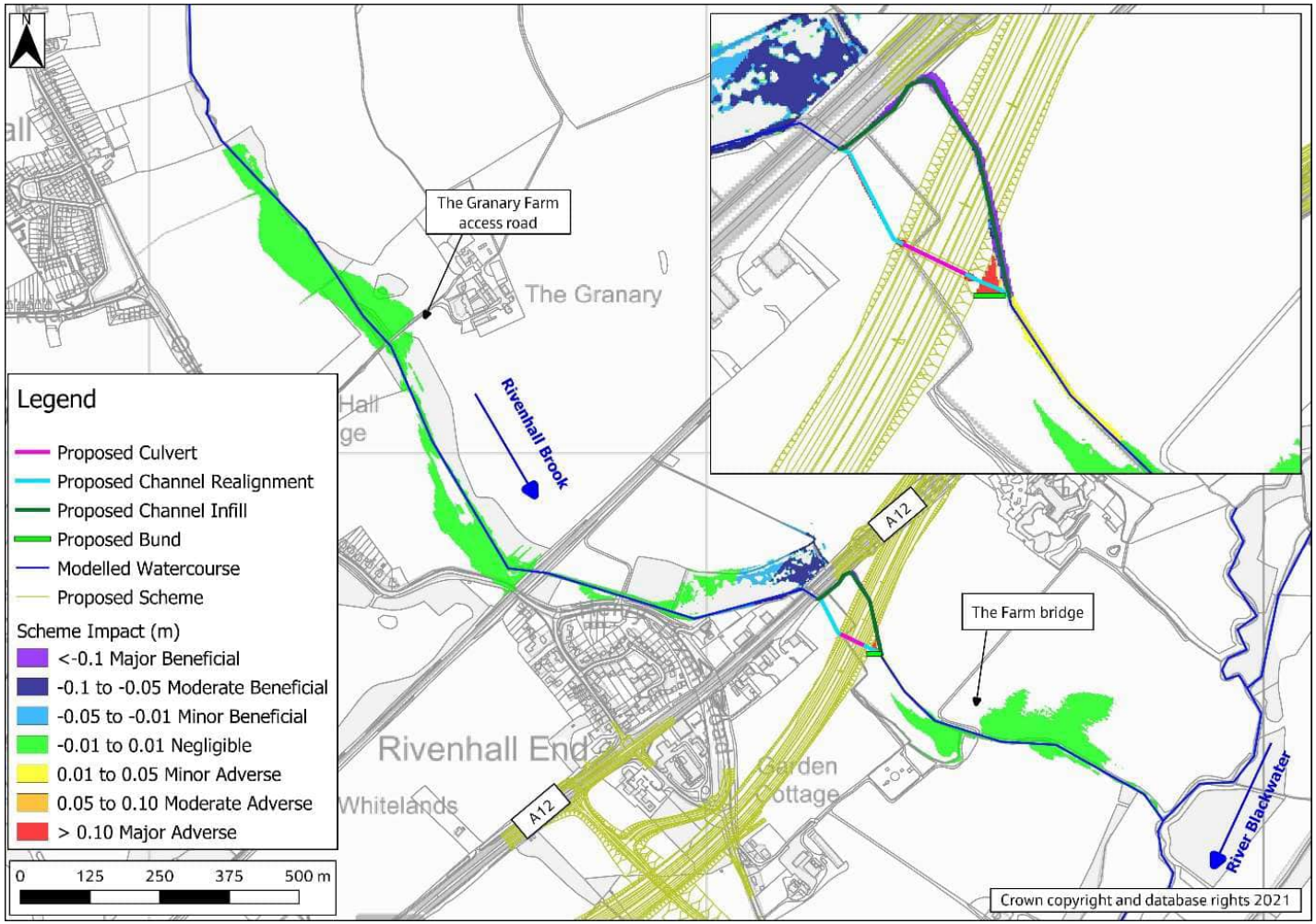
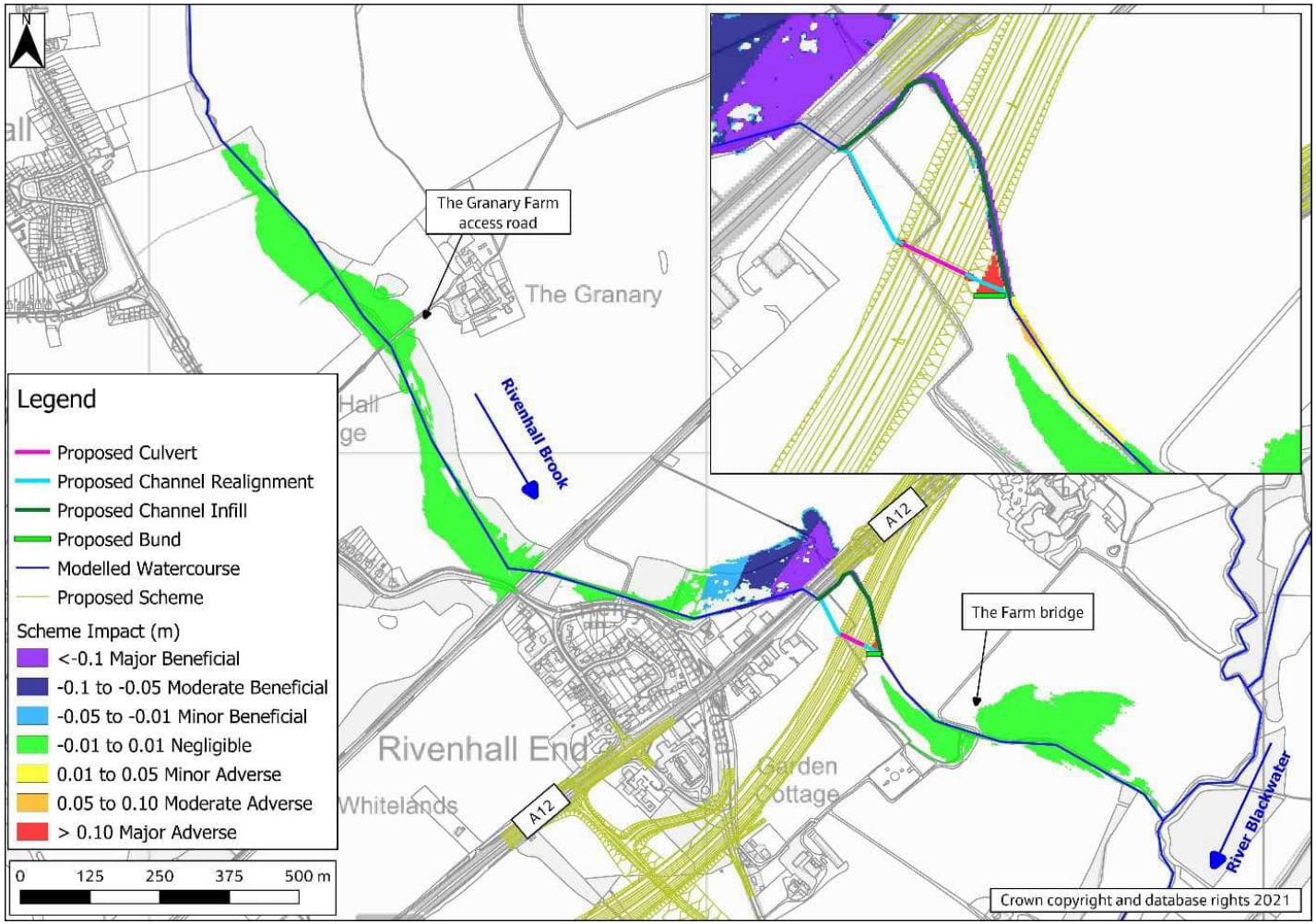


Plate D.6 Water level difference (with-scheme - baseline) for the 1% (1 in 100) AEP +38%CC event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVENHALL BROOK HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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ANNEXES

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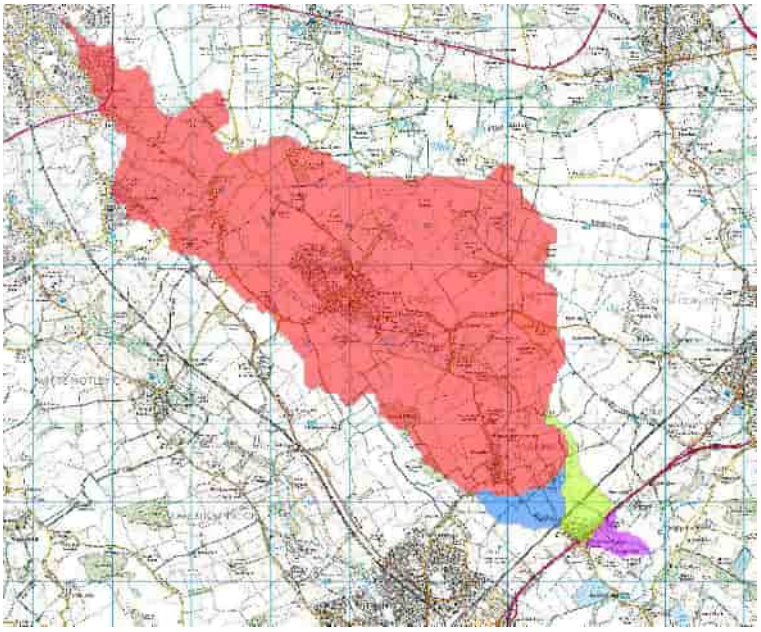
A.1 Supporting Information 20

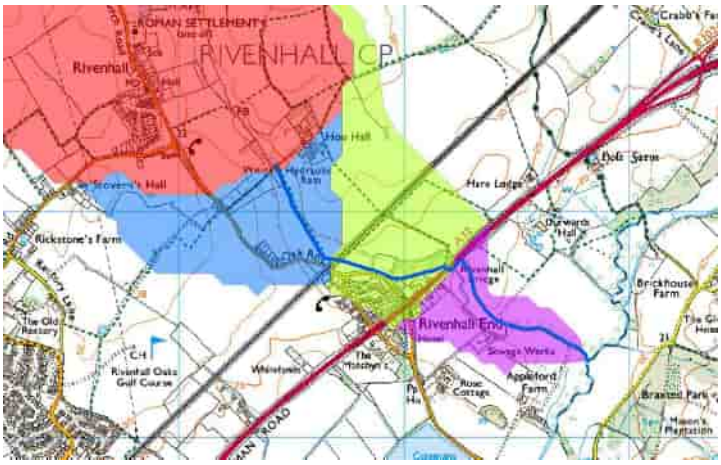
1 Introduction

1.1.1 This document is a supporting document to the Environment Agency's flood estimation guidelines. It provides a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This version of the record is for studies where flood estimates are needed at multiple locations.

2 Method Statement

2.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Approx. no. of flood estimates required • Peak flows or hydrographs? • Range of return periods and locations • Approx. time available	Highways England are planning Highway Improvements on the A12 from junction 19 to junction 25. A number of potential routes have been proposed. The current route of the road crosses the Roman River to the east of Marks Tey. The objective of the proposed study (hydrological assessment component) are: (i) to produce baseline and design hydrology for a river hydraulic model of Rivenhall Brook. Four locations along the watercourse have been requested by the hydraulic modellers for inputs into the hydraulic model. These reflect where there are river confluences or locations of crossings. A full hydrograph is required for each of the events 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 200 and 1,000-year design return periods). Climate change allowances – The climate change allowance for the River Blackwater Operational Catchment Area has been taken from the unpublished UKCP18 peak river flow climate change allowances. The upper climate change allowance for the River Blackwater Operational River Basin is 38% to the 2080s. 

2.2 Overview of the catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The catchment of the Rivenhall Brook covers an area of 17.32km^2 to the downstream model extent. It lies on a bedrock of London Clay Formation - Clay, Silt Sand and Gravel. In the headwaters soils are described as being slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The majority of the catchment has soils that are loamy and clayey soils with impeded drainage. The baseflow index is 0.391 with a standard percentage runoff of 44.6. Downstream of the existing A12, soils freely draining slightly acid loamy soils. The catchment is essentially rural and receives a low average annual rainfall of 565mm.

2.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – Version 8, September 2019

2.4 Gauging stations (flow or level)

2.4.1 (at the sites of flood estimates or nearby at potential donor sites).

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
Roman River	Bounstead Bridge		37021	TL984205	52.6	flat v crump weir	01/1965 - 08/1991
Blackwater	Stisted		37017	TL792242	139.2	flat v crump weir	01/1969 - present
Blackwater	Appleford Bridge		37010	TL844158	247.3	trapezoidal flume	01/1962 - present
River Ter	Crabbs Bridge		37003	TL786107	77.8	trapezoidal flume	01/1932 - present

2.5 Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	Other comments on station and flow data quality – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Bounstead Bridge	N/A					
Stisted	1969 - 2017	N	Y	N		AMAX data corrected for non-modular flows
Appleford Bridge	1962 - 2017	N	Y	Y		Irregular record. Exceptionally high flow in 2001 considered suspect
Crabbs Bridge	1963 - 2017	Y	Y	Y	N	AMAX data corrected for non-modular flows
Give link/reference to any further data quality checks carried out						

2.6 Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Reasons – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Stisted	Velocity area at high flows	N	
Appleford Bridge	High flow rated spillway	N	Rating accounts for mon-modular flow
Bounstead Bridge	N/A	N	
Crabbs Bridge	Theoretically rated	N	
Give link/reference to any rating reviews carried out		N/A	

2.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)					
Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies					
Other data or information (e.g. groundwater, tides)					

2.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes – there is nothing unusual about the catchment that would mean that the methods were not acceptable
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Locations have been selected at tributary inflows and road crossings
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	None
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	Both the statistical method and ReFH2 have been selected and the results compared, The catchment has been broken down into lateral catchments to the railway line, A12 and the downstream model extent. An additional flow estimate has been calculated for the downstream end of the model to allow flow consolidation.
Software to be used (with version numbers)	Peak flow data set v8 September 2019 WINFAP-FEH v4[1] / ReFH 2.3

[1] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

3 Locations where flood estimates required

3.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
RIV1	Rivenhall Brook	Upstream model extent	583400	217200	16.18	16.13
RIV2	Rivenhall Brook	Lateral catchment from upstream extent to the railway line crossing	583650	216750	N/A	0.46
RIV3	Rivenhall Brook	Lateral catchment from the railway line to A12 crossing	584200	216750	N/A	0.50
RIV4	Rivenhall Brook	Lateral catchment from the A12 to the downstream model extent and river confluence	584750	216400	N/A	0.23
TOT (total catchment to confluence)	Rivenhall Brook	Total river catchment to the confluence with the River Blackwater.	584750	216400	17.303	17.32
Reasons for choosing above locations						

3.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
RIV1	0.971	0.24	0.379	4.69	16.5	566	45.0	0.046	0.1491
RIV2	1.00	0.24	0.487	0.21	23.7	530	43.6	0.017	-
RIV3	1.00	0.24	0.445	0.55	16.9	565	33.0	0.094	-
RIV4	1.00	0.24	0.918	0.66	23.6	565	42.6	0.006	-
TOT	0.973	0.24	0.391	6.27	16.8	565	44.6	0.046	0.1478

3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Additional catchment area has been measured from Ordnance Survey 1:25,000 maps. An amendment has been made to the eastern side of the RIV1 FEH catchment area to incorporate the watercourse flowing eastwards at Lanham Wood. As the area adjustment was so minor, no adjustments were made to the other catchment descriptors. The existing A12 embankment will cause an obstruction to the catchment runoff and therefore the FEH boundaries for RIV3 and RIV4 have been amended to account for this. Lateral catchments have been calculated based on contributing areas between FEH catchment to the flow estimation points.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	Reference was made to the Geology of Britain Viewer [redacted]
Source of URBEXT	FEH web viewer
Method for updating of URBEXT	Calculation of UAF is based on the new UAF model implemented in WINFAPv4 and described in WINFAP 4 Urban Adjustment Procedures Issue 1.01

4 Statistical Method

4.1 Search for donor sites for QMED (if applicable)

Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	The Rivenhall Brook is not gauged. The Rivenhall Brook is a tributary of the River Blackwater which is gauged just downstream of the confluence (Appleford Bridge 37010) further upstream (Stisted 37017). Both of these catchments are larger than the study catchment at 247km^2 and 139km^2 respectively. The upper Blackwater catchment is chalk. The Roman River, to the east, is gauged at Bounstead Bridge (37017). Flow data is not available on the peak flows database due to poor quality. The River Ter is further away from the study catchment and is gauged at Crabbs Bridge (37003). The catchment area is more similar to the study catchment at 77.8km^2. The geology is the same as the study catchment being underlined by London Clay with an SPRHOST 41.76%.

4.2 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
37017	Rejected due to artificial transfer from Ely Ouse scheme, predominantly chalk in the upper catchment.	Not applicable				
37010	Rejected - groundwater-influenced catchment with major artificial influences.					
37017	No flow data					
37003	Accepted – nearby rural catchment with data suitable for QMED calculation.	AM	N	5.430	6.509	0.83
Which version of the urban adjustment was used for QMED at donor sites, and why? Note: The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8).						

4.3 Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average adjustment factor	
RIV1	DT	1.97	37003	8.0	0.40	0.93	N/A	N/A	1.83
TOT	DT	2.04	37003	8.1	0.40	0.93	N/A	N/A	1.89
Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?									
Which version of the urban adjustment was used for QMED, and why?						Calculation of UAF is based on the new UAF model implemented in WINFAPv4 and described in WINFAP 4 Urban Adjustment Procedures Issue 1.01			

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added.

When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why.

The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.

The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

4.4 Derivation of pooling groups

4.4.1 The composition of the pooling groups is given in the Annex. Several subject sites may use the same pooling group.

Name of group	**Site code from whose descriptors group was derived**	**Subject site treated as gauged?** (enhanced single site analysis)	**Changes made to default pooling group, with reasons** Note also any sites that were investigated but retained in the group.	**Weighted average L-moments**, L-CV and L-skew, (before urban adjustment)
Rivenhall total river catchment	TOT	N	7011 – Removed as short record 33054 – Removed as FARL < 0.95 26802, 27073, 33032, 26013, 26014 – Removed as BFIHOST > 0.65 49005, 36007, 27010, 53017, 28058 – Added to increase station years to 500	L-CV = 0.273 L-SKEW = 0.171
Notes Pooling groups were derived using the revised procedures from Science Report SC050050 (2008). The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.				

4.5 Derivation of flood growth curves at subject sites

Site code	**Method** (SS, P, ESS, J)	**If P, ESS or J, name of pooling group** (Error! Reference source not found.)	**Distribution used and reason for choice**	**Note any urban adjustment or permeable adjustment**	**Parameters of distribution** (location, scale and shape) after adjustments	**Growth factor for 100-year return period**
TOT	P	Rivenhall.feh	GL as best fit to national data	N/A	Location = 1.000 Scale = 0.281 Shape = -0.171	2.964
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters. Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010). Growth curves were derived using the revised procedures from Science Report SC050050 (2008).						

4.6 Flood estimates from the statistical method

Site code	**Flood peak (m^3/s) for the following return periods (in years)**									
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**200**	**1000***
RIV1	1.83	2.63	3.19	3.79	4.16	4.66	5.09	5.41	6.25	10.66
TOT	1.89	2.72	3.31	3.92	4.31	4.83	5.27	5.60	6.47	11.03
• 1,000 year flows derived by applying the ratio of the 1000:100 from the ReFH method. Ratio = 1.97										

5 Revitalised flood hydrograph (ReFH) method

5.1 Parameters for ReFH model

Site code	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
RIV1	CD	9.680	307.958	45.082	1.230
RIV2	CD	1.460	310.368	23.013	1.335
RIV3	CD	2.819	312.796	28.531	1.343
RIV4	CD	2.816	316.885	29.931	1.375
TOT	CD	11.364	316.885	48.907	1.326
Brief description of any flood event analysis carried out (further details should be given below or in a project report)		N/A			

5.2 Design events for ReFH method

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
RIV1	R	W	15.0	
RIV2	R	W	2.50	
RIV3	U	W	4.30	
RIV4	R	W	4.30	
TOT	R	W	18.0	
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?				

5.3 Flood estimates from the ReFH method

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	30	50	75	100	100 +38% CC	200	1000
RIV1	2.87	3.80	4.43	5.06	5.46	6.02	6.54	6.99	9.65	8.54	13.85
RIV2	0.22	0.32	0.39	0.46	0.50	0.56	0.61	0.64	0.88	0.76	1.25
RIV3	0.19	0.27	0.32	0.37	0.41	0.45	0.49	0.52	0.72	0.63	1.04
RIV4	0.08	0.12	0.14	0.16	0.17	0.19	0.21	0.23	0.32	0.27	0.45
TOT	2.69	3.54	4.11	4.69	5.05	5.56	6.04	6.45	8.90	7.87	12.69

6 Discussion and summary of results

6.1 Comparison of results from different methods

6.1.1 This table compares peak flows from various methods with those from the FEH Statistical method at example sites for two key return periods.

Site code	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	ReFH	Statistical	Ratio	ReFH	Statistical	Ratio
RIV1	2.87	1.83	1.57	6.99	5.41	1.29
TOT	2.69	1.89	1.42	6.45	5.60	1.15

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The statistical method is not considered appropriate for use on lateral catchment inflows. The ReFH2 has therefore been used to provide a consistent approach to flow estimation across the Rivenhall Brook catchment. Comparison of flow estimates by the 2 approaches show greater flows using the ReFH2 methods across the upstream and downstream river catchments.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The catchment areas have been more accurately determined using the OS mapping.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	Larger return periods become less reliable due to the level of extrapolation required. Recorded river flow records used in the development of the estimation methods are generally much shorter than the return period of interest.
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH **3** 12.5 or the factorial standard error from Science Report SC050050 (2008).	Confidence limits for statistical estimates of QMED at RIV_TOT for the total catchment area are 68% lower 1.4, upper 3.1 95% lower 0.9, upper 4.0
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	
Give any other comments on the study, for example suggestions for additional work.	

6.4 Checks

Are the results consistent, for example at confluences?	The comparison of 100 year flows at the downstream end of the model TOT ReFH2 Direct calculation – 6.45 m^3/s Sum of inflows – 8.38 m^3/s
What do the results imply regarding the return periods of floods during the period of record?	No recorded flows to compare
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	100 year growth factor for the whole catchment to the downstream model extent is 2.96 from the statistical method 2.40 from ReFH2 This is within the guidance
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	Ratio of 1,000 to 100 year flow at TOT the downstream of the model at the Blackwater confluence is 12.69/6.45 = 1.97
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	QMED specific runoffs are 1.55 from ReFH2 1.09 from statistical
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No earlier studies have been provided
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	

6.5 Final results

Target flows

6.5.1 Below are the target flows for the downstream end of the model using the critical catchment duration of 18 hours. The target flows at RIV1 is also shown, which are slightly higher than the d/s peaks. As such the modelled at the d/s modelling extent should be compared with both values, and in general, considered to be not be less than both values.

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+CC (100yr +cc)	0.5% (200-yr)	0.1% (1,000-yr)
TOT	2.69	3.54	4.11	4.69	5.05	5.56	6.04	6.45	8.90	7.87	12.69
RIV1	2.86	3.79	4.40	5.03	5.42	5.97	6.49	6.93	9.56	8.47	13.69

Model inflows

6.5.2 The ReFH2 model was run for each catchment with a critical catchment duration of 18 hours. The results below show the resultant peak flows for each sub-catchment.

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+CC (100yr +cc)	0.5% (200-yr)	0.1% (1,000-yr)
RIV1	2.86	3.79	4.40	5.03	5.42	5.97	6.49	6.93	9.57	8.47	13.69
RIV2	0.16	0.22	0.26	0.30	0.32	0.35	0.39	0.41	0.57	0.51	0.84
RIV3	0.16	0.21	0.25	0.29	0.31	0.34	0.37	0.40	0.55	0.49	0.80
RIV4	0.07	0.09	0.11	0.13	0.14	0.15	0.17	0.18	0.24	0.22	0.36

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	Yes, the inflow hydrographs were needed in the four sub-catchments. There were derived using the ReFH2 method. While deriving the model inflow, same storm duration is applied to all sub-catchments. For example, for the target flow at d/s modelling extent, a storm duration of 18hrs was used for all sub-catchments.

Acronyms

Abbreviation	Term
AM	Annual Maximum
AREA	Catchment area (km2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1989
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

Annex A

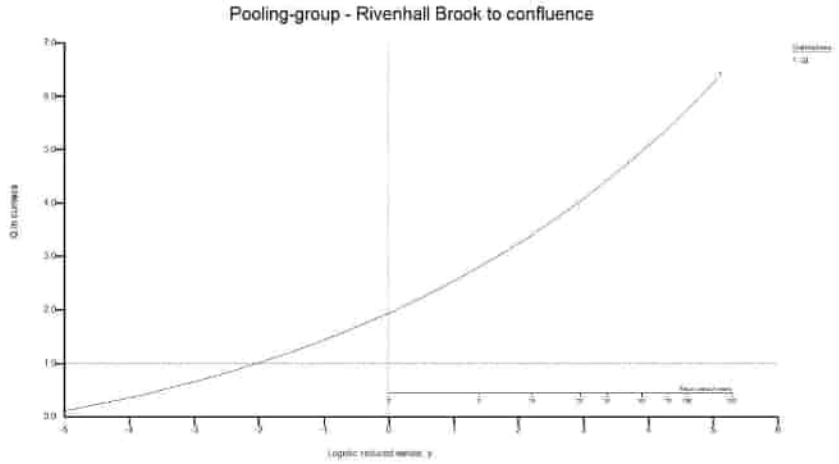
A.1 Supporting Information

A.1.1 Pooling group composition

Table A.1 Pooling Group Composition

Station	Distance	Years of data	QMED AMAX	L-CV	L-SKEW	Discordancy
36010 (Bumpstead Brook @ Broad Green)	1.336	51	7.5	0.372	0.184	0.951
41020 (Bevern Stream @ Clappers Bridge)	1.552	49	13.66	0.203	0.181	0.679
25019 (Leven @ Easby)	1.632	40	5.384	0.343	0.378	1.336
72014 (Conder @ Galgate)	1.747	50	16.465	0.233	0.162	0.143
36004 (Chad Brook @ Long Melford)	1.765	51	5.186	0.294	0.182	0.349
36003 (Box @ Polstead)	1.77	57	3.91	0.305	0.089	0.643
73015 (Keer @ High Keer Weir)	1.774	27	12.33	0.205	0.281	0.923
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	1.83	8	6.511	0.262	0.049	3.065
36007 (Belchamp Brook @ Bardfield Bridge)	1.866	53	4.63	0.371	0.119	1.035
27010 (Hodge Beck @ Bransdale Weir)	1.89	41	9.42	0.224	0.293	0.765
53017 (Boyd @ Bitton)	1.916	45	13.87	0.241	0.088	0.15
28058 (Henmore Brook @ Ashbourne)	1.935	13	8.838	0.188	-0.109	1.96
Total		485				
Weighted means				0.273	0.171	

Plate A.1 Rivenhall Brook Growth Curve



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVER BLACKWATER MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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0 Executive summary

0.1.1 This report describes the hydraulic modelling and the results obtained for the existing and proposed A12 highway crossing of the River Blackwater (NGR 585567 217695). The modelling and hydrology have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1-dimensional/2-dimensional (1D/2D) linked type using Flood Modeller software (Jacobs 2022) for the 1D domain and TUFLOW (BMT 2022) for the 2D domain and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 38% climate change (CC). Flow reconciliation was carried out to match the design hydrology. Model calibration was not undertaken as it had previously been completed by CH2M in 2017 using historic events that were larger than any more recent events. The model results were verified through comparison to Environment Agency (EA) flood maps. The model was then used to simulate baseline and with-scheme scenarios for the three design events. In addition, a with-scheme temporary works scenario (that included haul roads) was simulated for the 5% (1 in 20) and 1% (1 in 100) AEP events.

0.1.3 The with-scheme scenario involved minor widening of the highway embankment on the upstream side, larger widening on the downstream side and the extension of the existing bridge.

0.1.4 The analysis demonstrates that the proposed-scheme and its temporary works have negligible impact on fluvial flood risk at the River Blackwater, the former including an allowance for climate change and therefore no mitigation is proposed.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA, which took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) were:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater (this report)
- Domsey Brook
- Roman River
- Five minor watercourses were also modelled for the proposed-scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for the Middle Blackwater River to assess the baseline and with-scheme scenarios. This is a technical report, focused on the hydraulic modelling and catchment hydrology, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 At the A12 crossing of Middle Blackwater, the highway will be upgraded to three lanes per carriageway. This will require minor widening of the highway embankment on the north (upstream) side, plus a larger widening of the highway embankment on the south (downstream) side, and the

extension of the bridge under the highway. Full details of the with-scheme arrangements are given in Section 8.

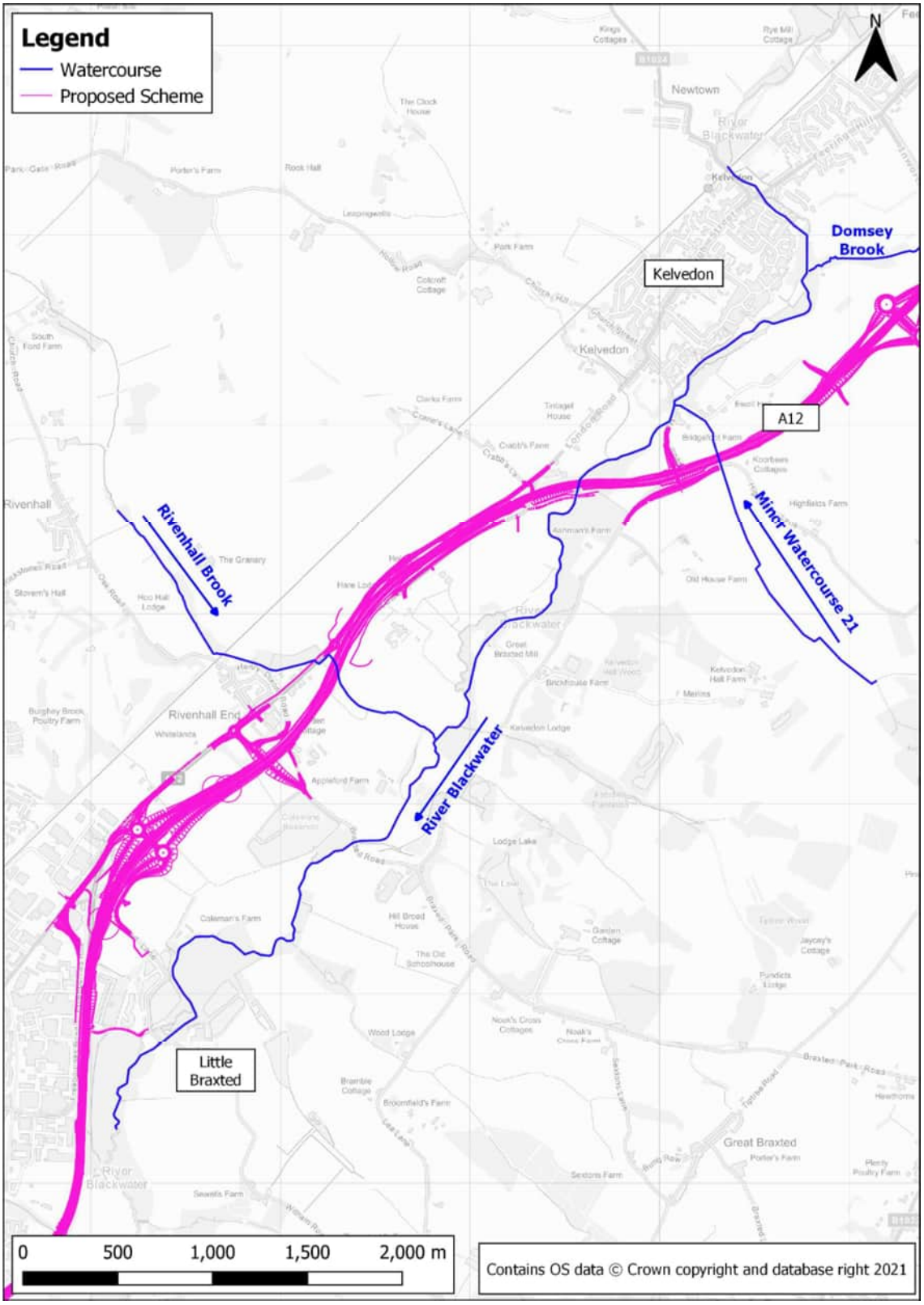
1.1.6 This report should be read in conjunction with the FRA.

1.2 Study area

1.2.1 The River Blackwater flows from the north of Braintree to Beeleigh where it joins the River Chelmer. The River Blackwater crosses the proposed-scheme once, approximately 1 km downstream of Kelvedon. The Middle Blackwater model represents approximately 7km of the river reach, extending from just north of Kelvedon to Little Braxted. Plate 1.1 shows the River Blackwater study area.

1.2.2 Rivenhall Brook, Domsey Brook and Minor Watercourse 21 (MWC21) are represented in the Middle Blackwater model as point or lateral inflows to the River Blackwater. Specific modelling of those watercourses has been carried out as part of the A12 Flood Risk Assessment and is reported separately (National Highways 2022).

Plate 1.1 Middle Blackwater study area



2 Input data

2.1.1 The data used to construct the hydraulic model for the Middle Blackwater is summarised in Table 2.1.

Table 2.1 Data used to build the Middle Blackwater hydraulic model

Data	Description	Source
LiDAR	1m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection and Ranging) data. Used to represent the topography for the whole model extent	Environment Agency, 2012
OS Mastermap	Ordnance Survey Mastermap data	National Highways
Hydraulic model for River Blackwater	One-dimensional/Two-dimensional (1D/2D) model built with Flood Modeller and TUFLOW software - BWT_Middle_Model_Domain1&2_2016-07-15.DAT This version of the model was produced by CH2M (2017)	Environment Agency

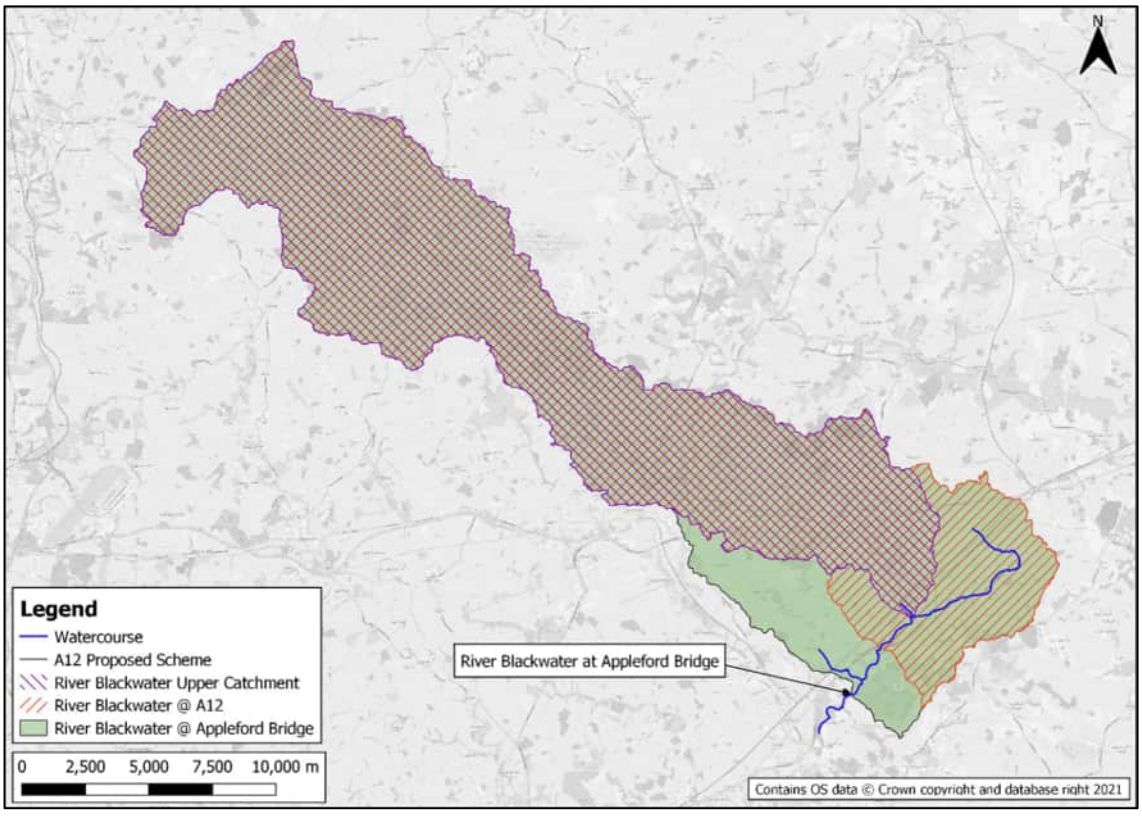
3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of the Middle Blackwater for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event is also required including an allowance for climate change (plus CC).

3.1.2 Plate 3.1 shows the Middle Blackwater catchment extent along with the flow gauge location within the modelled reach.

Plate 3.1 Middle Blackwater catchment extent



3.2 Background

3.2.1 Hydraulic modelling of the full River Blackwater and River Brain catchments was previously undertaken by CH2M in 2017 as part of a flood mapping exercise. The CH2M (2017) numerical hydraulic model was provided for use in this assessment. After a review of the CH2M hydrology used in the hydraulic model, it was decided to retain some of the original ReFH1 inflow units for the Middle Blackwater hydraulic model but also to update the model for this assessment, to ensure the hydrology analysis met with the latest methods and guidances. As the 2017 hydraulic model

was truncated from the top to make it suitable for the flood study of the A12 project extent, a new ReFH1 inflow unit was created near the upstream extent of the modelled reach considered in this study. In addition, the peak flow estimates were also updated using the latest methods of flood flow analyses and incorporated the most up-to-date hydrometric data available post-2017 assessment.

3.2.2 The River Blackwater at Appleford Bridge (37010) gauging station is located within the present study hydraulic model extent. Therefore, the hydrometric data available at this gauge has been reviewed and included in this analysis where appropriate. For full details of the estimation of design peak flows, derivation of hydraulic model inflow hydrographs and background hydrology refer to Appendix A.

3.3 Hydrological considerations

3.3.1 The River Blackwater is located in south east England. It has a catchment area of approximately 350 km² to Maldon where it flows into the North Sea. The river is a relatively rural catchment (arable, with areas of grassland and woodland) with urban areas including Braintree, Coggeshall, Witham and Maldon. The River Blackwater has five flow / level gauges in its catchment

- Copford Hall (River Pant)
- Stisted
- Appleford Bridge
- Guithavon Valley (on the River Brain) and
- Langford

3.3.2 The Appleford Bridge gauge (37010) is located within the modelled reach of the River Blackwater. The River Blackwater flow regime is identified (National River Flow Archive data sheet) as being affected by groundwater abstractions and intermittently affected since 1971 by the Ely/Ouse Transfer Scheme pumping.

3.3.3 The predominant land use in the catchment is arable, with areas of grassland and woodland. The geology of the catchment has been assessed as being composed of predominantly London Clay with chalk in the headwaters, overlain with boulder clay and some sand and gravels. The soils within the catchment have been assessed using Landis online mapping (Landis 2020) as predominantly lime rich loamy and clayey soils with impeded drainage in the upper reaches and freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The catchment to the River Blackwater at Appleford Bridge gauge (37010) has a BFIHOST value of 0.475 and a SPRHOST value of 40.1%.

3.4 Model inflow units

3.4.1 As noted in Section 3.2 a new ReFH1 inflow unit has been created at the upstream extent of the modelled reach for the River Blackwater upper catchment to just upstream of the Domsey Brook confluence at TL 86750 18850. The new ReFH1 inflow unit allows assessment of flood risk in the vicinity of the proposed-scheme and reduce model computation time. The ReFH1 inflow boundary units between the upper catchment and the River Blackwater at Appleford Bridge gauge (37010) (that is, associated with Domsey Brook, Rivenhall Brook and residual sub-catchments) were retained from the CH2M (2017) study. The ReFH1 inflow units were used within the hydraulic model to derive the inflow flood hydrograph shape. The use of ReFH1 boundary units was assessed and judged to be acceptable for deriving hydrograph shape for these catchments. This was based on the assessment that there is limited floodplain storage within the modelled reach and as there appears to be limited catchment change post the 2017 study. This was also supported by the results of the ReFH2.3 hydrograph shape sensitivity analysis undertaken as part of the current study within the Middle Blackwater model. The results of the sensitivity analysis indicated that for the 1% (1 in 100) AEP event the ReFH1 hydrograph shape (scaled to the final peak flow estimates) produced slightly conservative flood extent outline at the A12 crossing location of the River Blackwater in comparison to the corresponding flood extent when the ReFH2.3 flood hydrograph shape was adopted. For further details of the sensitivity analysis refer to Appendix A.

3.5 Peak flow derivations

3.5.1 The design peak flow estimates were updated as part of this study using the FEH (Flood Estimation Handbook) Statistical method, using both simple Pooling Group and Enhanced Single Site (ESS) analyses, and ReFH2.3 rainfall runoff method.

3.5.2 For estimating peak flows using the FEH Statistical method, the hydrometric data available at the Appleford Bridge gauge (37010) from 1962 to present-days, was reviewed. According to the NRFA (National River Flow Archive) website, this is a peak flow rated gauge and its flood record is classified as suitable for both QMED estimation and pooling group analysis. There is likely to be some uncertainty associated with the higher flow estimates for this gauge, but the gauge is believed to perform satisfactorily at QMED. Therefore, the index flood (QMED) was derived for the River Blackwater at Appleford Bridge using the full AMAX data recorded at the gauge. Standard Pooling and Enhanced Single-Site (ESS) analyses were undertaken to derive the flood growth curve at the gauging station and the statistical peak flow estimates for up to and including the 1% (1 in 100) AEP event. The latter were calculated by simply multiplying the index flood with the corresponding growth factors. The ReFH2.3 method was also used (with default catchment descriptors) in order to derive peak flow estimates.

3.5.3 After reviewing the growth curves and peak flow estimates produced by these methods it was decided to adopt the steeper growth curve produced by the ESS analysis method. A figure showing the flood frequency curves from different method for the River Blackwater at Appleford Bridge is provided in Appendix A. For consideration of rarer events the Environment Agency guidance was followed which indicates that these flows can be estimated by multiplying the FEH Statistical 1% (1 in 100) AEP flow by the ratio of the ReFH2.3 derived T-year peak flow, QT (for T>100-years) and the ReFH2.3 derived 1% (1 in 100) AEP flow, Q100.

3.5.4 At the other flow estimation point, the A12 crossing of the River Blackwater, the AEP peak flows were derived using the following approach:

- QMED was derived using catchment descriptors (CDs); which was then adjusted using the River Blackwater at Appleford Bridge gauge as the donor. The peak flow estimates for up to and including the 1% (1 in 100) AEP event were derived by multiplying the adjusted QMED by the flood growth curve derived from the ESS analysis at the River Blackwater at Appleford Bridge gauge.
- The ratio of the ReFH2.3 estimates of the QT/Q100 was again used to extend the flood frequency curve from the 1% (1 in 100) AEP event to rarer events.

3.5.5 The peak flows estimated from the above method at the two target locations (Appleford Bridge gauge and A12 crossing), are presented in Table 3.1.

Table 3.1 River Blackwater (Middle) peak flows (m^3/s) for range of AEP events at the two target locations

Inflow	Peak Flow (m^3/s)										
	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	1% AEP +38% CC	0.5% AEP	0.1% AEP
	1 in 2	1 in 5	1 in 10	1 in 20	1 in 30	1 in 50	1 in 75	1 in 100	1 in 100 +CC	1 in 200	1 in 1000
River Blackwater @ Appleford gauging station	13.2	18.9	22.9	27.2	29.8	33.3	36.3	38.5	53.2	46.2	71.4
River Blackwater @ A12	12.7	18.2	22.1	26.2	28.7	32.1	35.0	37.2	51.3	44.5	68.8

3.6 Model runs and the critical storm duration

Introduction

3.6.1 For the Middle Blackwater hydraulic model, model runs were undertaken using the River Blackwater critical storm duration and rainfall for all catchments/sub-catchments. Sections of the A12 run in close proximity to the River Blackwater within the model extent with the A12 crosses the River Blackwater to the south of Kelvedon at TL 85550 17650.

Model simulation

3.6.2 For the Middle Blackwater hydraulic simulations, the model requires a total of four inflows, including three point inflows (at the top end of the River Blackwater Middle model, for the Domsey Brook and Rivenhall tributaries) and one lateral inflow for the residual catchment, as described below:

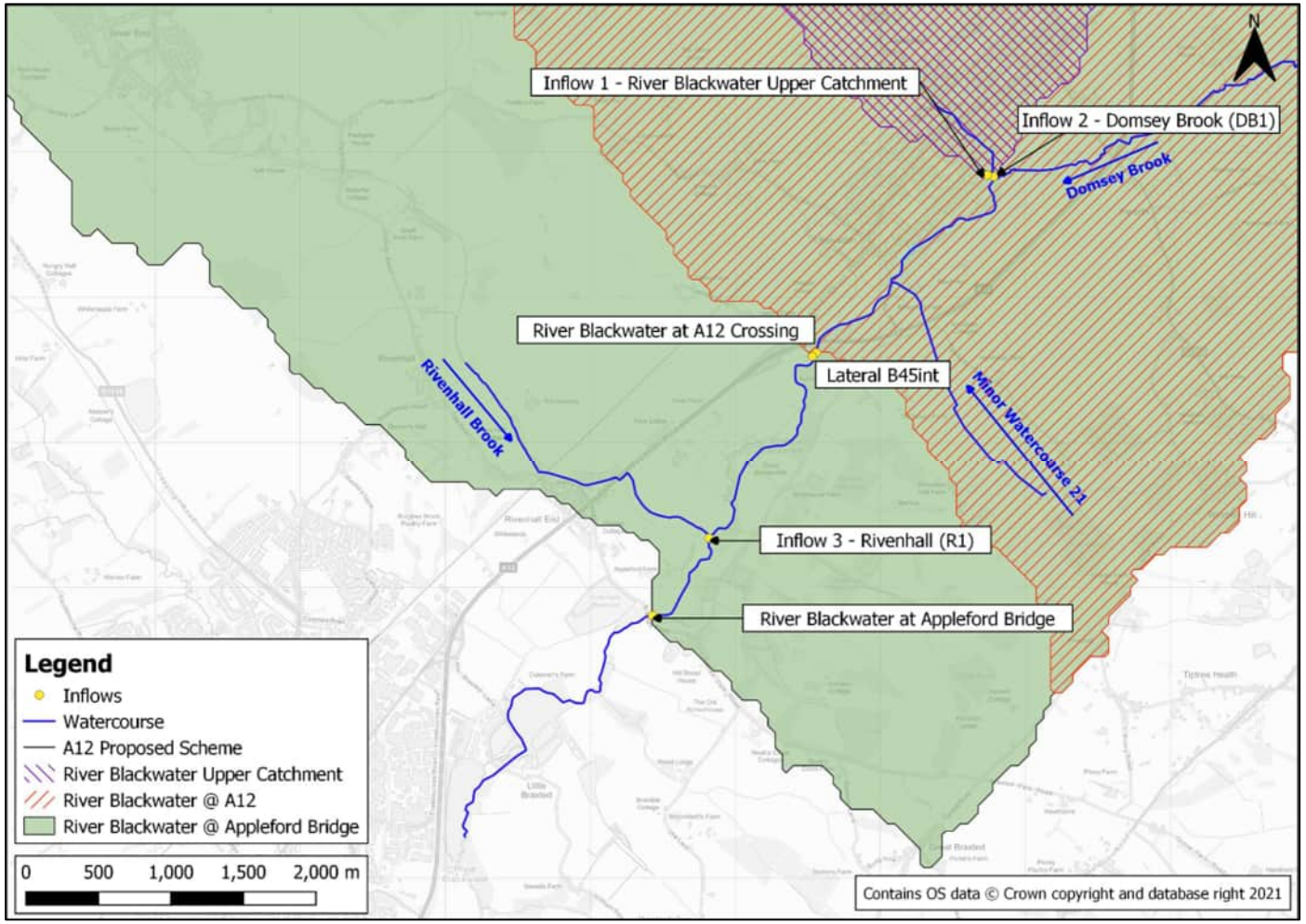
- Inflow 1 – River Blackwater Upper catchment (new ReFH1 inflow location)
- Inflow 2 – Domsey Brook tributary - DB1 (as per CH2M design hydrology)
- Inflow 3 – Rivenhall Brook tributary - R1 (as per CH2M design hydrology) and
- Lateral – B45int (as per CH2M design hydrology) for the residual catchment between the River Blackwater Upper catchment / Domsey Brook inflow and the River Blackwater at Appleford Bridge gauge (the Middle Blackwater downstream modelling extent). Note that Minor Watercourse 21 is included in this catchment

3.6.3 For the Middle Blackwater hydraulic model runs, the adopted critical storm duration is 29.7 hours, which was derived using the ReFH1 unit for all the catchment up to the Appleford Bridge gauge (default FEH catchment descriptors). In order to assess the potential worst-case flood scenarios from the River Blackwater at the proposed-scheme, the model has been run using the same 29.7 hours critical storm duration and rainfall for all sub-catchments/model inflows.

Model inflow locations

3.6.4 Plate 3.2 presents the model inflow locations along the modelled reach.

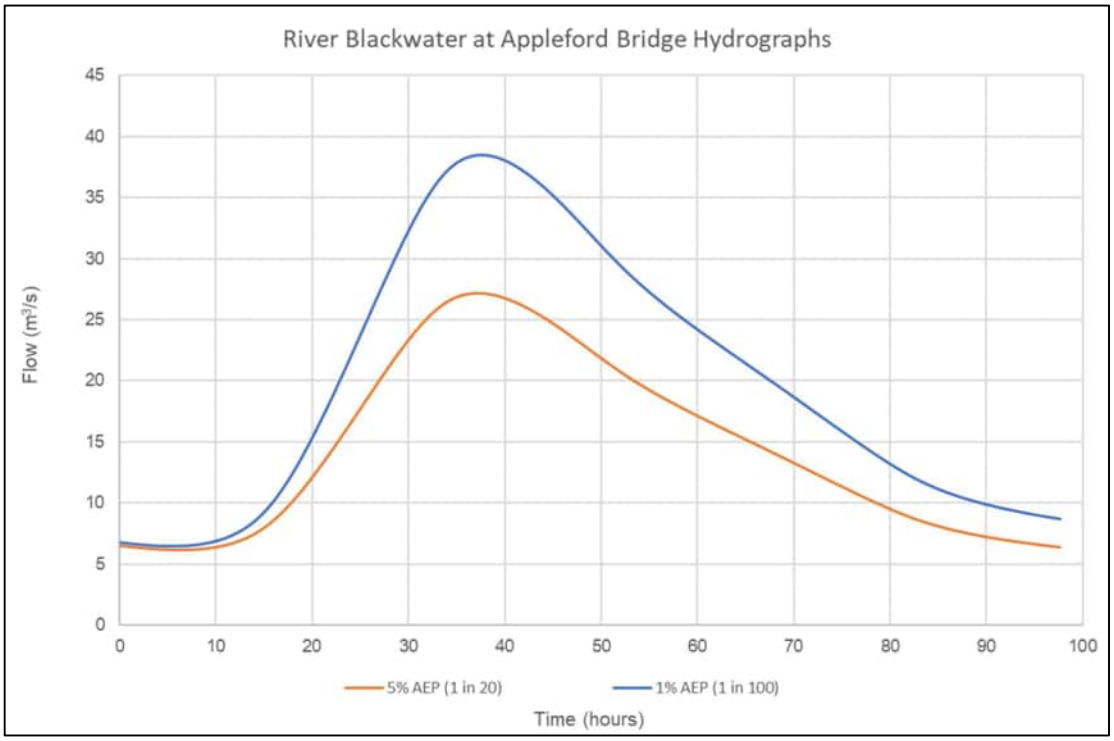
Plate 3.2 Schematisation of the Middle Blackwater model inflows



Inflow hydrographs

3.6.5 Plate 3.3 presents the 5% (1 in 20) and 1% (1 in 100) AEP design flood hydrographs of River Blackwater at Appleford Bridge gauge for the 29.7 hours critical storm duration scaled to the adopted peak flow estimate at the gauge.

Plate 3.3 River Blackwater at Appleford Bridge hydrographs



3.7 Climate change

3.7.1 A climate change allowance for peak river flows was obtained from the latest government guidance at the time of assessment (Environment Agency 2021). This produced an uplift allowance of 38% (the Combined Essex Management catchment, higher 2080s scenario) which was applied to the 1% (1 in 100) AEP design event giving a peak flow of 53.2m^3/s for the River Blackwater at Appleford Bridge gauge (37010).

3.8 Flow reconciliation

3.8.1 A flow reconciliation exercise was carried out by comparing the peak modelled flow at the Appleford Bridge Gauging Station which is at the downstream end of the model (flow from node “BWT25480u” and PO lines) and at the A12 crossing (node “BR_BWT22880u”). The Appleford Gauge was adopted as the primary location for flow reconciliation. A number of scaling factors were tested within the hydraulic model. Based on this analysis a scaling factor of 0.75 was adopted for the 5% (1 in 20) AEP event and 0.7 for the 1% (1in 100) AEP and 1% (1 in 100) AEP

+38%CC events and applied to the ReFH1 inflow units. Table 3.2 shows the comparison of the results using the adopted scaling factors for the target AEP events and the flow difference at Appleford Bridge Gauge.

Table 3.2 River Blackwater flow reconciliation results

AEP event	Modelled flows at the Appleford Gauging Station (m^3/s)	Target flows at the Appleford Gauging Station (m^3/s)	Flow Difference at the Appleford Gauging Station (%)
5% (1 in 20)	27.8	27.2	+2.2
1% (1 in 100)	39.5	38.5	+2.5
1% (1 in 100) +38%CC	52.7	53.2	-0.9

3.9 Summary

3.9.1 Design peak flow estimates for the River Blackwater were derived using the FEH methodology applying the available hydrometric data at the gauge, catchment descriptors available from the FEH Web Service; and adopting the climate change allowance as per the latest EA guidance. The hydrograph shape was adopted from the previous model which had used ReFH1; following a sensitivity analysis which indicated that the ReFH1 produces slightly conservative flood extent in comparison to that from the ReFH2.3 hydrograph shape (for further details refer to Appendix A). Design inflow hydrographs to be used in the numerical hydraulic model were derived for the River Blackwater and all sub-catchments using the River Blackwater ReFH1 critical storm duration to the Appleford Bridge gauge.

4 Baseline modelling

4.1 Methodology

4.1.1 The hydraulic model uses a linked 1D/2D schematisation, where the river channel is represented as the 1D component and is linked to the floodplain, represented by a 2D domain. The model was modified from a model provided by the Environment Agency that utilised a 1D/2D schematisation for simulating low magnitude events and a 2D only version for simulating large magnitude events. The 1D/2D model was truncated to cover the required study area and updated to run design flood events. The river modelling package Flood Modeller Pro (version 4.5) was used for the 1D component and TUFLOW (version 2018-03-AE-iDP-w64) for the 2D component.

4.2 Existing model

4.2.1 As mentioned, the model reported here is an upgraded version of that produced by CH2M in 2017 for an Environment Agency flood risk mapping study. Although the modelling is deemed generally suitable for this study, the model schematisation and resolution required updating to represent the interactions between the River Blackwater and the new alignment of the A12 Highway more accurately. Numerical stability issues have also been noted during the review of the existing model, which uses a simplified “2D only” version for simulations of events of a larger magnitude than the 1% (1 in 100) AEP event.

4.3 Model updates

4.3.1 The focus of the baseline model updates has been to stabilise the existing model so that all the required AEP events can be simulated and to increase the resolution of the results. Most of the numerical non-convergences of the incoming 1D/2D model occur at Kelvedon or upstream of it. The model was therefore truncated, removing approximately 12km from the upstream end which removed the instabilities and optimised model run times and file sizes.

4.3.2 Model resolution has been increased in the 2D domain with cell size reduction from 10m to 4m to better represent the proposed-scheme arrangement at the River Blackwater A12 crossing. The simulation timestep was therefore reduced: from 5s to 2s for the 2D model and from 2.5s to 1s for the 1D model.

4.3.3 A number of additional changes were made to improve stability further. Some simulation parameters were slightly changed as described below but kept within a reasonable range following guidance provided by the software developer:

- Minor localised model updates were made to the 1D/2D linkages.

- The boundary viscosity factor (2D model) was retained from the incoming model. It has a value of 3 (default value is 0).

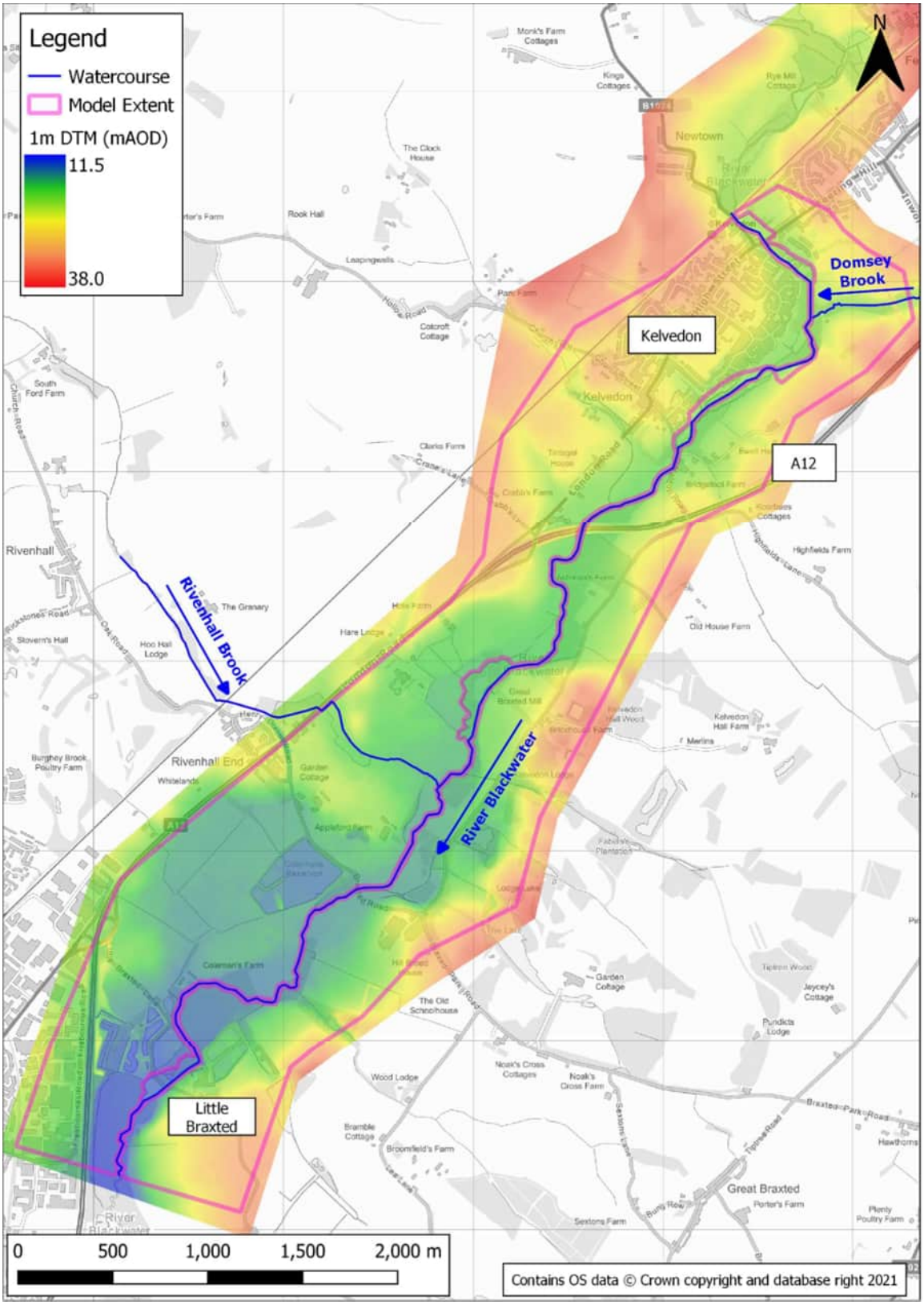
4.4 Watercourse schematisation – 1D domain

Model extent

4.4.1 The 1D model extent was trimmed by removing approximately 12km from the upstream end. The new model extent ensures the area of interest around the A12 crossing is sufficiently far from upstream and downstream boundaries to ensure they do not affect the results at the A12 crossing.

4.4.2 Plate 4.1 shows the extent of the model domain.

Plate 4.1 Middle Blackwater baseline model schematisation



In channel geometry

4.4.3 Most of the in-channel geometry, including the hydraulic structures, was left unchanged from the incoming model as it adequately represents the River Blackwater. Two cross-sections (BWT20890d and BWT25063) were modified with bank markers to improve conveyance.

4.4.4 A number of islands standing between the River Blackwater and its bypass channels (or mill races) were ignored in the incoming model. This was seen to be a key source of numerical instability in the incoming model. The model was therefore updated at three locations; Station Road bypass channel, Easterford Mill upstream of the A12 crossing and Great Braxted Mill downstream of the A12 crossing (Plate 4.2). At these locations 1D cross-sections were extended using LiDAR, to include representation of the islands, which improved the model performance.

4.4.5 At the Greys Mill bypass channel, the 1D schematisation was seen to cause numerical instability due to volume exchanges between Middle Blackwater and its low-lying bypass channel. These exchanges are primarily controlled by the mill sluice gate and weir structures. Both watercourses were originally modelled as 1D parallel channels, with a single spill unit for lateral volume exchanges. Under high flows, this arrangement was observed to overly constrict cross flow between the two reaches, leading to numerical instabilities. The necessity to improve the schematisation in this area is driven by its proximity to the A12 crossing, which is situated 600m downstream from Grey's Mill. The low-lying bypass was therefore removed from the 1D domain and represented in the 2D domain with a gully line, oriented along the mill race alignment, next to the River Blackwater on its south side. This allowed the correct lateral spillage between the two reaches to be modelled via a 1D/2D link. The bed elevation of the bypass channel was obtained from the 1D cross-sections and the terrain profile was used to represent the required bank-top levels.

4.4.6 Another source of instability from the incoming model was identified at several locations where very narrow floodplain "corridors" were schematised within the 2D domain, conveying limited flows in 2D. Therefore, the incoming model was updated using 1D extended cross-sections to replace those 2D corridors in order to simplify the schematisation and improve the model convergence.

In channel hydraulic friction

4.4.7 Review of the incoming model showed that a uniform Manning's 'n' roughness value of n = 0.04 has been applied within all cross sections. According to Chow (1959), a Manning's roughness of n = 0.04 accounts for clean, winding channels with some pools and shoals. Based on inspection of the watercourse from the available aerial and street-side photography, the in-channel hydraulic roughness values are considered to be acceptable. Therefore, the hydraulic roughness (Manning's 'n' coefficient) from the incoming model was not modified.

4.4.8 As described above, there are a few locations where cross-sections have been extended to consider the floodplain in 1D or to include local islands, which cover a variety of land use, including buildings, urban area and thick vegetation. A general out-of-bank roughness value of n = 0.06 was adopted in these locations.

In channel hydraulic structures

4.4.9 Review of the incoming model showed that the general schematisation of the in-channel structures appeared correct. Information for detailed checking of dimensions was not available. The structures were therefore not modified from the incoming model and were assumed to be correct.

4.4.10 A detailed structure schedule is provided in Appendix B.

Boundary conditions – 1D domain

4.4.11 The 1D model downstream boundary comprises a pair of stage-time boundary units. These were not modified from the incoming model and are assumed to be correct. As per the incoming model, a constant stage of 13.19 mAOD (node LBRAX296d) and 13.7 mAOD (node BWT27615d) is used on River Blackwater and Little Braxted Mill Race, respectively. Sensitivity testing has shown that these boundary conditions do not affect model result at the area of interest (see Section 6.4).

4.4.12 The upstream model inflow boundary was relocated to the railway crossing (north of Kelvedon), which is 2.6 km upstream from the A12 crossing, where the model topography allows accurate schematisation of the required inflows. Lateral inflows remain unchanged from the incoming model.

4.4.13 The 1D boundary conditions applied to the model are described in Table 4.1.

Table 4.1 1D boundary conditions

Type of Boundary	Node Labels	Description
QTBDY (Flow-Time Boundary)	BWT20348R	QT boundary. Upstream point inflow applied on River Blackwater at Kelvedon Railway crossing.
QTBDY	DB1	QT boundary representing ReFH1 data. Lateral point inflow applied on River Blackwater at Domsey Brook confluence.
QTBDY	B45int	ReFH1 boundary. Lateral distributed inflow split evenly at two locations: downstream of Ewell Hall Chase Bridge at Grey's Mill, where Minor Watercourse 21 discharges, and upstream of Great Braxted Mill bypass channel.
QTBDY	R1	QT boundary representing ReFH1 data.

		Lateral point inflow applied on River Blackwater at Rivenhall Brook confluence.
HTBDY (Head-Time Boundary)	LBRAX296d	Head Time downstream boundary. Constant Stage of 13.19 mAOD applied to River Blackwater just downstream of Little Braxted Lane. Sensitivity testing has shown that this boundary assumption does not affect model results in the area of interest (see Section 6.4).
HTBDY (Head-Time Boundary)	BWT27615d	Head Time downstream boundary. Constant Stage of 13.70 mAOD applied to Little Braxted Mill Race just downstream of Little Braxted Lane. Sensitivity testing has shown that this boundary assumption does not affect model results in the area of interest (see Section 6.4).

4.5 Floodplain schematisation – 2D domain

Floodplain topography

4.5.1 The 2D active area of the original model covered approximately 18.5 km². It has been reduced to an area of 7.2 km² to optimise the run time and focuses on the model updates in the area of interest. The topography is represented using a 4m resolution square grid. The levels for the grid cells are based on a Digital Terrain Model (DTM) derived from 1m resolution LiDAR data flown in 2012.

4.5.2 Only one modification has been made to the DTM, which was to represent Colemans and The Willows Reservoirs. Polygons were used to set the reservoirs as full at the start of the simulation. The incoming model representation had variable levels across the reservoir surfaces due to the LiDAR return being affected by the water surface. The correction enforces a flat surface. The assumed levels of 14.5 mAOD for Colemans Reservoir and 13.5 mAOD for The Willows Reservoir have been estimated from inspection of LiDAR levels at the reservoir perimeter.

Floodplain hydraulic friction

4.5.3 Hydraulic roughness coefficients are applied over each grid cell of the 2D domain as shown in Table 4.2, depending on land use taken from OS Mastermap data. These Manning's n values are deemed acceptable and have not been changed from the incoming model.

Table 4.2 Manning's 'n' coefficients – 2D domain

Land use	MasterMap feature code	Manning's 'n'
Default	1	0.04
Buildings	10021	0.5

Land use	MasterMap feature code	Manning's 'n'
General surface, residential yards	10053	0.040
General surface, step	10054	0.025
General surface, grass, parkland	10056	0.030
Water inland	10089	0.035
Slope	10096	0.040
Natural environment (coniferous/non coniferous trees) heavy woodland and Forest	10111	0.100
Roads tracks and paths - manmade	10119	0.020
Roads tracks and paths, tarmac or dirty tracks	10123	0.025
Rail	10167	0.050
Roads tracks and paths, tarmac	10172	0.020
Roads tracks and paths (roadside) pavement	10183	0.020
Structures, roadside structures	10185	0.030
Pylon	10193	0.040

Boundary conditions – 2D domain

4.5.4 The downstream HQ (stage-flow) boundaries has not been modified and assumed correct from the previous model. Sensitivity testing has shown that these boundary conditions do not affect model result at the area of interest (see Section 6.4).

1D/2D linkage

4.5.5 In the original model, the island between the Easterford Mill race channel and the River Blackwater was not modelled. Glass walling occurred causing incorrect water levels and instabilities. To improve the model, the link between the 1D and the 2D domains has been re-schematised at Easterford Mill race, where the two channels were modelled parallel to each other in 1D.

Bank-top elevations

4.5.6 Bank-top levels alongside River Blackwater were left unchanged for most of the model. However, there were some discrepancies between LiDAR and the bank-top levels used in the incoming model in the vicinity of the A12 crossing. A refinement of the bank-top values has therefore been carried out in the area of interest using the LiDAR data.

4.5.7 The 2D floodplain around the A12 crossing has been improved from the incoming model with the addition of 2D gully lines to represent the small drains discharging into River Blackwater between Grey's Mill and Great Braxted Mill. LiDAR data was utilised to set the invert levels of those drains.

4.5.8 Plate 4.3 shows the locations where the top of bank representation of River Blackwater as well as minor drains discharging into River Blackwater were updated with LiDAR data in the vicinity of the A12 crossing.

Plate 4.2 Middle Blackwater 1D geometry updates from incoming model

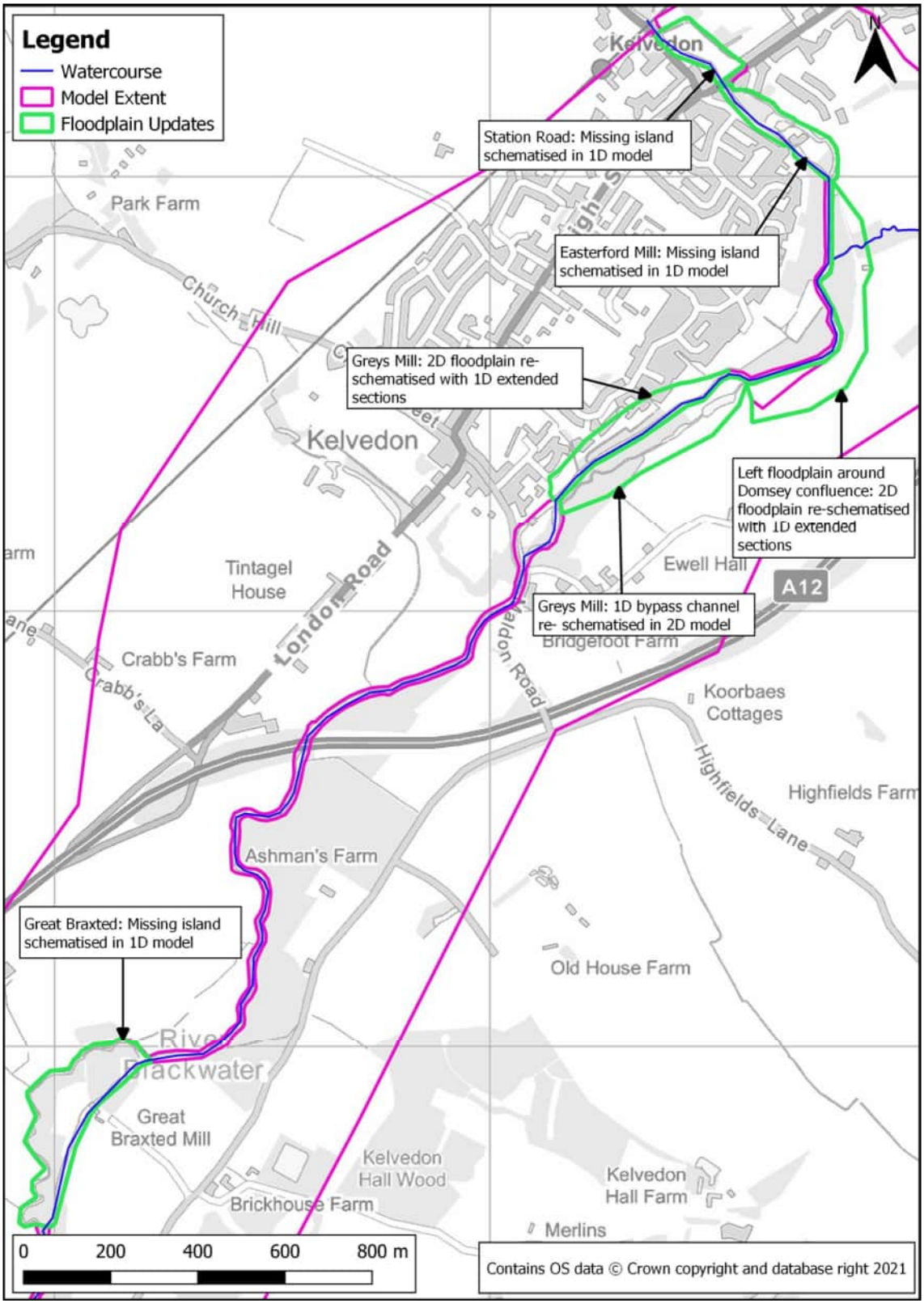
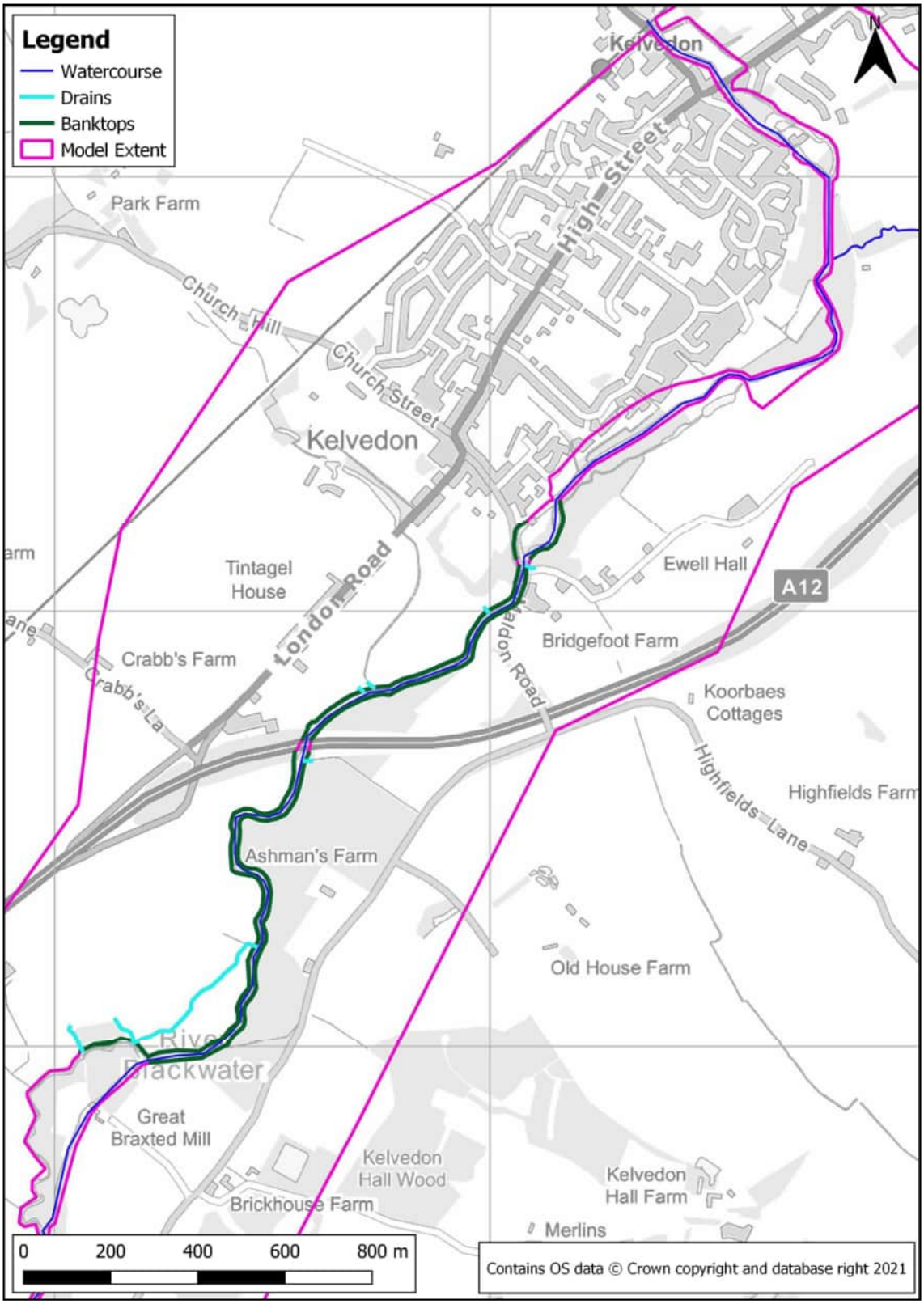


Plate 4.3 Middle Blackwater bank-top updates from incoming model



5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model.

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope.

Table 5.1 Modelled events

Scenario	AEP Event/Return Period		
	5%	1%	1% +38%CC
	(1 in 20)	(1 in 100)	(1 in 100 +CC)
Baseline	X	X	X
With-scheme	X	X	X
With-scheme and temporary works	X	X	
Roughness Sensitivity (in-channel, floodplain, and structures)		X	
Hydrological Inflow Sensitivity		X	
Downstream Boundary Sensitivity (1D and 2D)		X	

6 Model proving

6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution, within a specified number of iterations, for each model time step. The convergence of the 1D model was checked and is within tolerance for all simulated events except for the 1% (1 in 100) AEP +38%CC simulation. For this event, in both baseline (Plate 6.1) and with-scheme scenarios (Plate 6.2), convergence plots feature spikes of poor model convergence on either side of the hydrograph peak. The location of these issues has been identified to be at model node COAL239u, which is remote from the A12 crossing. The 1D model convergence is therefore deemed acceptable.

Plate 6.1 1D model convergence – 1% (1 in 100) AEP +38%CC event for the baseline scenario

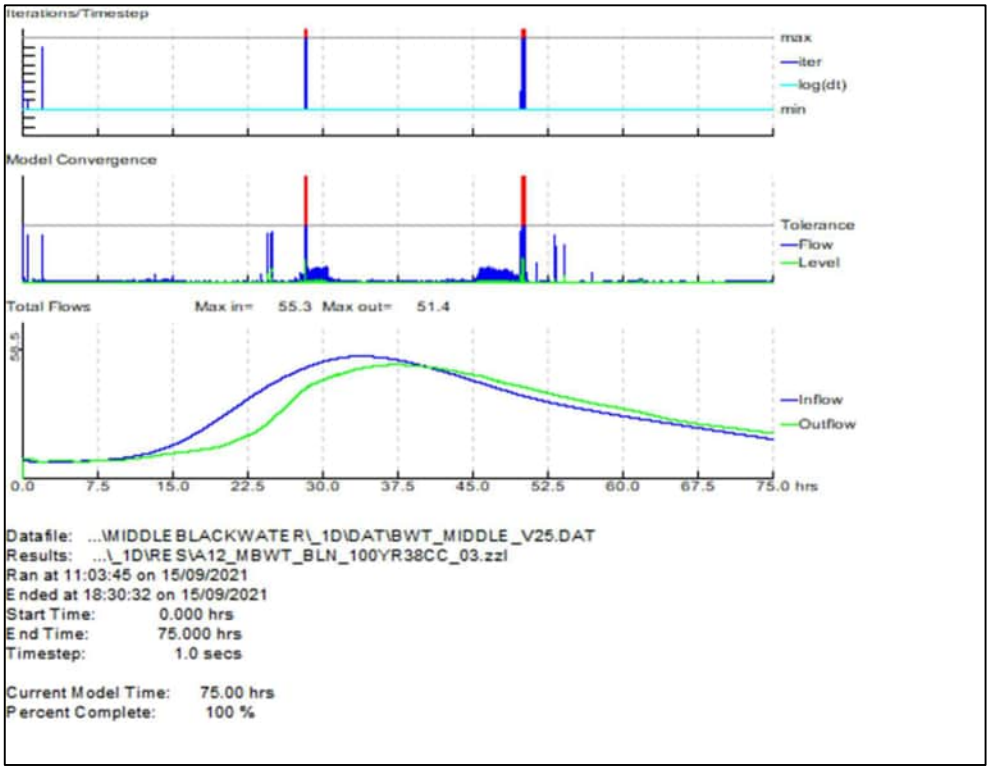
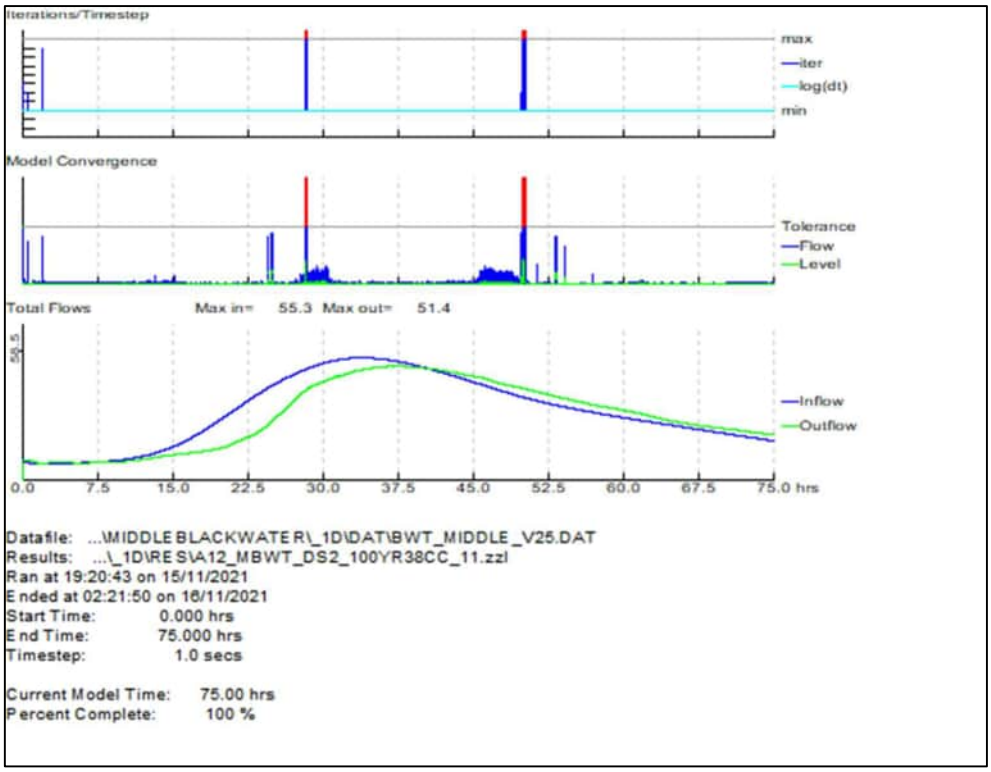


Plate 6.2 1D model convergence – 1% (1 in 100) AEP +38%CC event for the with-scheme scenario



6.2.2 The change in volume (dVol) in the 2D domain during the model simulation can be seen in Plate 6.3 and Plate 6.4 for the baseline and with-scheme 1% (1 in 100) AEP +38%CC event simulations. This plot is typical for all the events and scenarios simulated. There is some variability in dVol evident at the start of the simulation, which reflects the adjustment in the volumes exchanged between 1D and 2D during the initial time steps following the changes made to the model. The change in volume remains smooth during the rest of the simulation, which is acceptable.

6.2.3 The cumulative mass error diagnostics output from the TUFLOW 2D model has also been checked for each model simulation. The tolerance range recommended by the software manual is +/- 1% mass balance error. Plate 6.3 and Plate 6.4 show the cumulative mass error remains within recommended tolerances except at the start of the simulation at which time there is an insignificant volume of water in the 2D domain and is therefore deemed acceptable.

Plate 6.3 Cumulative mass error and change in volume – 1% (1 in 100) AEP +38%CC event – baseline scenario

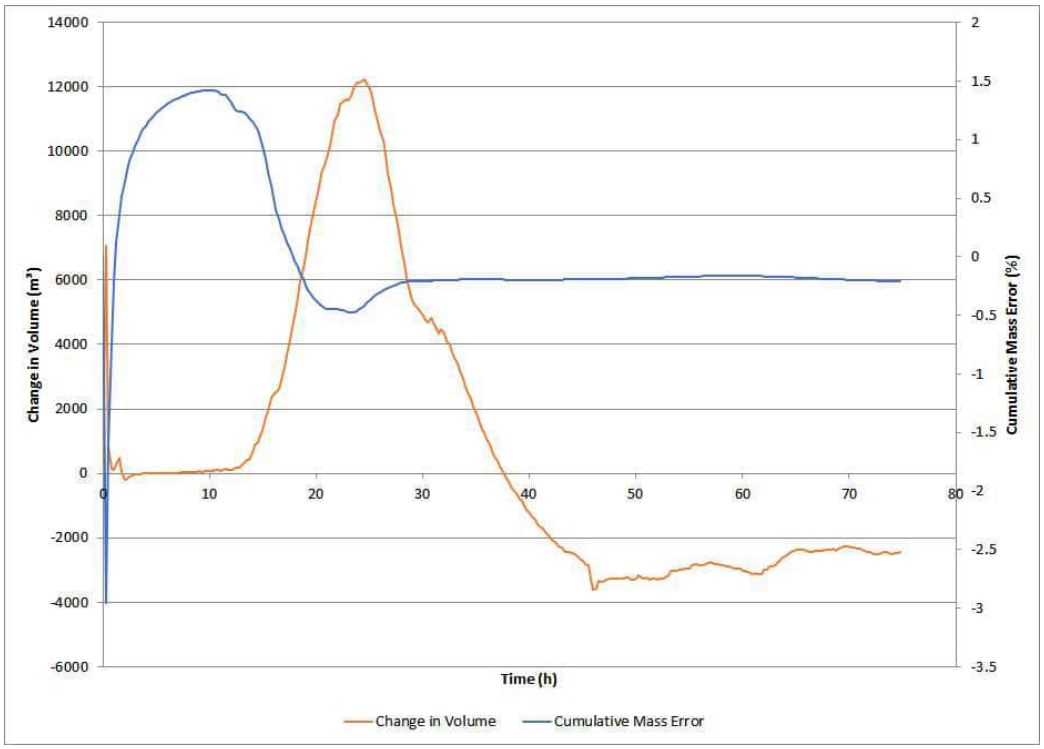
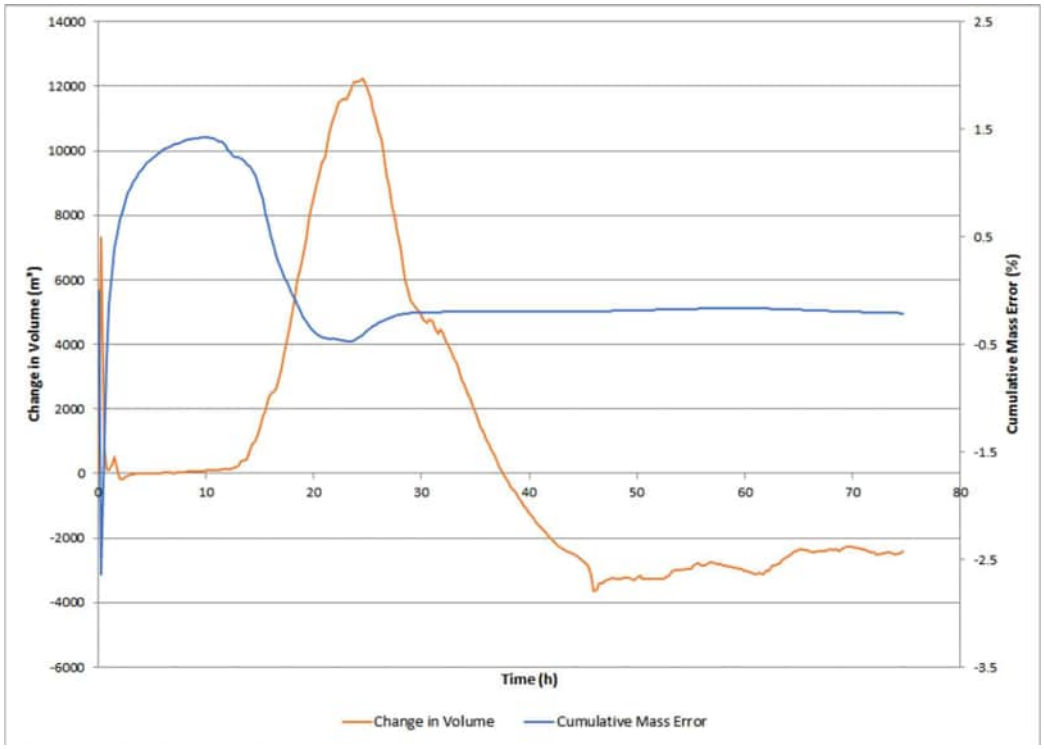


Plate 6.4 Cumulative Mass Error and change in volume – 1% (1 in 100) AEP +38%CC event – with-scheme scenario



6.3 Calibration and verification

Calibration

6.3.1 There is one gauge within the modelled reach of the River Blackwater at Appleford Bridge (37010). This gauge has been classified by the NRFA Peak Flows dataset as suitable for both QMED estimation and pooling group analysis.

6.3.2 The incoming model was calibrated using historical large magnitude flood events by JBA in 2010 (calibration event: October 2001) and CH2M in 2017 (calibration events: October 2001, February 2009, January 2013 and November 2014). This hydraulic model has been adopted by Jacobs for the present study but was truncated to cover the current study extent only. The design hydrology (peak flows) has therefore been revised by Jacobs based on new methodology as well as recent data.

6.3.3 As the hydraulic model has already been calibrated for past large magnitude flood events and given the post 2017 flood events have been of a lower flood magnitude (ranked 28 or greater) with AEP events equal to or less than QMED (50% AEP event), it was decided that further model re-calibration was not required for the current study.

Verification using historic data

6.3.4 Environment Agency historic flood maps are available, which show the maximum extent of all individual recorded flood events. Plate 6.5 shows the comparison of historic flood map against the 1% (1 in 100) AEP flood extent predicted by the model. There is generally a good match between the two extents with minor differences upstream of the A12 crossing and at the downstream end of the model. It should also be noted that the historic flood map is a composite of anecdotal data and carries inherent uncertainties.

Verification using Environment Agency flood maps

6.3.5 Plate 6.6 shows the modelled 1% (1 in 100) AEP flood extent and the Environment Agency published Flood Zone 3. The comparison shows that the two maps are generally very similar. However, in some areas the Environment Agency flood map predicts a slightly larger flood extent than the model results. This is expected as the present study includes significant refinement of the modelling process.

Verification conclusion

6.3.6 In conclusion, there is limited available data for verification of the River Blackwater model. As stated above, the model has been previously verified by JBA in 2010 and CH2M in 2017, and the Environment Agency flood map shows a generally similar flood extent to the present model (Plate 6.6). Therefore, the model is considered to be appropriate for use in the present study.

Plate 6.5 Modelled 1% (1 in 100) AEP event flood extent vs. Environment Agency historic flood map

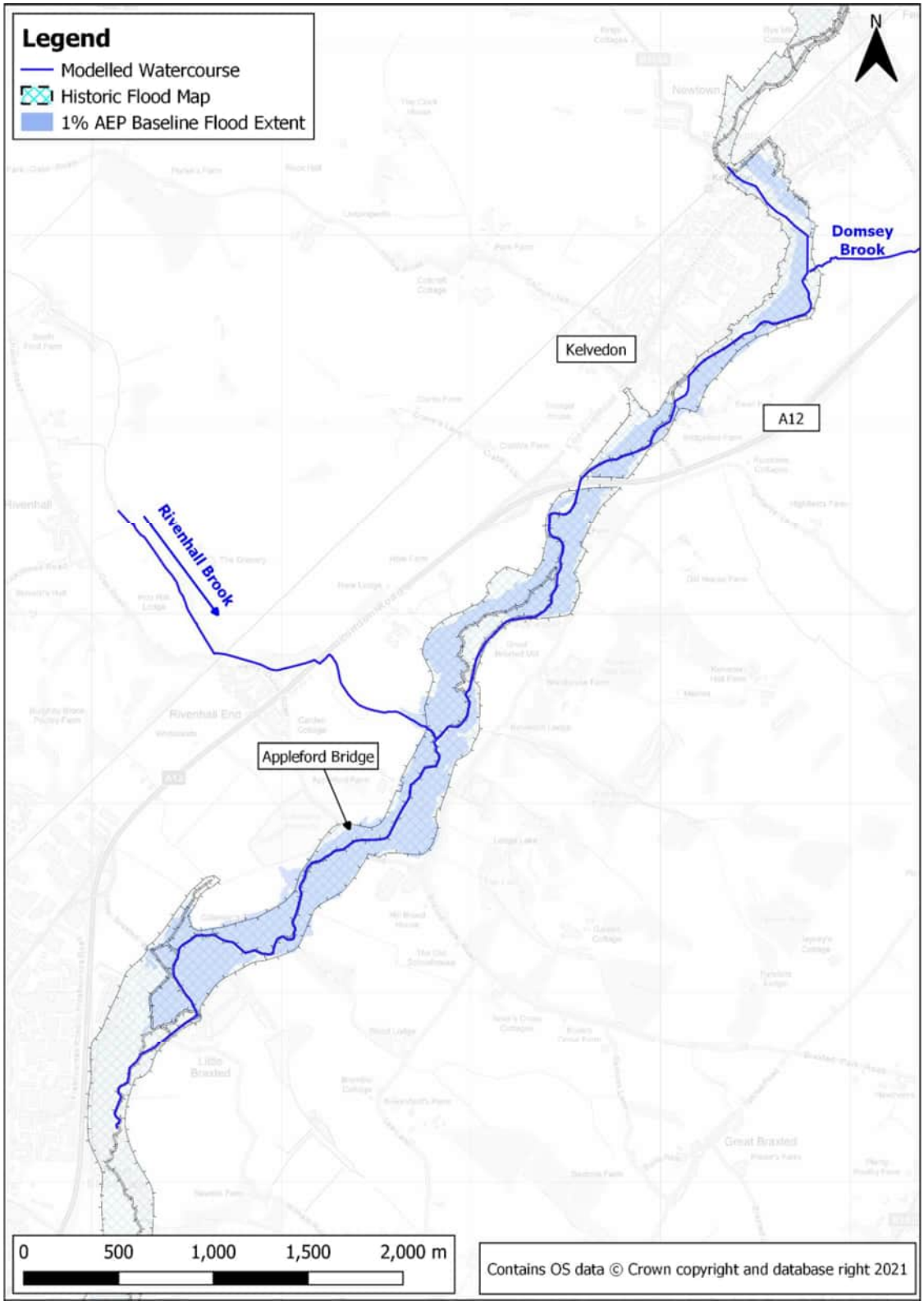
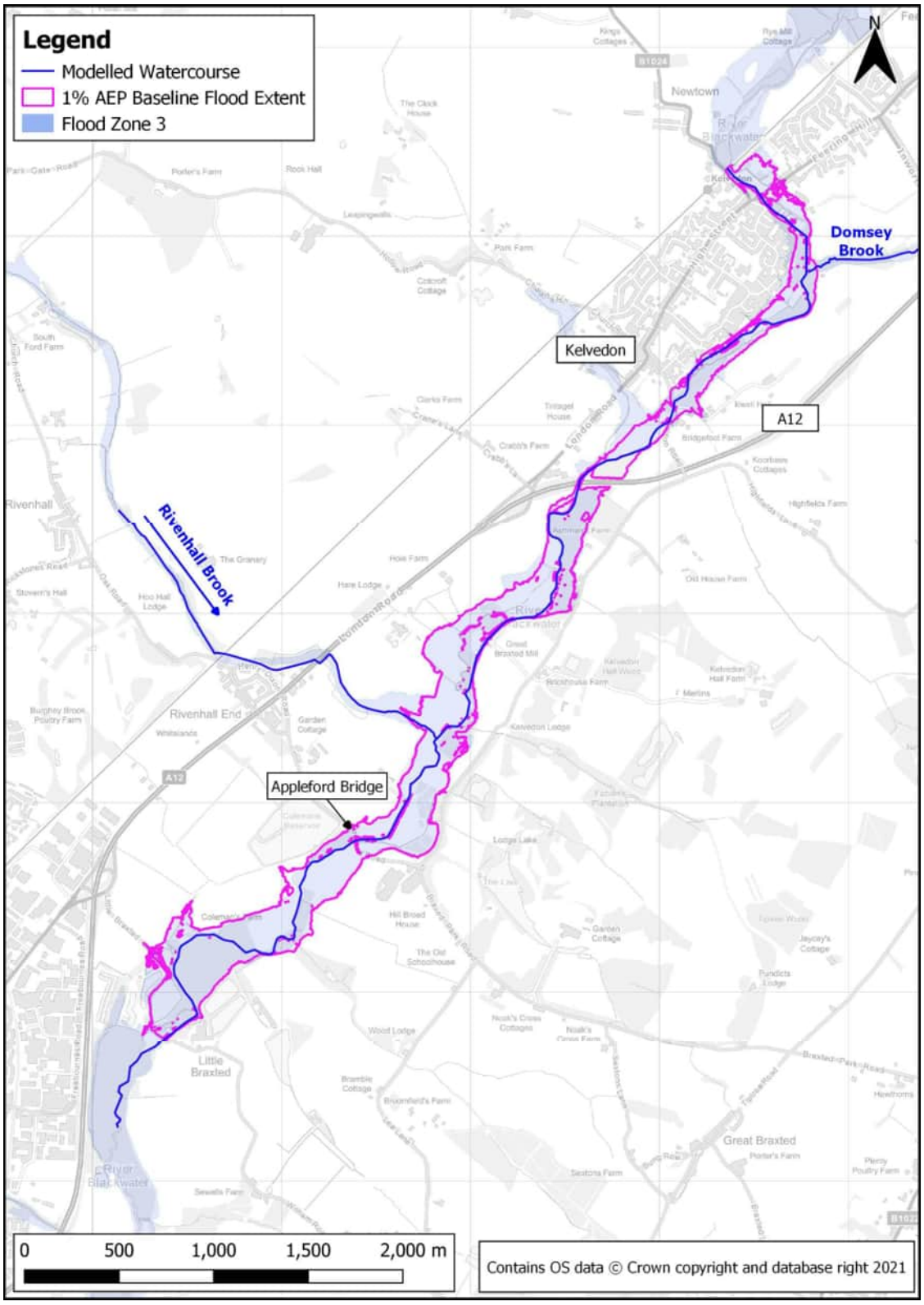


Plate 6.6 Modelled 1% (1 in 100) AEP event flood extent vs. Environment Agency Flood Zone 3



6.4 Sensitivity analysis

6.4.1 All sensitivity tests have been performed for the 1% (1 in 100) AEP event.

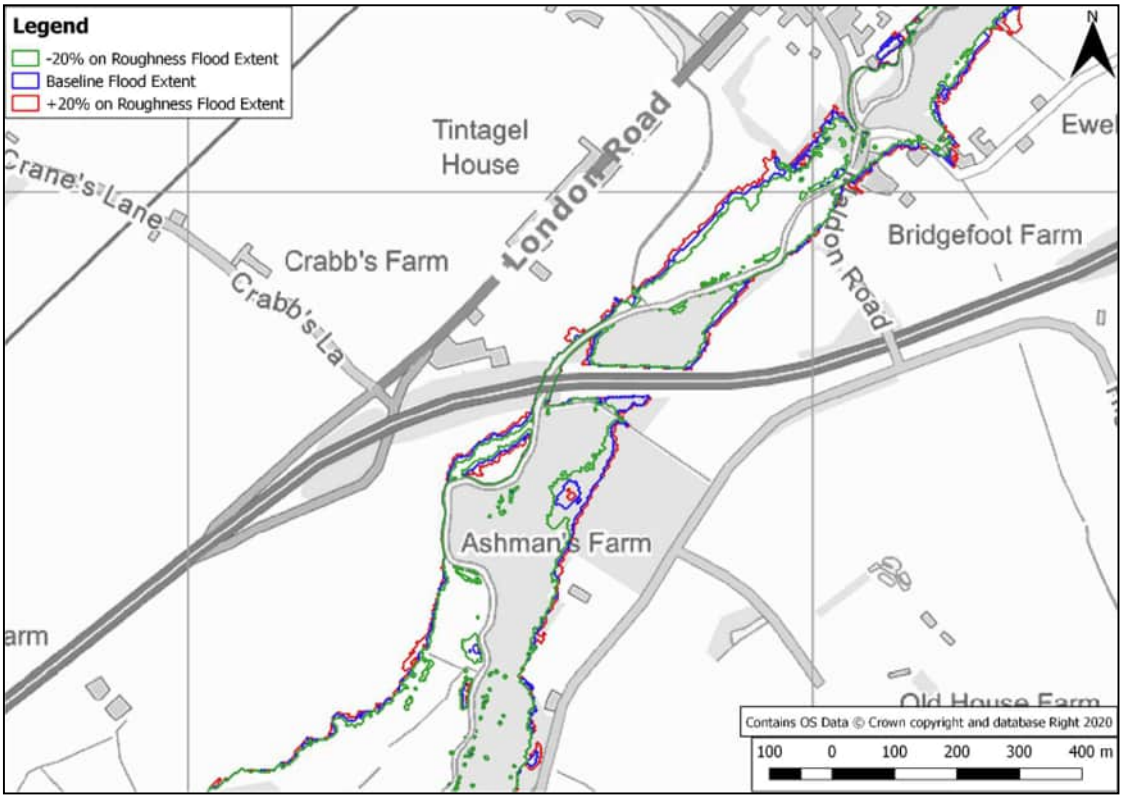
Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Table 6.1 shows the impact of changing the model roughness on in-channel peak water levels. The results show that maximum water levels are generally not very sensitive to changes to roughness. However, it can be observed at the A12 crossing that the modelled flood extent around the proposed-scheme is insensitive to roughness changes on the upstream side of the A12 crossing and moderately sensitive to roughness changes on the downstream side of the A12 crossing. This is shown in Plate 6.7.

Table 6.1 Roughness sensitivity results – 1% (1 in 100) AEP event

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	0.13	-0.05	0.06
-20% Roughness	0.08	-0.16	-0.07

Plate 6.7 Roughness sensitivity results at A12 crossing – 1% (1 in 100) AEP event



Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Table 6.2 shows the impact of changing the model inflows on in-channel peak water levels. The results show that the maximum in-channel water levels are not very sensitive to changes in flow. Similar observations can be made with the floodplain response where relatively little change in the flood extent is observed for the adjusted flows.

Table 6.2 Flow sensitivity results – 1% (1 in 100) AEP event

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.17	0.00	0.09
-20% Flow	0.00	-0.21	-0.11

Downstream boundary condition sensitivity

6.4.4 The water level in the 1D HT (stage-time) downstream boundaries were adjusted by increasing and decreasing the constant stage by 0.5m, while stage in the 2D HQ boundary was increased and decreased by 0.2m for the two sensitivity tests. Table 6.3 shows the response at the downstream end of the model. The location at which there is no change in water level as a result of changing the downstream boundary has been identified. Distances from this location in relation to the downstream end of the model (tailwater distance) and in relation to the proposed-scheme are also shown.

6.4.5 The results show that any effect of the downstream boundaries is restricted to the immediate upstream area and is remote from both the proposed-scheme location and Appleford gauge by a considerable distance.

Table 6.3 Downstream boundary sensitivity results – 1% (1 in 100) AEP event

Sensitivity	Water Level Difference at LBRAX291u and BWT27600u (m)	Tailwater Distance (m)	Distance to proposed-scheme (m)
Increasing the stage in Downstream Boundary Condition*	0.421	291	4360
Decreasing the stage in Downstream Boundary Condition*	-0.360	291	4360

*Differences in water level are only observed downstream of Little Braxted Mill Race control structures.

7 Baseline model results

7.1.1 In-channel maximum water levels have been inspected at key locations in relation to the proposed-scheme. Table 7.1 shows in-channel maximum water levels for all the events modelled. Maximum water levels for these nodes for all events and scenarios can be found in Appendix C.

7.1.2 Plate 7.1 shows the baseline scenario maximum flood extent for all the events modelled. The results show that within the modelled extents, water is contained within the wider River Blackwater valley as far as Appleford Bridge. There is no flooding to the A12 highway itself at the crossing location and the A12 bridge structure generates limited head loss as the headwall and abutments are above the maximum water level in all of the modelled flood events.

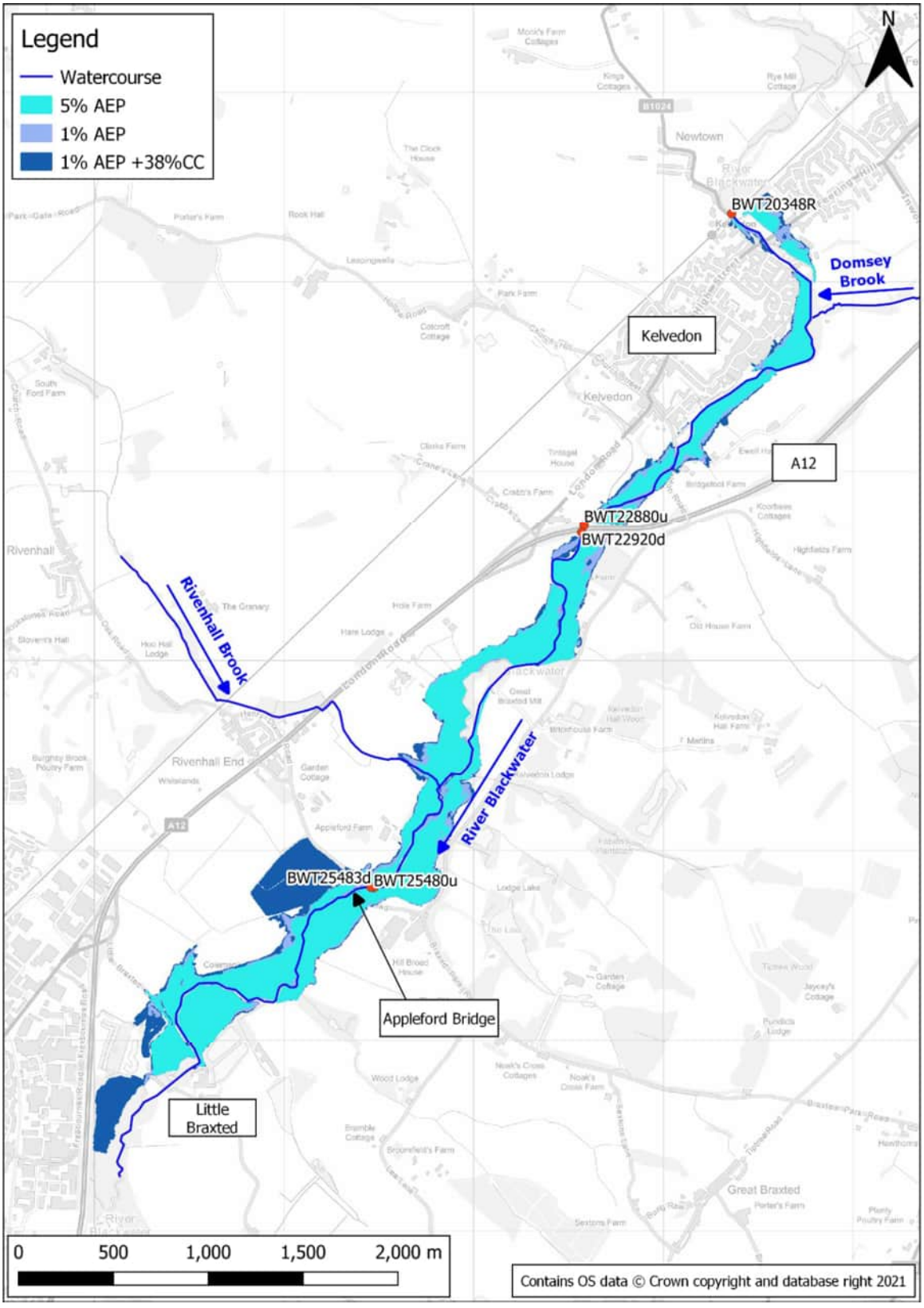
7.1.3 More widely, no significant hydraulic controls were observed on the River Blackwater. Several Mill Race structures including: Easterford Mill, Grey's Mill, Great Braxted Mill and Little Braxted Mill, generate small local head losses of between 0.3 to 0.4 metres for the modelled events.

7.1.4 A number of properties are at risk of flooding in Kelvedon and around Little Braxted even in the 5% (1 in 20) AEP event. These properties are remote from the A12 crossing.

Table 7.1 In-channel maximum water levels at key locations

Node Labels	Description	Water Level (mAOD)		
		5% AEP (1 in 20)	1% AEP (1 in 100)	1% AEP +38%CC (1 in 100+CC)
BWT20348R	Upstream of the railway	22.05	22.27	22.57
BWT22880u	Upstream of the existing A12 Road bridge	19.03	19.20	19.34
BWT22920d	Downstream of the existing A12 Road bridge	19.03	19.20	19.34
BWT25480u	Upstream of the Appleford Gauging Weir	16.27	16.56	16.70
BWT25483d	Downstream of the Appleford Gauging Weir	16.20	16.50	16.65

Plate 7.1 Baseline scenario modelled flood extents



8 With-scheme modelling

8.1 Model modifications

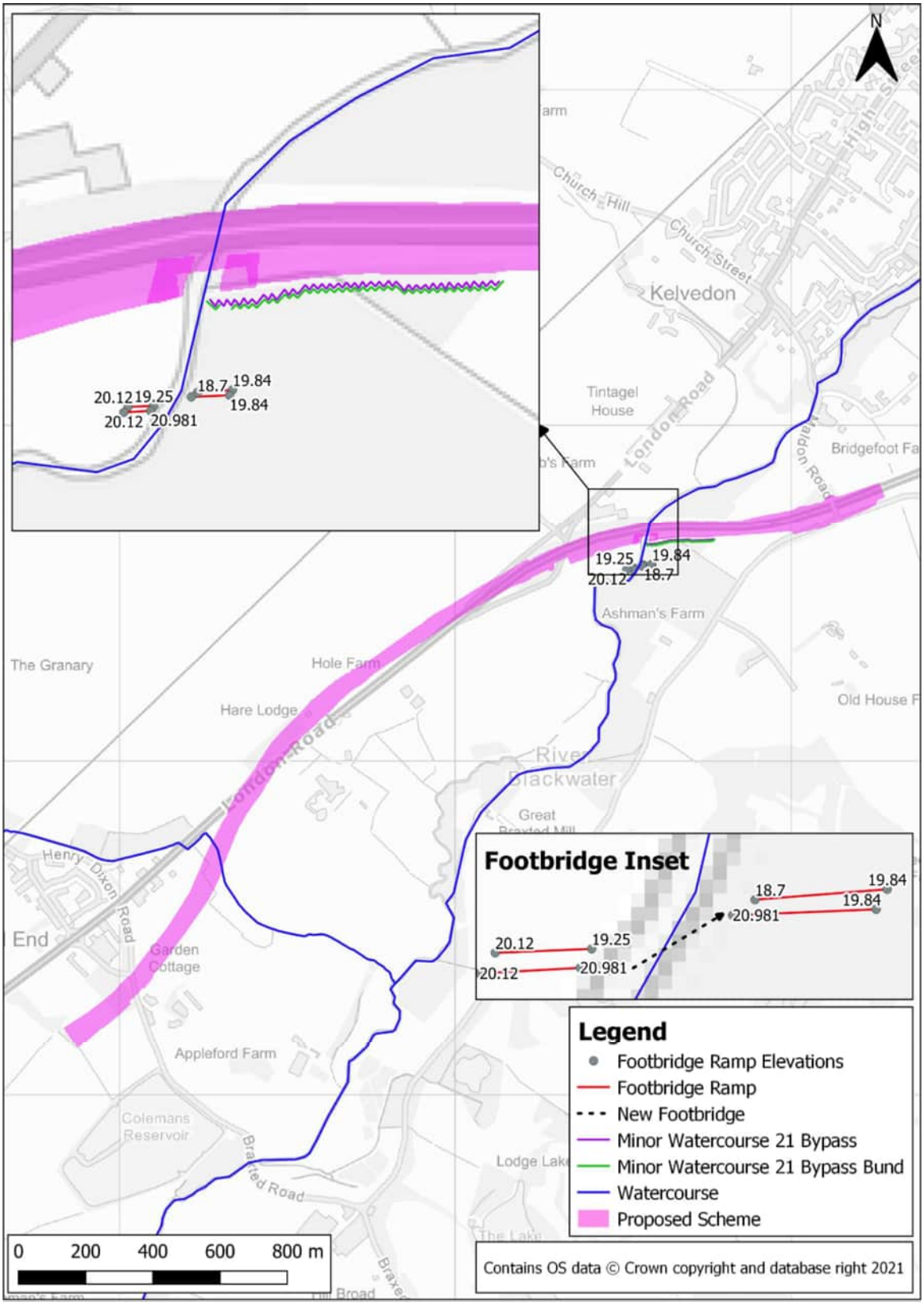
8.1.1 The new layout of the A12 highway includes 3 lanes on both carriageways. This will require the bridge to be extended to accommodate the additional lanes. The bridge widening will be carried out on the downstream face of the structure only.

8.1.2 The proposed-scheme was provided as an AutoCAD drawing file. This was converted to a topographic ascii file, which was superimposed on to the 2D domain of the baseline model as a 2D feature. Wing walls were specified within the downstream opening to limit the impact of the updated structure. No changes were required to the 1D model. Plate 8.1 shows the location and overhead footprint of the proposed-scheme.

8.1.3 In addition, the channel and bund for Minor Watercourse 21 (MWC21) were schematised into the 2D domain topography. Their locations are shown in Plate 8.1.

8.1.4 Thirdly, a new footbridge will be built further south from the existing footbridge which will be replaced as it is currently too close to the proposed-scheme. The bridge itself was not modelled as its level is higher than the level of flooding in the largest event modelled. However, it has large abutments that include ramps leading up to the bridge. This can be seen in the inset of Plate 8.1. The ramps were modelled in the 2D domain using Z-lines to modify the topography with the levels (in mAOD) shown in Plate 8.1.

Plate 8.1 Proposed-scheme layout



8.2 With-scheme results

8.2.1 It was found that the proposed-scheme did not significantly change the flood risk predicted by the model in comparison with the existing situation. The flood map in Plate 8.2 shows the impact on maximum flood levels due to the enlargement of the A12 for the 1% (1 in 100) AEP +38%CC. An increase in flood levels of greater than 10mm is classified as adverse and may require mitigation. Throughout the modelled area, the difference in flood levels is less than 10mm, except for a very small patch downstream of the scheme where the increase in flood level is approximately 25mm. This is due to the ramps to the new footbridge on the floodplain. However, this is within the Order Limit, therefore no mitigation is necessary.

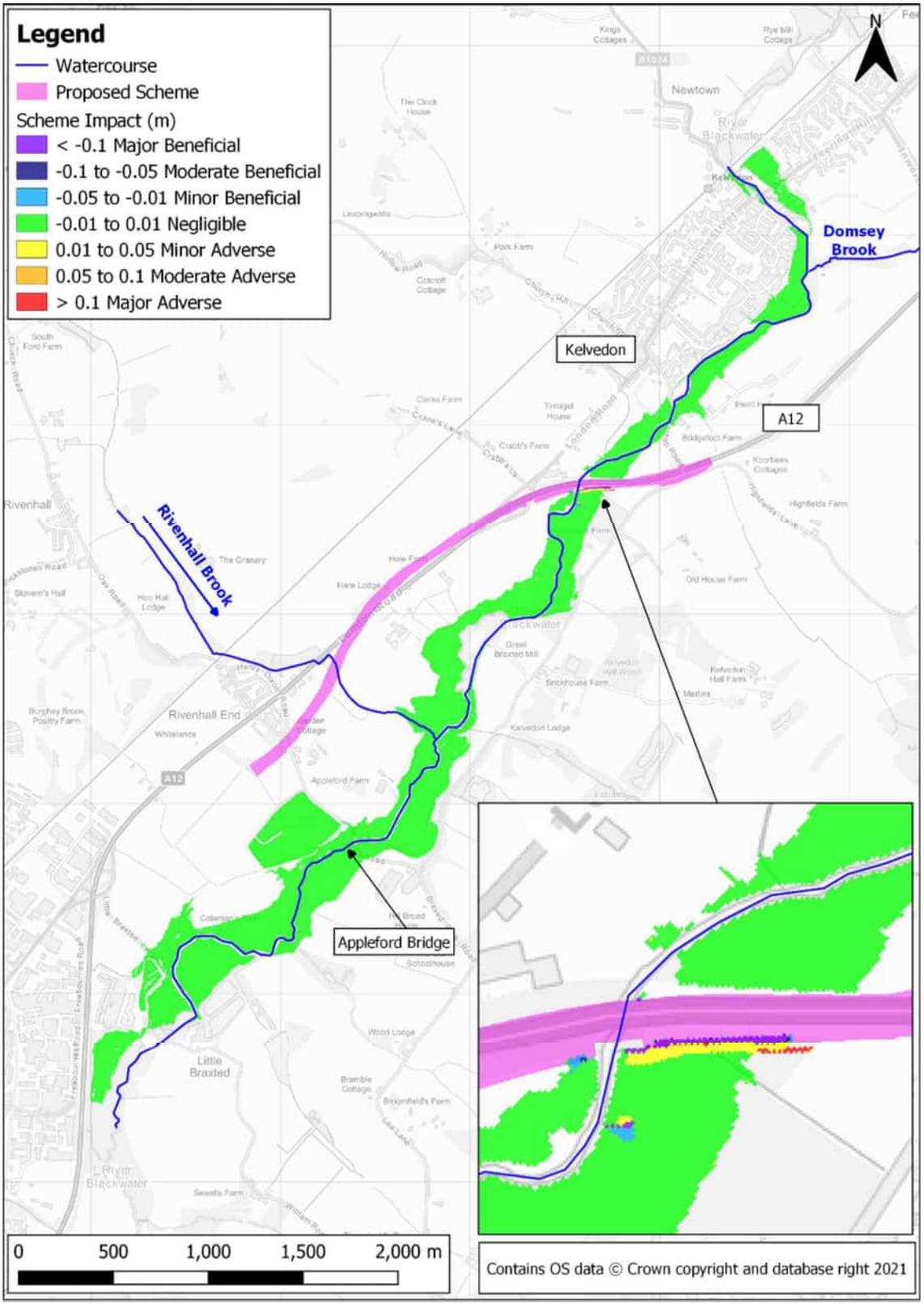
8.2.2 Table 8.1 compares the in-channel water levels at key locations between the baseline and with-scheme scenarios for the 1% (1 in 100) AEP +38%CC event. They show that peak water levels within the channel immediately upstream and downstream of the proposed-scheme do not change by more than 10mm.

8.2.3 Tabulated results and long profiles of maximum water levels for all events and scenarios for the key locations can be found in Appendix C. Flood Maps for all events can be found in Appendix D.

Table 8.1 Comparison of in-channel Maximum Water Levels at Key Locations – 1 % (1 in 100) AEP +38%CC Event

Node Labels	Description	Baseline Water Levels (mAOD)	With-Scheme Water Levels (mAOD)	Difference (m)
BWT20348R	Upstream of the railway	22.57	22.57	0.00
BWT22880u	Upstream of the A12 Road bridge	19.34	19.34	0.00
BWT22920d	Downstream of the A12 Road bridge	19.34	19.34	0.00
BWT25480u	Upstream of the Appleford Gauging Weir	16.70	16.70	0.00
BWT25483d	Downstream of the Appleford Gauging Weir	16.65	16.65	0.00

Plate 8.2 Impact due to the proposed-scheme for the 1% (1 in 100) AEP +38%CC event



9 Temporary works modelling

9.1 Temporary works model changes

9.1.1 Modelling of the temporary works required to facilitate the construction of the proposed-scheme was undertaken. A haul road of 10m width, east of the A12 carriageway (see Plate 9.1) was incorporated into the model by modifying the elevation of the ground in the 2D domain using zshape file.

9.1.2 In addition, the bed at the A12 crossing bridge unit was modified to account for temporary works, as shown in Plate 9.2.

Plate 9.1 Proposed-scheme layout with temporary works haul roads

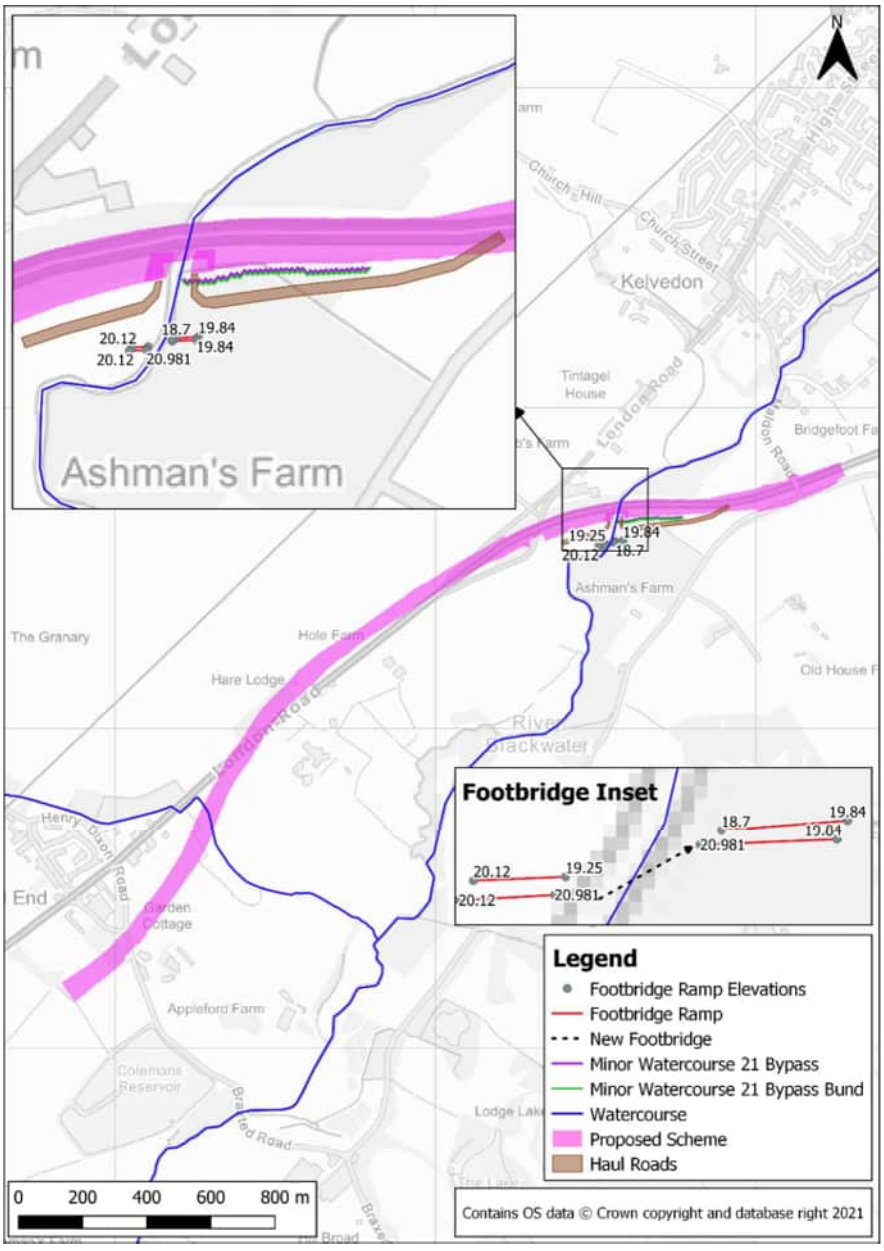
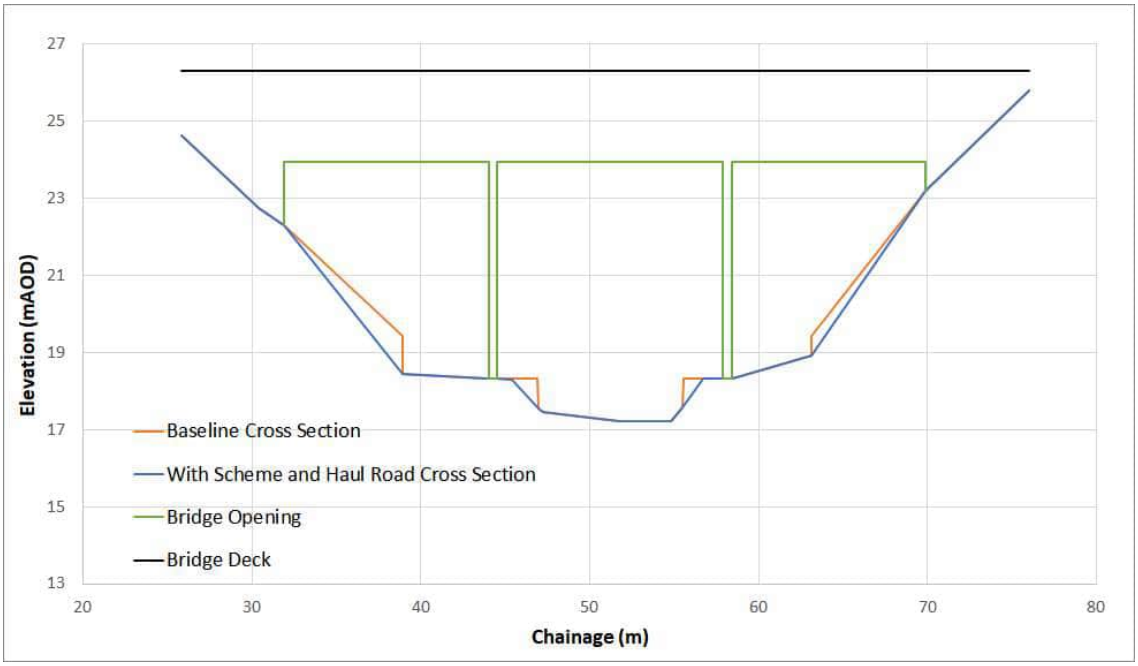


Plate 9.2 A12 bridge cross section change at unit BR_BWT22880u



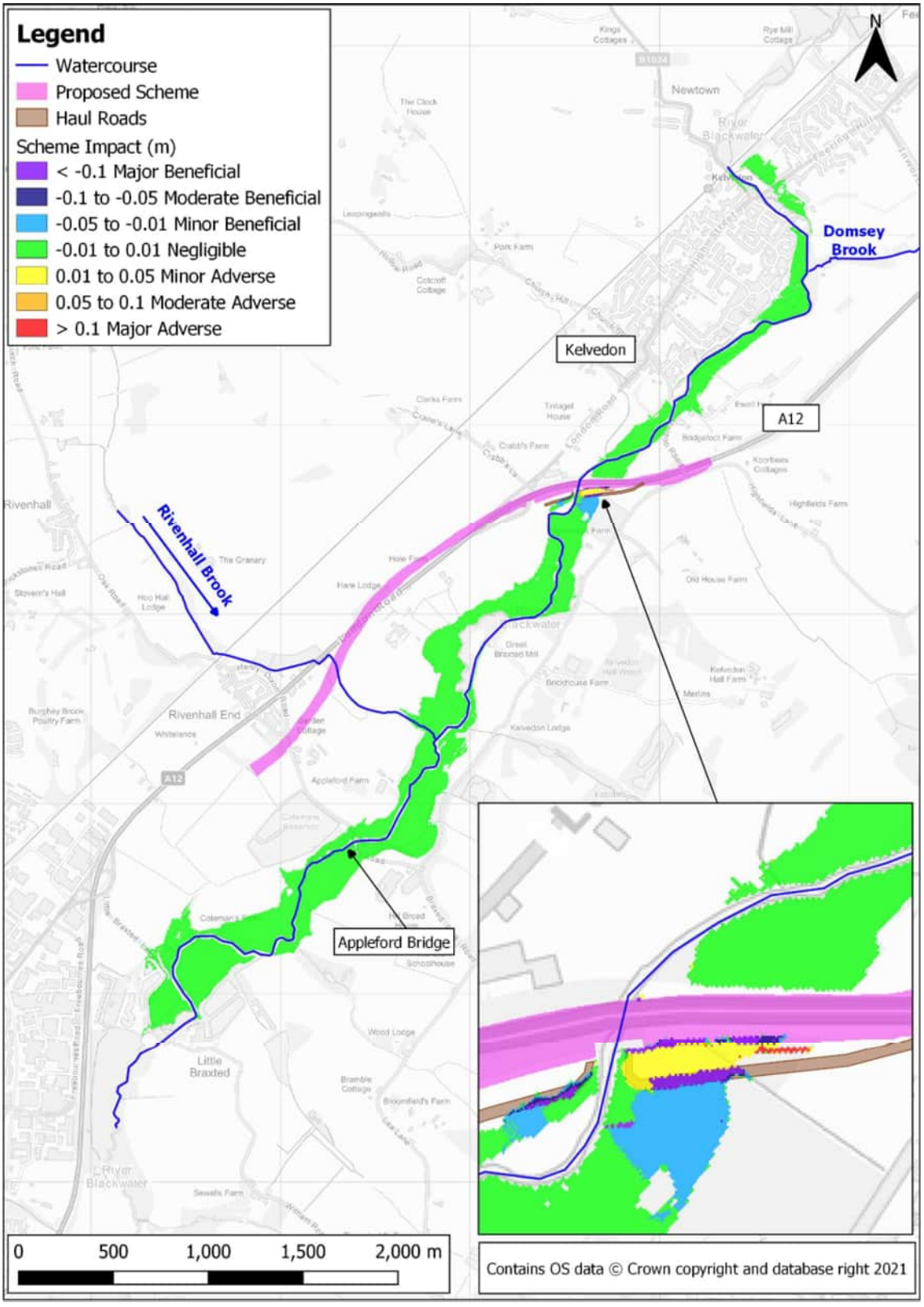
9.2 Temporary works model results

9.2.1 The inclusion of the haul road in the model shows an increase in water levels between the haul road and the A12 up to 35mm for the 1% (1 in 100) AEP event.

9.2.2 The haul road close to the channel is affected by flooding. This can be seen in Plate 9.3 which shows the impact of the proposed-scheme and temporary works on flood depths for the 1% (1 in 100) AEP event. Water level difference maps for the 1% (1 in 100) and 5% (1 in 20) AEP events can be found in Appendix D.

9.2.3 No properties are affected by the increase in water levels.

Plate 9.3 Impact due to the scheme and haul road for the 1% (1 in 100) AEP event



10 Model assumptions and limitations

10.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model update process.

10.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed-scheme and are therefore, suitable for the flood risk assessment.

10.2 Hydrology

10.2.1 The main hydrological assumptions used in this study are as follows:

- ReFH1 hydrograph shapes are suitable to use in this investigation.
- The result of a sensitivity analysis using the ReFH2.3 hydrograph shape indicates that the ReFH1 hydrograph shape predicts slightly conservative flood extent at the A12 crossing
- The ReFH1 units from the incoming model are suitable for both inflow and residual (lateral inflow) catchments
- ReFH2.3 is likely to be overestimating lower AEP event design peak flows (that is, for peak flow estimates up to and including the 1% (1 in 100) AEP event) for the target catchment (based on gauged data for the River Blackwater at Appleford Bridge) and therefore ReFH2.3 peak flow estimates were not adopted for lower AEP events
- The River Blackwater at Appleford Bridge is suitable to estimate design peak flows up to the 1% (in 100) AEP event using Enhanced Single Site statistical analysis, and
- ReFH2.3 ratios are more appropriate for higher AEP flood events and therefore this method was adopted for the 0.5% (1 in 200) AEP and rarer flood event

10.3 Hydraulic modelling

10.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for Middle Blackwater are as follows:

- Selected model cross-sections have been extended using LiDAR data
- Bank-top levels alongside River Blackwater were left unchanged from the incoming model, except around the A12 crossing where

LiDAR data has been used instead as it is considered as more accurate

- The LiDAR data is assumed to accurately represent the modelled floodplain
- Hydraulic coefficients for structures have been retained from the incoming model and confirmed as accurate using available guidance within the Flood Modeller software
- In-bank channel roughness has been retained from the incoming model and confirmed as accurate using the best available information (survey data and aerial photographs), with reference to the available guidance (Chow 1959)
- Average Manning's 'n' value of 0.06 has been applied to the out-of-bank channel roughness based on survey data and aerial photographs, with reference to the available guidance (Chow 1959)
- A 4m grid has been used to represent the anticipated scheme features with an sufficient level of detail, in particular the floodplain adjacent to the proposed-scheme embankment footprint
- The hydraulic model was calibrated and verified by CH2M in 2017, therefore, no recalibration was deemed necessary for the present study
- Contribution from Minor Watercourse 21 is part of the lateral flow B45int, hence represented in the baseline scenario. However, with the proposed-scheme in place, this distribution changes. This change is not modelled in the with-scheme and temporary works scenarios because the response from it is expected to be much earlier and smaller than the response of the Middle Blackwater River
- The existing footbridge was not modelled in the baseline scenario as the abutments are too small to be represented in the model, unlike the new footbridge which has large abutments with ramps

11 Conclusion

11.1.1 This report has detailed the modelling carried out to assess the baseline and with-scheme flood risk for Middle Blackwater with reference to the location of the proposed-scheme.

11.1.2 The new A12 road layout was modelled for design storms up to the 1% (1 in 100) AEP +38%CC event, and it was found that the proposed embankment footprint displaces a small area of flooding on the downstream side of the highway. The model results show a very small patch of increased flood level local to the downstream face of the new bridge, but no significant impact elsewhere. This patch is within the Order Limit, therefore no mitigation is proposed.

11.1.3 The inclusion of the haul roads increases water levels downstream of the proposed-scheme for the 5% (1 in 20) and 1% (1 in 100) AEP events. It also shows flooding of the actual haul road. However, the effect is small for the 5% (1 in 20) AEP event and the likelihood of the haul road flooding in the 1% (1 in 100) AEP event is small. Hence no mitigation is required.

11.1.4 The results of the baseline modelling have shown that a number of properties are at risk of flooding in Kelvedon and around Little Braxted. However, these are all notably remote from the proposed-scheme location and are shown by the model results to be entirely unaffected by the introduction of the proposed-scheme.

11.1.5 The A12 highway remains free from flooding up to the 1% (1 in 100) AEP +38%CC event for both baseline and with-scheme model simulations.

Acronyms

Abbreviation	Term
AM	Annual Maximum
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
ESS	Enhanced Single Site flood frequency analysis
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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Appendix A **River Blackwater Hydrology Appendix**

Please see document HE551497-JAC-EWE-SCHW-RP-LE-0078.docx

Appendix B **Structure Schedule**

Table B.1 Baseline Modelled Structure Schedule

Structure Name	Node Labels	XY Coordinates	Modelling Approach
Feering Hill (B1024) Bridge on River Blackwater in Kelvedon	BR_BWT20530u (Main bridge) 20530FloodU (flood relief on left bank)	586534.3, 219216.1	Represented as arch bridges without any spill unit in parallel. Lower and upper transition distance of respectively 0m and 0.6m are applied on the flood relief bridge, whilst 0m is applied on main bridge. Spill was ignored as road embankment is never overtopped. FM default coefficients applied for all other parameters.
Bridge Meadows Road Bridge on River Blackwater	EAST3156BrUp	586591.3, 219192.9	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of respectively 0m and 0.6m are applied. Spill coefficient of 1.0 to represent the concrete road. FM default coefficients applied for all other parameters.FM default coefficients are applied to all the other parameters.
Access Bridge on River Blackwater Flood Relief Channel	COAL55OrU	586464.3, 219354.9	Represented as a rectangular orifice with a spill unit in parallel. A bore area of 0.282m^2 is used. Spill coefficient of 1.0 to represent the crest. FM default coefficients applied for all other parameters.
Feering Hill (B1024) culvert on River Blackwater Flood Relief Channel	COAL239u	586584.2, 219247.6	Represented as a rectangular orifice without any spill unit in parallel. A bore area of 1.0m^2 is used. Spill was ignored as road embankment is not overtopped locally. FM default coefficients are applied to all the other parameters.
Vertical Sluices to allow water to pass from	SL_BWT20730u (North Sluice) SL_EASTW0u	586663.5, 219104.4	Represented as vertical sluice gates. Northern and southern sluice have a weir crest elevation of respectively

Easterford Mill Race to River Blackwater	(South Sluice)		19.75 mAOD and 19.86 mAOD, with a breadth of weir of respectively 1.4m and 1.09m . Their single gate of respectively 1.61m and 1.09m height is left open for the entire flood event. FM default coefficients are applied to all the other parameters.
Easterford Mill culvert	BWT20770Ou	586693.7, 219064.6	Represented as a rectangular orifice with a spill unit in parallel. A bore area of 0.77m^2 is used. Spill coefficient of 1.0 to represent the crest. FM default coefficients applied for all other parameters.
Vertical Sluices to allow water to pass from Greys Mill Race to River Blackwater	SL_GREYW0u SL_GREYW0u2	586143.1, 218240.6	Represented as vertical sluice gates. SL_GREYW0u is a sluice with 3 gates of 1.33m height and 0.84m breadth of weir. SL_GREYW0u2 is a single gate sluice of 1.13m height and 0.35m breadth of weir. Both sluices have a weir crest elevation of 18.6 mAOD and are left open for the entire flood event. FM default coefficients applied for all other parameters.
Side Weir to allow water to pass from Grey's Mill Race to River Blackwater	SP_GBRAX0u	585206, 216966	Represented as a spill unit. Spill coefficient of 1.7 is used to represent the concrete crest. FM default coefficients applied for all other parameters.
Grey's Mill culvert	BWT22107OrU	586102.2, 218211.7	Represented as a rectangular orifice without any spill unit in parallel. A bore area of 3.26m^2 is used. Spill was ignored as surrounding ground is not overtopped locally. FM default coefficients are applied to all the other parameters.
Bridge on Maldon Road	BR_BWT22230u	586074.2, 218099.6	Represented as an USBPR1978 bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Bridge includes two flood relief culverts with section area of 0.64m^2 each.

			FM default coefficients applied for all other parameters.
A12 Road Bridge	BR_BWT22880u	585575.3, 217696.6	Represented as an USBPR1978 bridge without any spill unit in parallel. Lower and upper transition distance of 0m is applied. Bridge does not include any flood relief culvert. Spill was ignored as road embankment is not overtopped locally. FM default coefficients applied for all other parameters.
Great Braxted Mill access bridge on River Blackwater	BR_GBRAX178u	585063.7, 216980.5	Represented as an USBPR1978 bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Bridge does not include any flood relief culvert. Bridge does not include any flood relief culvert. Spill coefficient of 1.0 to represent the crest. FM default coefficients applied for all other parameters.
Great Braxted Mill Culverts	BWT24100Or1U BWT24100Or2U BWT24100Or3U	585097.8, 216865.8	Represented as a rectangular orifice without any spill unit in parallel. BWT24100Or1U has a bore area of 3.41m^2, while other two have a bore area of 2.3m^2. Spill was ignored as surrounding ground is not overtopped locally. FM default coefficients are applied to all the other parameters.
Appleford Gauging Weir	BWT25480u	584464.1, 215807.0	Represented as spill unit. Spill coefficient of 1.94 is used to represent the concrete crest. FM default coefficients applied for all other parameters.
Appleford Bridge	BR_BWT25550u	584392.3, 215803.5	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied.

			Spill coefficient of 1.0 to represent the bridge parapet and surrounding ground. FM default coefficients applied for all other parameters.
River Blackwater bypass weir upstream of Little Braxted Mill	SP_LBRAX0u	583539, 214909	Represented as a spill unit. Spill coefficient of 2.0 is used to represent the concrete crest. FM default coefficients applied for all other parameters.
Side Weirs to allow water to pass from Little Braxted Mill Race to River Blackwater	SP_LBRAXW0u SP_BWT27525u	583420, 214790	Represented as 2 spill units. Spill coefficient of 1.4 is used to represent the concrete crest whilst 1.0 is used to represent the dilapidated weir. FM default coefficients applied for all other parameters.
Little Braxted Mill culvert	BW27600OrU	583376.5, 214741.7	Represented as a rectangular orifice without any spill unit in parallel. A bore area of 0.45m^2 is used. Spill was ignored as surrounding ground is not overtopped locally. FM default coefficients are applied to all the other parameters.
Access Bridge on Little Braxted backwater	LBRAX130OrU	583449.2, 214863.5	Represented as a rectangular orifice with a spill unit in parallel. A bore area of 3m^2 is used. Spill coefficient of 1.0 to represent the crest. FM default coefficients applied for all other parameters.
Little Braxted Lane Bridge on River Blackwater	LBRAX291u	583307.4, 214803.9	Represented as a rectangular orifice with a spill unit in parallel. A bore area of 21.612m^2 is used. Spill coefficient of 1.0 to represent the road crest. FM default coefficients applied for all other parameters.

Appendix C **Maximum Water Level Tables and Long Sections**

Table C.1 Water Levels

Model Node	Description	Maximum Water Levels (mAOD)								
		5% (1 in 20) AEP			1% (1 in 100) AEP			1% (1 in 100) AEP +38%CC		
		Baseline	With-Scheme	Difference (m)	Baseline	With-Scheme	Difference (m)	Baseline	With-Scheme	Difference (m)
BWT20348R	Upstream of the railway	22.05	22.05	0.00	22.27	22.27	0.00	22.57	22.57	0.00
BWT22880u	Upstream of the existing A12 Road bridge	19.03	19.03	0.00	19.20	19.20	0.00	19.34	19.34	0.00
BWT22920d	Downstream of the existing A12 Road bridge	19.03	19.03	0.00	19.20	19.20	0.00	19.34	19.34	0.00
BWT25480u	Upstream of the Appleford Gauging Weir	16.27	16.27	0.00	16.56	16.56	0.00	16.70	16.70	0.00
BWT25483d	Downstream of the Appleford Gauging Weir	16.20	16.20	0.00	16.50	16.50	0.00	16.65	16.65	0.00

Plate C.1 Baseline In-channel Peak Water Levels

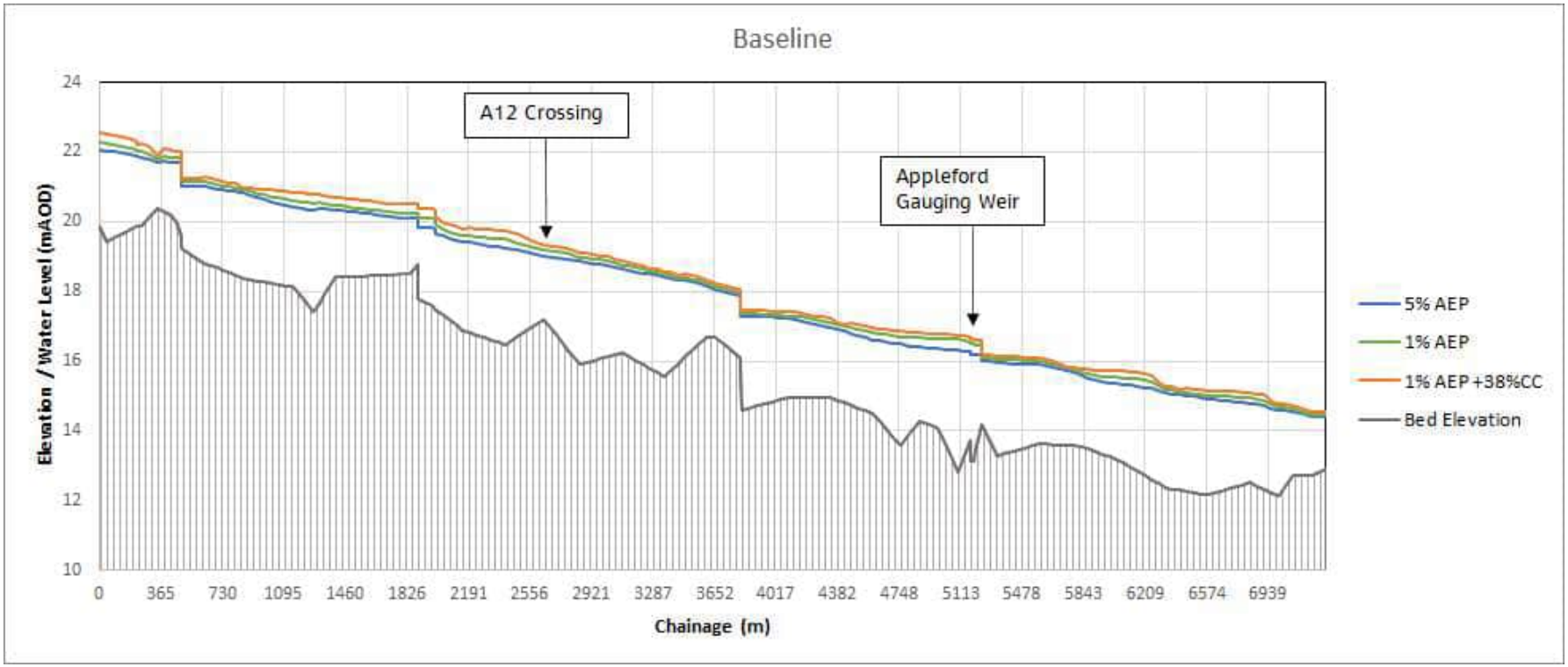
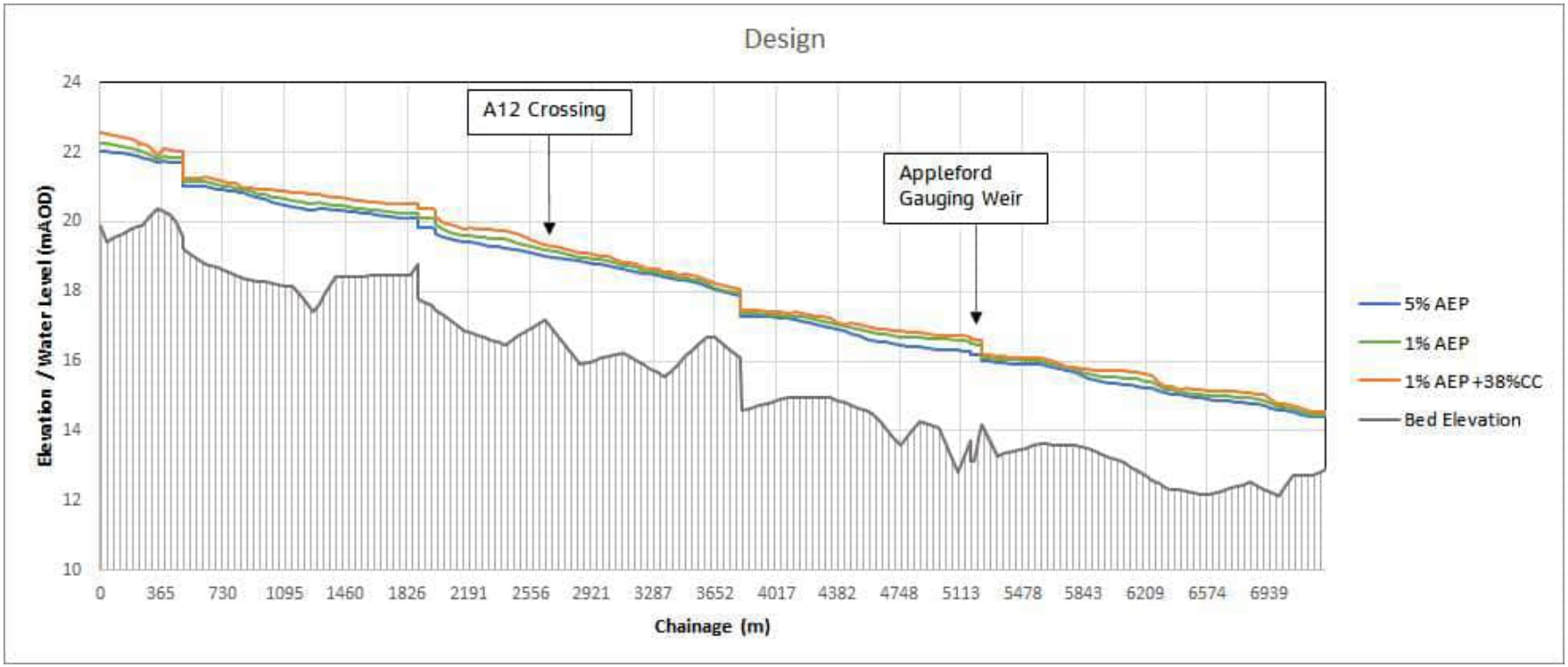


Plate C.2 With-Scheme In-channel Peak Water Levels



Appendix D **Flood Maps**

Plate D.1 With-Scheme Scenario Modelled Flood Extents

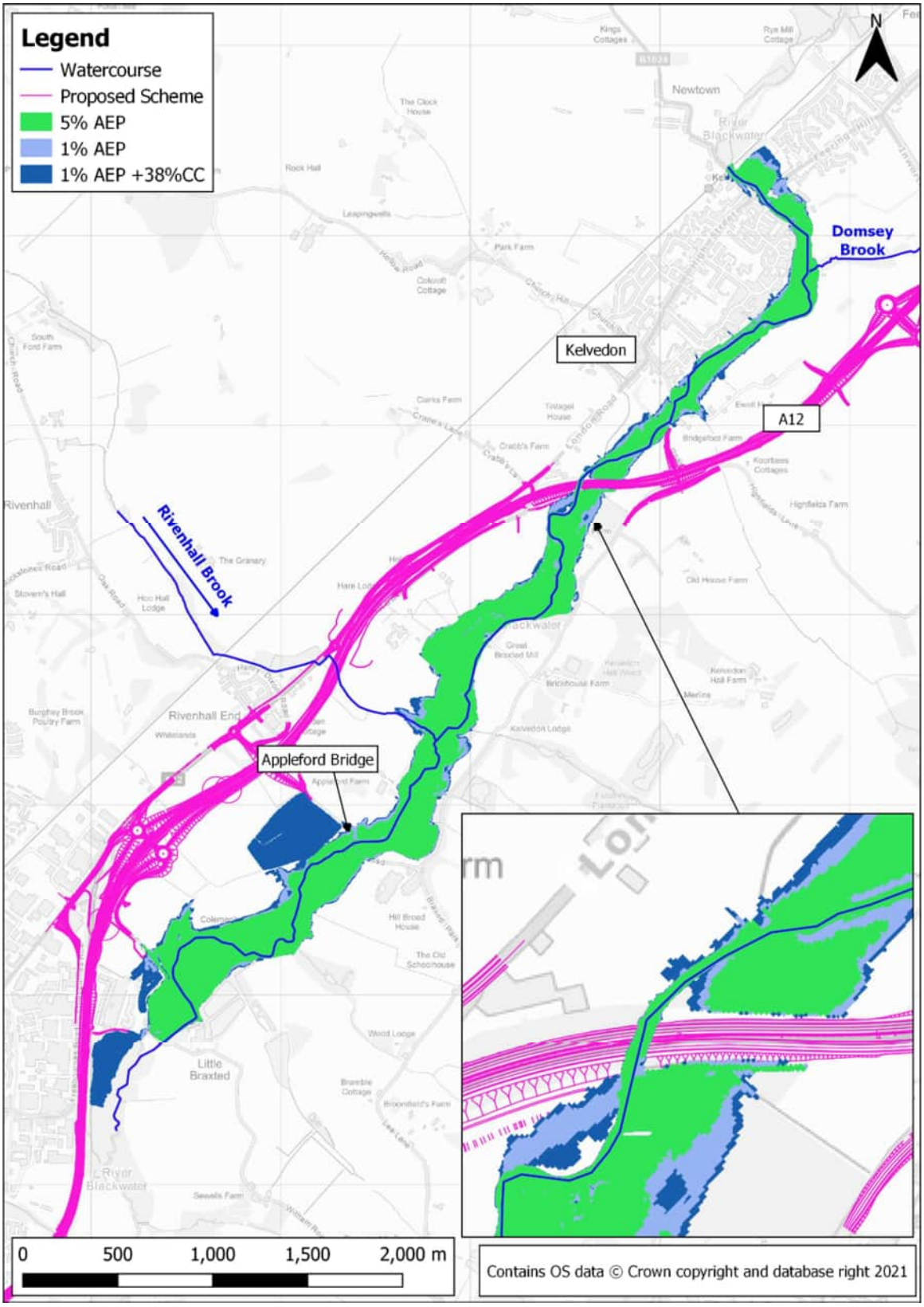


Plate D.2 Haul Roads Scenario Modelled Flood Extents

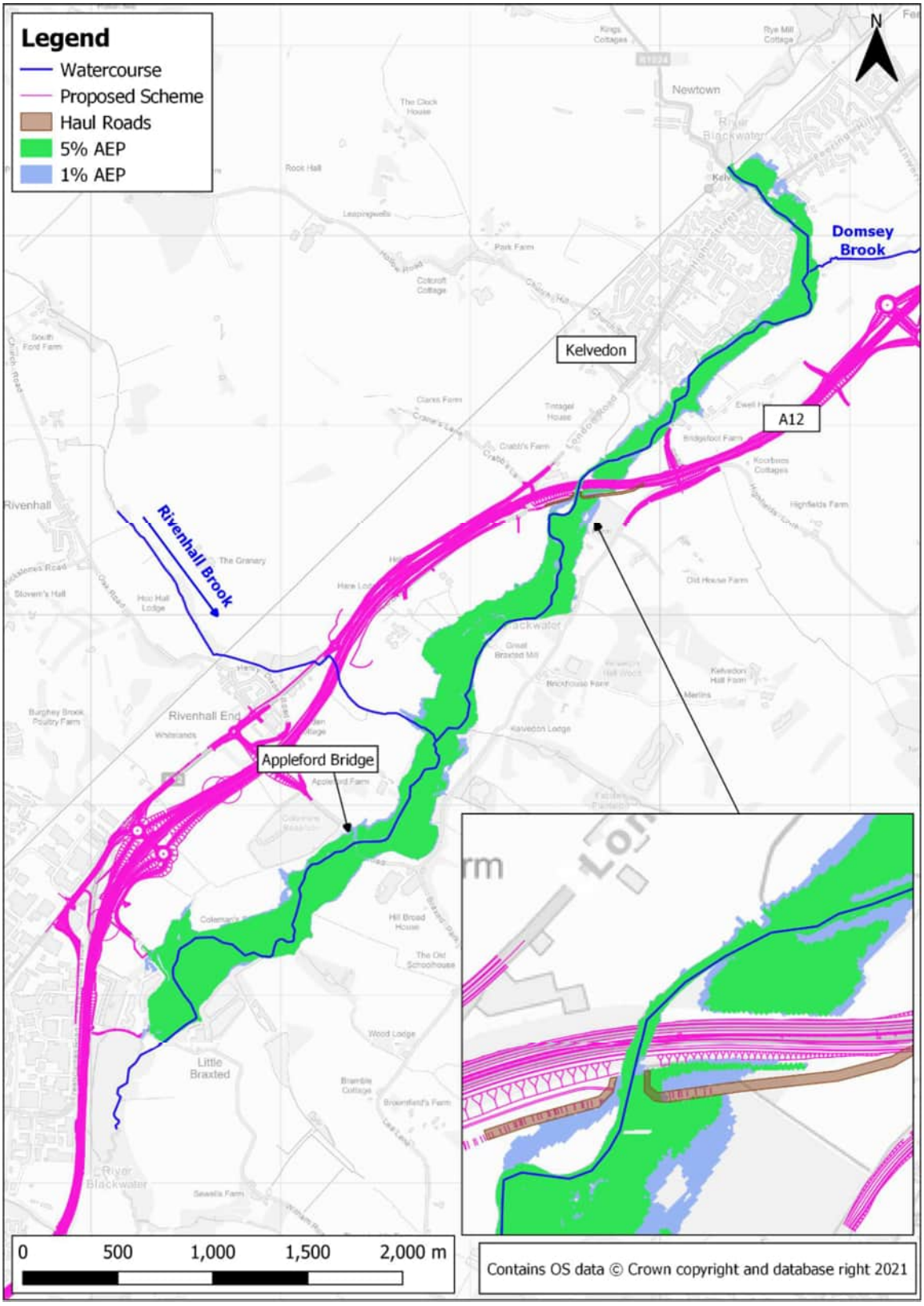


Plate D.3 Impact due to the Scheme for the 5% (1 in 20) AEP Event

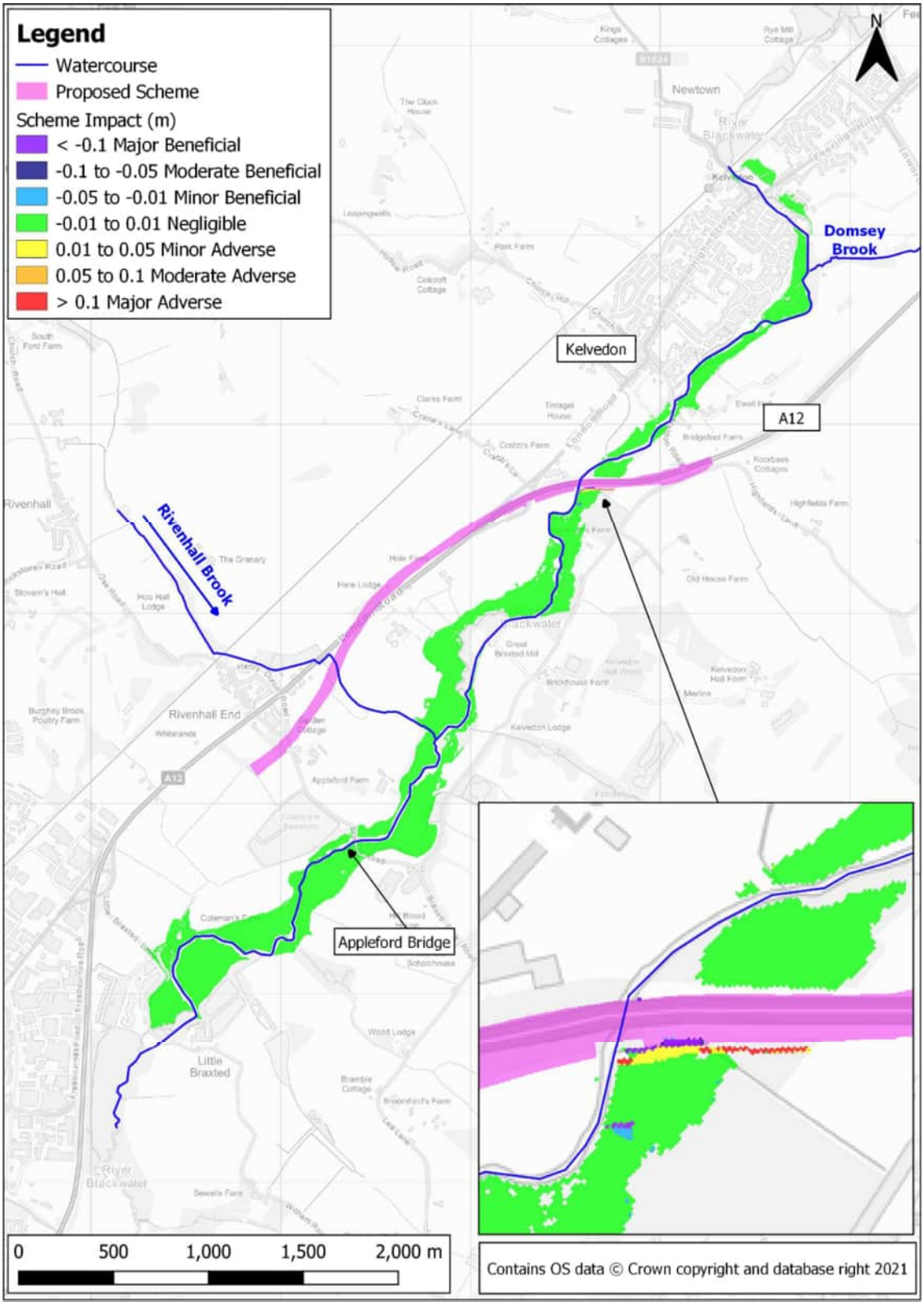


Plate D.4 Impact due to the Scheme for the 1% (1 in 100) AEP Event

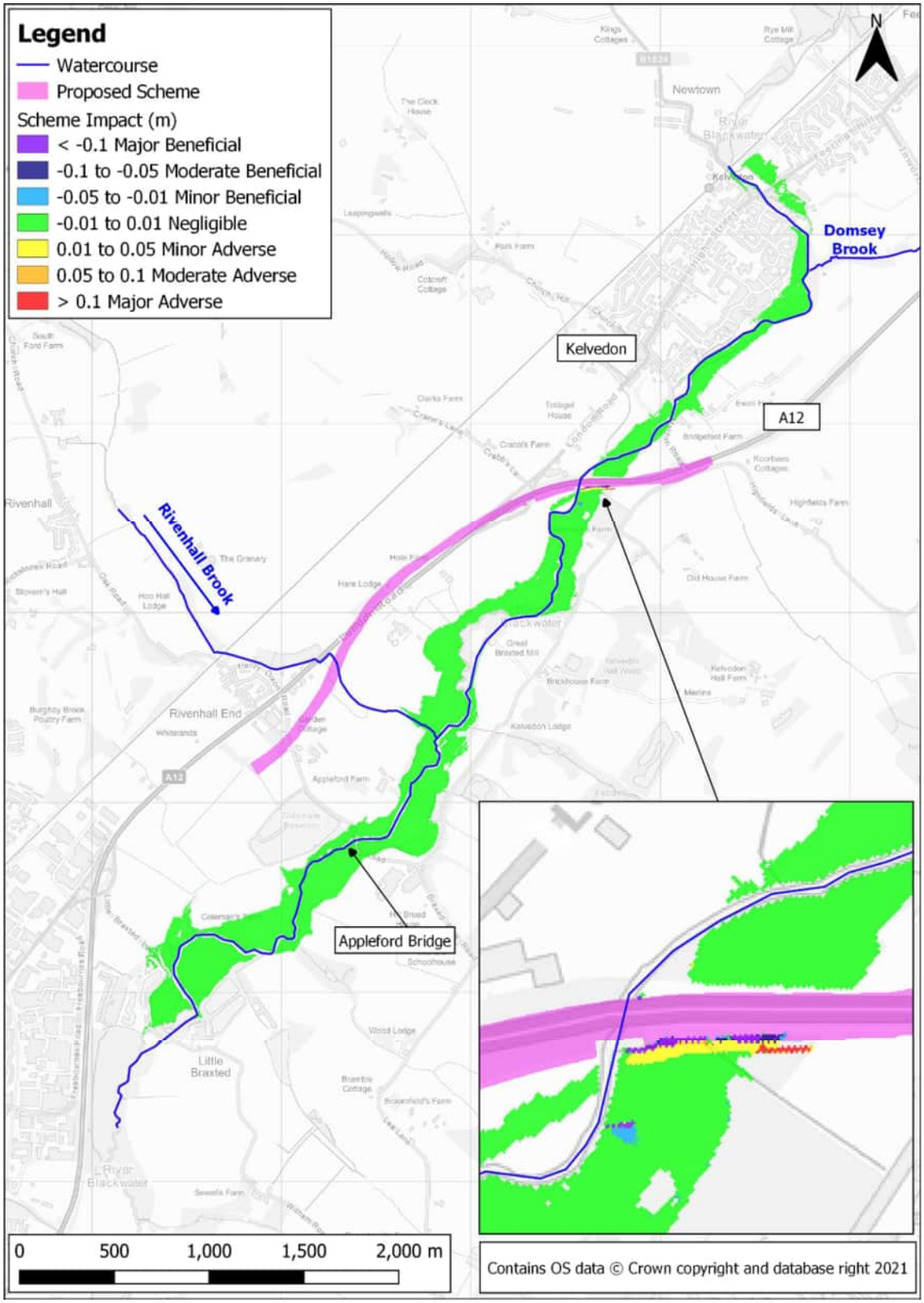


Plate D.5 Impact due to the Scheme for the 1% (1 in 100) AEP + 38%CC Event

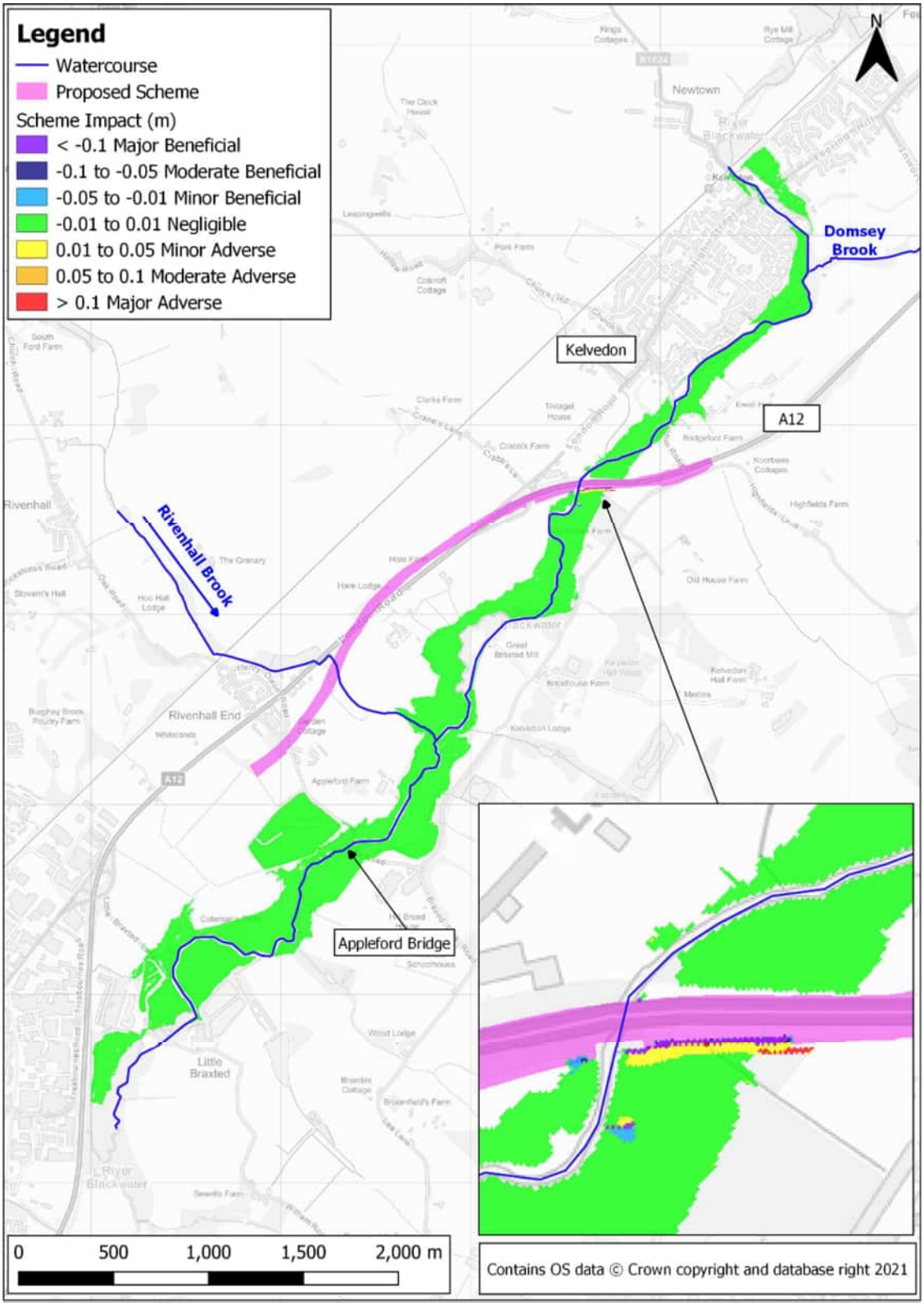


Plate D.6 Impact due to the Scheme and Haul Roads for the 5% (1 in 20) AEP Event

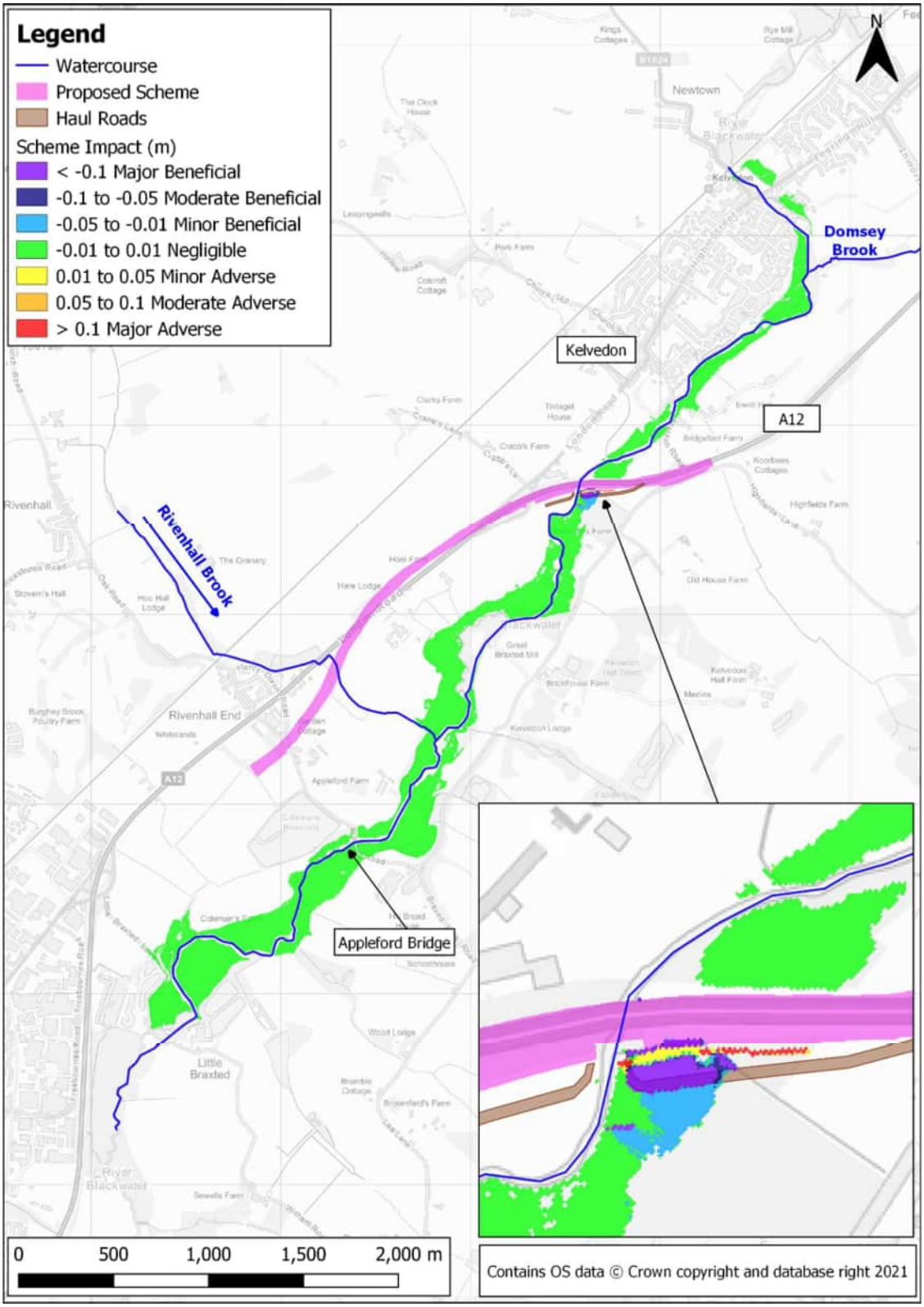
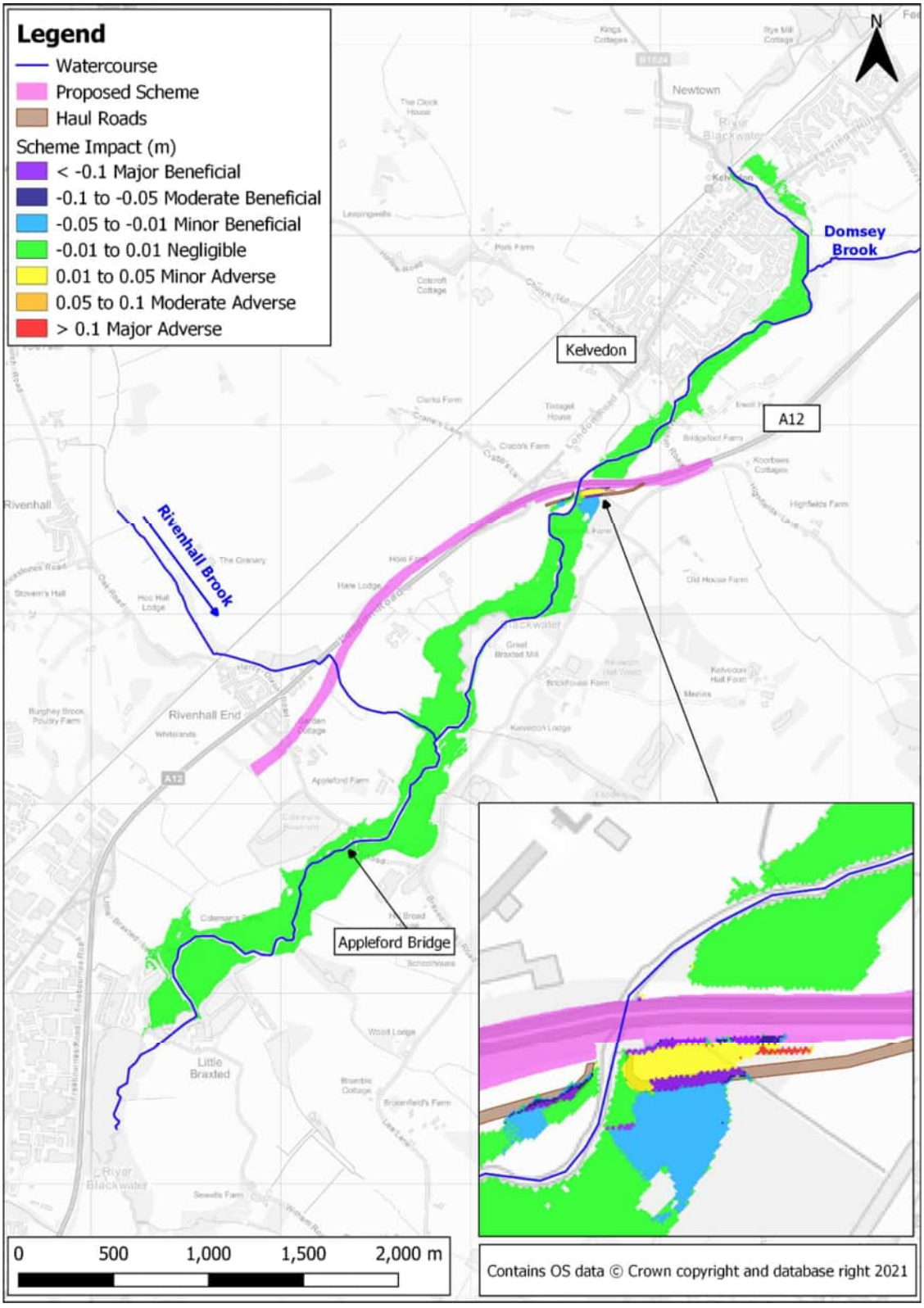


Plate D.7 Impact due to the Scheme and Haul Roads for the 1% (1 in 100) AEP Event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT RIVER BLACKWATER HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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1 Executive Summary

This document has been prepared to support the A12 Middle Blackwater with scheme hydraulic modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This record sheet is to be used where flood estimates are needed at a single location.

3 Method Statement

3.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Peak flow or hydrograph? • Range of return periods • Approx. time available	The A12 is proposed to be widened and therefore design peak flow and hydrographs are required for input into the River Blackwater and River Brain numerical hydraulic models to assess flood risk at the A12 crossing and in the vicinity of the A12 road improvements. Design peak flows and hydrographs are required for the following Annual Exceedance Probability (AEP) events: 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 1% plus climate change adjustment (plus CC), 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 100 plus CC, 200 and 1000-year design return periods). For climate change uplift the following factor will be adopted: • 1% AEP (100-year return period) event plus 38% uplift (based on the Environment Agency (EA) 2021 guidance[1] - Combined Essex Management Catchment peak river flow allowances higher, 2080s scenario). Hydraulic modelling of the River Blackwater and River Brain was undertaken by CH2M in 2017 as part of a flood mapping exercise. After a review of the CH2M hydrological analysis it has been decided to retain some of the ReFH1 method IED units for the River Blackwater model but with new ReFH1 method IED units being created at the start of the Jacobs hydraulic model on the River Blackwater and River Brain. This is due to this assessment only being required to assess flood risk in the vicinity of the A12 road improvements scheme and not for the full River Blackwater and River Brain catchments. In order to incorporate increased availability of data post-2017 assessment and updates to WINFAP software, the design peak flows in the vicinity of the scheme will also be updated for this assessment. This assessment will be undertaken using the Appleford (37010) gauge on the River Blackwater and Guithavon Valley (37009) gauge on the River Brain which are located within the Jacobs model extents. For further details refer to Annex A.2 (River Blackwater and River Brain Hydrology Update Memo - issued to the Environment Agency April 2020).

[1] Environment Agency (2021) Flood Risk Assessments: Climate Change Allowances guidance.

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The River Blackwater and River Brain are located in the south east of England. The River Blackwater has a catchment area of approximately 350km^2 to Maldon where it flows into the North Sea. The River Blackwater has a relatively rural catchment with urban areas such as Braintree, Coggeshall, Witham and Maldon. The River Blackwater has five flow / level gauges in the catchment: Copford Hall, Stisted, Appleford, Guithavon Valley (on the River Brain) and Langford. The Appleford gauge is located within the modelled section of the River Blackwater. The River Blackwater flow regime is identified (NRFA data sheet) as being affected by groundwater abstractions and intermittently affected since 1971 by the Ely/Ouse Transfer Scheme pumping. The predominant land use is arable, with areas of grassland and woodland. The geology of the catchment has been assessed as being composed of predominantly London Clay with chalk in the headwaters, overlain with boulder clay and some sand and gravels. The soils within the catchment have been assessed using Landis[2] as predominantly lime rich loamy and clayey soils with impeded drainage in the upper reaches and freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The River Brain is a tributary of the River Blackwater and has a catchment area of 67.7km^2 to its confluence with the River Blackwater just downstream of the A12 crossing. The River Brain has a moderately urbanised catchment with urban areas including Witham and Braintree. The River Brain has the Guithavon Valley (37009) gauge located within its catchment. The Guithavon Valley gauge is located within the town of Witham. The predominant land use within the catchment is arable with areas of grassland and woodland. The geology of the catchment is London Clay with superficial deposits mainly Bolder Clay and some sands and gravels. The soils within the catchment have been assessed using Landis as being composed of predominantly lime rich loamy and clayey soils with impeded drainage, freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils.

[2] http://www.landis.org.uk/soilscapes/

3.3 Source of flood peak data

Was the NRFA/HiFlows UK dataset used? If so, which version? Record any changes made.	Yes, WINFAPv4 was used with the National Rivers Flow Archive Peak Flow Dataset version 8 (downloaded March 2020). These contain the most up to date Peak Flows dataset at the time of assessment. It is noted that both the Appleford and Guithavon Amax series have been revised since the CH2M (2017) analysis. It appears that the EA have reviewed these series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. This EA reviewed AMAX data has been used.

3.4 Gauging stations (flow or level)

3.4.1 As noted above the River Blackwater has five flow/level gauges located within its catchment (Copford Hall, Stisted, Appleford, Guithavon Valley and Langford). The Appleford Gauge (37010) is located within the model extent. The River Brain further has the Guithavon Valley gauge (37009) located within its catchment within the model extent.

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
Blackwater	Appleford Bridge	037010	37010	TL844158	247.3	Level	1962 to present
River Brain	Guithavon Valley	037009	37009	TL817147	60.7	Level	1962 to present
River Pant	Copford Hall	037016	37016	TL668313	62.5	Level	1965 to present
Blackwater	Stisted	037017	37017	TL792242	139.2	Level	1969 to present
Blackwater	Langford	N/A	37004	TL835090	337	Level	1932 to 1968

3.5 Data available at each flow gauging station

Station name	Start and end of data in Peak Flows dataset	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Appleford Bridge	1962-2018	No	Yes	Yes	No	Station with poor quality record gauging a groundwater-influenced rural catchment with major artificial influences. Groundwater -influenced regime. Intermittently affected, since 1971, by Ely / Ouse Transfer Scheme pumping, significantly affected in the late 1980s and in the 1990s. Abstractions from both chalk and gravel aquifers for private water supply. Previous studies by CH2M (2017) indicates that the rating curve for this gauge may be unreliable and further spot gauging's particularly at higher flows was recommended to improve the rating. This study has investigated the availability of additional high flows spot gauging's post the CH2M (2017) study as uncertainty with higher flows was noted. At the present time no additional high flows spot gaugings are available to improve the rating. This gauge has been assessed as suitable for QMED but there is a degree of uncertainty with the higher AMAX values. It appears that the EA have reviewed since 2017 the AMAX series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. The new interpretation appears in the current Peak Flows dataset which is used in this study.

Station name	Start and end of data in Peak Flows dataset	Update for this study?	Suitable for QMED?	Suitable for pooling ?	Data quality check needed?	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Guithavon Valley	1962-2018	Yes*	Yes	Yes	No	Runoff increased by effluent returns and reduced by industrial /agricultural abstraction. Drowning noted to occur at low levels but noted to have minimum effect. A previous study by CH2M (2017) suggests further work is required to improve the confidence in the rating. This study has investigated the availability of additional high flows spot gauging's post the CH2M (2017) study but at the present time no additional high flows spot gauging's are available to improve the rating. This gauge has been assessed as suitable for QMED but there is a degree of uncertainty with the higher AMAX values. It appears that the EA have reviewed since 2017 the AMAX series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. The new interpretation appears in the current Peak Flows dataset which is used in this study. The Guithavon Valley updated QMED based on the revised AMAX data is similar to the previous CH2M QMED.
Copford Hall	1962-2018	No	Yes	Yes	No	No POT data are presented for this station as they have no non-modular flow correction applied. Intermittent pumping of Ely/Ouse Transfer Scheme has major effect on station, which is only 5km downstream of the Great Sampford Outfall. Some industrial / agricultural abstraction.

Station name	**Start and end of data in Peak Flows dataset**	**Update for this study?**	**Suitable for QMED?**	**Suitable for pooling ?**	**Data quality check needed?**	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Stisted	1969-2018	No	Yes	No	No	No POT data are present for this station as they have no non-modular flow correction applied. AMAX data have been corrected. Flows can be artificially increased by Ely Ouse transfer scheme. Urban runoff from Braintree. Minor adjustments need for ground and surface water abstractions, sewage and industrial effluent and Ely Ouse/Essex discharges. Net effect – minor export of water from catchment.
Langford	1932-1968	No	No	No	No	Quality of data unknown. Complex of weirs and sluices gauging River Blackwater and monitoring intake on what was the Southend Water Company.
Give link/reference to any further data quality checks carried out			No further data quality checks were undertaken.			

3.6 Rating equations

Station name	**Type of rating** e.g. theoretical, empirical; degree of extrapolation	**Rating review needed?**	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Appleford Bridge	N/A	No	Full rating review outside the scope of this study. The rating curve for Appleford Bridge was assessed by CH2M (2017) and Hydro-Logic (2013-2014). The CH2M study recommended that further spot gauging's should be carried out at higher flows and the rating reviewed as there were discrepancies noted in the consistency of flow estimates between gauges on the River Blackwater. This study has investigated the availability of additional high flows spot gauging's post the CH2M study as uncertainty with higher flows was noted. At the present time no additional high flows spot gaugings are available to improve the rating. It appears that the EA have reviewed since 2017 the full AMAX series and have preferred estimates that make explicit allowance for non-modularity. These adjustments have tended to alter the higher annual maximum flows on the record, and in some cases by a significant amount. The new interpretation appears in the current Peak Flows dataset which is used in this study.

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Guithavon Valley	N/A	No	Detailed rating review outside the scope of this study. A rating review was carried out for Guithavon Valley by CH2M (2017) and Hydro-Logic (2013-2014). The CH2M study noted the rating curve for Guithavon Valley was low and it recommended that further spot gaugings should be carried out particularly at higher flows and the rating reviewed. CH2M then derived a rating for their study from their hydraulic model. This resulted in an unrealistically steep growth curve. This study has investigated the availability of additional high flows spot gauging's post the CH2M (2017) study as uncertainty with higher AMAX values is noted. At the present time no additional high flows spot gaugings are available to improve the rating.
Copford Hall	N/A	No	Rating review outside the scope of this study.
Stisted	N/A	No	Rating review outside the scope of this study.
Langford	N/A	No	Rating review outside the scope of this study.
Give link/reference to any rating reviews carried out			

3.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)	No - detailed rating reviews are outwith the scope of this study. There is also insufficient high flows spot gaugings to update the current ratings at the present time. If more high flows gaugings become available, then a rating review for the Appleford (37010) and Guithavon (37009) gauges would be desirable.				
Historic flood data – give link to historic review if carried out.					

Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies (e.g. CFMPs, Strategies)					
Other data or information (e.g. groundwater, tides)					

3.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes - FEH Statistical method considered appropriate for both the River Blackwater and River Brain. As both catchments have gauges within the model extents (River Blackwater @ Appleford (37010) – and River Brain @ Guithavon Valley (37009)) both FEH Statistical ESS analysis and standard Pooling group analysis will be undertaken for the Appleford and Guithavon gauges. Since there remains some uncertainty with the robustness of the higher peak flow estimates at these gauges the steeper growth curve from the two methods (ESS and standard pooling group analyses) will be used to be precautionary. (The index flood, QMED to be based on the gauged data where there is more confidence in the flows). Corresponding peak flows will also be generated using ReFH2.3 method for comparison purposes. Neither the River Blackwater nor the River Brain are understood to have atypically large amounts of flood storage within their catchments. On this basis the peak flows will be established based upon the precautionary statistical values and the model will be run using ReFH1 inflow boundaries (as in the CH2M 2017 study) in which the storm duration is set to the default

	catchment descriptors value. These inflows will then be uniformly scaled to gain reconciliation with the statistically derived target flows. (This helps avoid the recognised[3] water balance problem associated with ReFH1 which can manifest itself when storm duration optimisation is used). Since the model being used does not need to represent all the sub-catchments upstream of where the A12 model extent starts, a new single catchment ReFH1 representation is used to supply the upstream inflow at the start of each model. For the Middle Blackwater model the new single catchment ReFH1 representation is to the point on the River Blackwater just upstream (586750 218850) of the Domsey Brook confluence. The CH2M boundary units DB1, B45, R1 are then adopted below this location. For the Lower Blackwater model the new single catchment ReFH1 representation for the River Blackwater is to the Appleford gauging station with B60 and B80 CH2M boundary units being adopted below this location. For the River Brain the single catchment ReFH1 representation is to the Guithavon Valley gauging station with the CH2M boundary unit BRA30 being adopted below this location.
Outline the conceptual model, addressing questions such as: • What is likely to cause flooding at the site (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might the site flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Flooding in the vicinity of the A12 and the Appleford and Guithavon gauging stations is judged to be due to peak flows and not sensitivity to flood volume issues. This was based upon a preliminary assessment of the results of the existing model, coupled with visual inspection of the EA flood maps for the catchments.

[3] WHS, 2016. The Revitalised Flood Hydrograph Model ReFH2.2: Technical Guidance

Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, use permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	Both catchments are long thin and relatively flat catchments. The River Blackwater catchment to the Appleford gauge has a BFIHOST (0.475) and SPRHOST (40.08) values in the middle of the normal range and do not suggest the catchment to be particularly permeable or impermeable. The River Blackwater @ Appleford (37010) catchment is classified by FEH as 'essentially rural' with only limited urban areas. The FEH Web Service records a FARL of 0.991 for the catchment to the gauge. The geology of the catchment has been noted by the FEH Web Service as having high permeability bedrock in the upper catchment (estimated as making up approximately 25% of catchment), with mixed permeability bed rock making up the remaining 75% of the catchment. The FEH catchment descriptors FPEXT value to the Blackwater @ Appleford gauge is 0.0977. This suggests the Blackwater catchment has some but limited floodplain storage within the catchment. The River Brain catchment to the Guithavon gauge has a BFIHOST (0.534) and SPRHOST (36.43) values in the normal range and do not suggest the catchment to be overly permeable. The River Brain @ Guithavon Valley (37009) catchment is classified by FEH as 'moderately urbanised' with urban areas including Witham in the lower catchment and Braintree. The FEH Web Service records a FARL of 0.997 for the River Brain to the Guithavon Valley gauging station. There is therefore little attenuation due to reservoirs/lakes within the River Brain catchments. The geology of the catchment has been noted by the FEH Web Service as being composed of mixed permeability bedrock. The FEH catchment descriptors FPEXT value to the Brain @ Guithavon Valley gauge is 0.0839. This suggests the River Brain catchment has some but limited floodplain storage within the catchment.
Initial choice of method(s) and reasons	The FEH Statistical method (ESS and Pooling) has been undertaken for both the Blackwater @ Appleford (37010) and the Brain @ Guithavon

	Valley (37009) gauges. ReFH2.3 was also undertaken for these catchments for comparison purposes. Design peak flows were then adopted following a comparison of the peak flows from the FEH Statistical methods. The most conservative of the FEH Statistical peak flows was adopted as there is uncertainty associated with the higher flows for both gauges. The design hydrograph shape is based on the output from the ReFH1 IEDs which was uniformly scaled to agree with the finalised peak flow within the model at the Blackwater @ Appleford (37010) gauge for the River Blackwater model and the Brain @ Guithavon Valley (37009) gauge for the River Brain. Flows were then checked at the target locations within the model – in the vicinity of the A12 and reconciled where appropriate.
Software to be used (with version numbers if relevant) e.g. FEH Web Service (date accessed) WINFAPv4 WINFAP-FEH v5 data files (previously HiFlows) ReFH2.2	FEH Web Service – accessed March 2020 WINFAPv4 NRFA Peak Flow Dataset - downloaded March 2020 ReFH1 – IEDs only ReFH2.3 / FEH13 rainfall data

3.9 Site details

Watercourse	Site	Easting	Northing	AREA on FEH Web service(km^2)	Revised AREA if altered (km^2)
River Blackwater	@ A12	585550	217650	223.58	223.58
River Blackwater	@ Appleford Gauge	584450	215800	249.12	249.12
River Brain	@ Guithavon Valley	581800	214750	60.27	60.27
River Brain	@ A12	582750	213700	66.64	67.65

3.10 Catchment descriptors (incorporating any changes made)

Watercourse	Site	FARL	PROPWET	BFIHOST	BFIHOST 2019	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT 2000	FPEXT
River Blackwater	@ A12	0.993	0.26	0.484	0.469	27.86	26.50	573	39.67	0.0253	0.0933
River Blackwater	@ Appleford Gauge	0.991	0.26	0.475	0.461	28.01	25.90	572	40.08	0.026	0.0977
River Brain	@ Guithavon Valley	0.997	0.30	0.534	0.514*	14.02	24.2	572	36.43	0.0737	0.0839
River Brain	@ A12	0.997	0.29	0.531	0.514	14.63	23.60	570	36.69	0.0848	0.0852

*Assessed using River Brain @ Blackwater FEH catchment descriptor as a donor for $BFIHOST_{2019}$, as value not provided in WINFAP.

3.11 Checking catchment descriptors

Record how the catchment boundary was checked and describe any changes (refer to maps if needed)	The River Blackwater catchment at the Appleford gauging station (584450 215800) and at the River Blackwater catchment at the A12 (585550 217650) were checked using ordnance survey mapping data. The River Brain catchment at the Guithavon Valley gauge (581800 214750) and at the A12 (582750 213700) were also checked using ordnance survey mapping data and amended slightly for this assessment.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	The soils within the River Blackwater catchment have been assessed using Landis which reports soils are predominantly lime rich loamy and clayey soils with impeded drainage in the upper reaches and freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The soils within the River Brain catchment have also been assessed using Landis which reports soils are predominantly lime rich loamy and clayey soils with impeded drainage, freely draining slightly acid loamy soils, slightly acid loamy and clayey soils with impeded drainage and slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. Based on this assessment the FEH SPRHOST / BFIHOST values were not amended.
Source of URBEXT	$URBEXT_{2000}$ has been extracted for the catchments from the FEH Web Service / from WINFAP (for the River Brain @ Guithavon Valley (37009)).
Method for updating of URBEXT	$URBEXT_{2000}$ amended within WINFAPv4 using urban adjustment procedures.

4 Statistical Method

4.1 Overview of estimation of QMED

Used	Condition	Approach followed
x	N >=30	Estimate QMED using annual maxima
	14=< N =<29	Estimate QMED from annual maxima & optionally adjust for climatic variation
	2=< N= <13	Estimate QMED from POT data & adjust for climatic variation
	N <2 & suitable donor site with 20 years or more of record	Ignore record at subject site; transfer QMED from donor site
	N <2 & suitable donor with 10 to 19 years of record & 12 month overlap between records	Estimate QMED using procedure based on flood peak regression
	N <2 & suitable donor with 10 to 19 years of record but no 12 month overlap	Ignore record at subject site; transfer QMED from donor site
	N <2 & no long-record site nearby	Estimate QMED from very short POT record
	N <2 & no long-record site nearby	Treat site as ungauged catchment
	N <2 & no long-record site nearby	Defer analysis until longer flow record available
	N <2 & no long-record site nearby	(Abstract flood event information and apply the UH rainfall-runoff model as an alternative, to the pooling group procedure. Particularly recommended when site is urbanised)
X	Ungauged catchment	Estimate QMED from catchment descriptors
X	Ungauged catchment	Estimate QMED by data transfer from donor catchment
	Ungauged catchment	Estimate QMED by data transfer from analogue catchment
	Ungauged catchment	Estimate QMED from channel dimensions

	Ungauged catchment	Compare to regional pattern of mapped QMED adjustment factors

4.2 Search for donor sites for QMED

Comment on potential donor sites

Mention:

- Number of potential donor sites available
- Distances from subject site
- Similarity in terms of AREA, URBEXT, FARL and other catchment descriptors
- Quality of flood peak data

Include a map if necessary. Note that donor catchments should usually be rural.

The River Blackwater has a number of flow gauges including a gauge at Stisted, Appleford and Langford. The Appleford gauging station (37010) is located within the River Blackwater model extent and has been assessed (by Peak Flows dataset) as suitable for QMED and Pooling. The AMAX data from this gauge will therefore be used in this analysis to derive QMED at the gauge and also will be used as a QMED donor for the target location the A12 (585550 217650).

The River Brain also has the Brain @ Guithavon Valley gauging station which is also assessed (by Peak Flows dataset) as suitable for QMED and Pooling. The Guithavon Valley gauging station is also located within the River Brain model extent and therefore the AMAX series from this gauge will be used for QMED estimation at the gauge and as a QMED donor at the target location the A12 (582750 213700). The 2018 water year AMAX value supplied by the Environment Agency for this study was added to the AMAX series.

The CDs and QMED at the target locations (the River Blackwater @ A12, the River Blackwater @ Appleford, the River Brain @ Guithavon Valley and the River Brain @ the A12) are presented below.

NAME	River Blackwater @ A12	River Blackwater @ Appleford	River Brain @ Guithavon Valley	River Brain @ A12
Gauge No.	-	37010	37009	-
AREA	223.58	249.12	60.27	67.65
SAAR	573	572	572	570
FARL	0.993	0.991	0.997	0.997
SPRHOST	39.67	40.08	36.4	36.69
BFIHOST	0.484	0.475	0.534	0.531
Years	-	56*	57**	-
QMEDcd's	15.4	17.1	4.6	5.2
QMED data	-	13.2	3.9	-
Ratio(R) $QMED_{data}$/ $QMED_{cds}$	-	0.767	0.838	-

**AMAX data from 2018 water year not added as noted as suspect and comment reported drift. **AMAX data from 2018 water year added.*

Method	Location	Initial estimate of QMED (m^3/s)	Data transfer				Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	
AM	River Blackwater @ A12	15.41	37010	1.31	0.736	0.823	12. 68*
AM	River Blackwater @ Appleford	-	37010	-	-	-	13.15
AM	River Brain @ Guithavon Valley	-	37009	-	-	-	3.89
AM	River Brain @ the A12	5.18	37009	1.00	0.786	0.871	4.51**

Notes
Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.
When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added below.
When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why.
The data transfer procedure is the revised one from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.
If more than one donor has been used, give the weights used in the averaging.

*Final estimate of QMED extracted from WINFAP using Appleford (37010) as donor.

**Final estimate of QMED extracted from WINFAP using Guithavon (37009) as donor.

Final estimate of Qmed from FEH Statistical method

QMED (River Blackwater @ A12) = 12.68 m^3/s

QMED (River Blackwater @ Appleford) = 13.15m^3/s

QMED (River Brain @ Guithavon) = 3.89m^3/s

QMED (River Brain @ A12) = 4.51 m^3/s

4.3 Derivation of pooling group – River Blackwater

Name of saved .feh group file: Appleford (37010)
Target return period (years) for 5T rule *: T = 100 years

*(Updated recommendations advise that a standard 500 years is used regardless of target return period i.e.T=100yr)

Target site: River Blackwater @ Appleford (gauge 37010)

For initial and final pooling group sites refer to Annex A.4.

Total number of sites	12
Total number of years	583
Total number of initial highly discordant sites	0
Discordant sites	
Total number of short records (< 7 years) removed	0
List sites removed with reasons	High SPRHOST (>0.65) – 30003, 43005
Number of pooled years after sites removed	498

Is subject site included in pooled group: yes / ~~no~~

If no state reason why: _____

Test statistics on validity of pooling group for flood frequency analysis

Heterogeneity test H2 value = 2.3468

Status			
	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable	X	2 < H2 < 4
	Review essential		H2 > 4

Value

Goodness-of-fit test	Z values	GL	acceptable / ~~not acceptable~~	0.8731
		GEV	acceptable / ~~not acceptable~~	-1.5780
		PT3	~~acceptable~~ / not acceptable	-2.4514
	other			

(Note: in the FEH the GL is the generally favored distribution for use)

ACTION is construction of flood frequency curve valid?

YES: / NO: review the pooling group further

Comment ?	No; need to add sites to increase pooling group above 500 years

4.4 Revision of Pooling Group

Revision No. 1

Station Number	Reason for changes in pooling group
20001	Added to increase pooling group above 500 years.

Number of sites 11 Years 520

Heterogeneity test H2 value = 2.0376

Status			
	Review not necessary		H2 < 1
	Review optional		1 < H2 < 2
	Review desirable	/	2 < H2 < 4
	Review essential		H2 > 4

Note: FEH Vol.3, chapter 16.3.2: “The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review.”

Goodness-of-fit test	Z values			Value
		GL	acceptable / ~~not acceptable~~	0.0746
		GEV	~~acceptable~~ / not acceptable	-2.2328
		PT3	~~acceptable~~ / not acceptable	-2.9849
	other			

ACTION	Is construction of flood frequency curve valid?
~~No~~	Yes
Comment ?	Pooled group has been reviewed, therefore proceed to construction of FFC.

Derivation of flood growth curves

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 1% AEP**
ESS	GL – L-median GEV – L-median		2.930 2.786
Pooling	GL – L-median GEV – L-median	-	2.797 2.662
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

Refer to Annex A.3 to A.5 for final pooling group details.

4.5 Flood estimates from the statistical method – Enhanced Single Site

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.438	1.745	2.066	2.174	2.265	2.533	2.760	2.930	3.372	4.037	4.610
River Blackwater @ A12*	12.7	18.2	22.1	26.2	27.6	28.7	32.1	35.0	37.2	42.8	51.2	58.5
River Blackwater @ Appleford**	13.2	18.9	22.9	27.2	28.6	29.8	33.3	36.3	38.5	44.3	53.1	60.6

**used River Blackwater @ Appleford growth curve as donor.*

***used urbanised flood frequency results.*

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.485	1.804	2.107	2.203	2.281	2.497	2.667	2.786	3.073	3.448	3.73
River Blackwater @ Appleford	13.2	19.5	23.7	27.7	29.0	30.0	32.8	35.1	36.6	40.4	45.3	49.0

4.6 Flood estimates from the statistical method – Pooling

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.408	1.693	1.992	2.093	2.178	2.427	2.638	2.797	3.208	3.827	4.360
River Blackwater @ A12*	12.7	17.9	21.5	25.3	26.5	27.6	30.8	33.4	35.5	40.7	48.5	55.3
River Blackwater @ Appleford	13.2	18.5	22.3	26.2	27.5	28.6	31.9	34.7	36.8	42.2	50.3	57.3

**used River Blackwater @ Appleford growth curve as donor.*

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.452	1.748	2.030	2.119	2.192	2.392	2.550	2.662	2.929	3.278	3.540
River Blackwater @ Appleford	13.2	19.1	23.0	26.7	27.9	28.8	31.5	33.5	35.0	38.5	43.1	46.6

4.7 Derivation of pooling group – River Brain @ Guithavon

Name of saved .feh group file: Guithavon (37009)
Target return period (years) for 5T rule *: T = 100 years

*(Updated recommendations advise that a standard 500 years is used regardless of target return period i.e.T=100yr)

Target site: River Brain @ Guithavon

For initial and final pooling group sites refer to Annex A.7.

Total number of sites	10
Total number of years	516
Total number of initial highly discordant sites	1
Discordant sites	26013
Total number of short records (< 7 years) removed	0
List sites removed with reasons	26013, 26003, 33032 – removed as permeable catchments (BFIHOST>0.65)
Number of pooled years after sites removed	401

Is subject site included in pooled group: ~~yes~~ / no

If no state reason why: Target site not included in standard pooling as moderately urbanized catchment. Subject site included in ESS analysis.

Test statistics on validity of pooling group for flood frequency analysis

Heterogeneity test H2 value = -0.0464

Status			
	Review not necessary	X	H2 < 1
	Review optional		1 < H2 < 2

Review desirable		2 < H2 < 4
Review essential		H2 > 4

					Value
Goodness-of-fit test	Z values	GL	~~acceptable~~ /not acceptable		2.7097
		GEV	acceptable /~~not acceptable~~		-0.6755
		PT3	acceptable /~~not acceptable~~		-0.4797
	other				

(Note: in the FEH the GL is the generally favored distribution for use)

ACTION is construction of flood frequency curve valid?

YES: / NO: review the pooling group further

Comment ?	No; need to add sites to increase pooling group above 500 years.

4.8 Revision of Pooling Group – River Brain @ Guithavon

Revision No. 1

Station Number	Reason for changes in pooling group
37014 / 54036	To increase pooling group above 500 years.

Number of sites	9	Years	501

Heterogeneity test	H2 value =	1.8506

Status	Review not necessary		H2 < 1
	Review optional	X	1 < H2 < 2
	Review desirable		2 < H2 < 4
	Review essential		H2 > 4

Note: FEH Vol.3, chapter 16.3.2: “The ideal pooling-group is homogeneous. However, a representative but heterogeneous pooling-group gives better flood frequency estimates than either single-site data or a pooling-group that has been made homogeneous by inappropriately removing sites. In general, it is anticipated that a significant proportion of pooling-groups will remain heterogeneous, even after review.”

				Value
Goodness-of-fit test	Z values	GL	acceptable /~~not acceptable~~	0.1297
		GEV	~~acceptable~~ /not acceptable	-2.9096
		PT3	~~acceptable~~ /not acceptable	-2.7412
	other			

ACTION Is construction of flood frequency curve valid?

~~No~~ Yes

Comment ?	Pooled group has been reviewed, therefore proceed to construction of FFC.

Derivation of flood growth curves

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 100-year return period**
ESS	GL – L-median GEV – L-median		3.674 3.501
P	GL – L-median GEV – L-median	-	2.715 2.544
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

~~No~~ Yes

Comment ?	Pooled group has been reviewed, therefore proceed to construction of FFC.

Derivation of flood growth curves

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 100-year return period**
ESS	GL – L-median GEV – L-median		3.674 3.501
P	GL – L-median GEV – L-median	-	2.715 2.544

Method: SS – Single site P – Pooled J – Joint analysis	**If P / SS, distribution used and reason for choice** **If J, details of averaging**	**If SS, parameters of distribution** (location, scale and shape)	**Growth factor for 100-year return period**
Note: Growth curves were derived using the revised procedures from Science Report SC050050 (2008).			

Refer to Annex A.6 to A.8 for final pooling group details.

4.9 Flood estimates from the statistical method - Enhanced Single Site – River Brain @ Guithavon

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.587	2.005	2.450	2.602	2.729	3.107	3.431	3.674	4.314	5.290	6.142
River Brain @ Guithavon Valley	3.9	6.2	7.8	9.5	10.1	10.6	12.1	13.3	14.3	16.8	20.6	23.9
River Brain @ the A12*	4.5	7.2	9.0	11.1	11.7	12.3	14.0	15.5	16.6	19.5	23.9	27.7

**The River Brain @ Guithavon growth curve will be used as a donor for the target site the River Brain @ A12 which is located a short distance downstream of the gauge.*

For the ESS analysis the River Brain @ Guithavon (37009) was included in the statistical analysis. This gauge is not included in the pooling group analysis due to the catchment being moderately urbanised. In order to mitigate this the L-Moments were deurbanised.

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.650	2.089	2.515	2.651	2.763	3.075	3.324	3.501	3.932	4.508	4.950
River Brain @ Guithavon Valley	3.9	6.4	8.1	9.8	10.3	10.7	12.0	12.9	13.6	15.3	17.5	19.3

4.10 Flood estimates from the statistical method – Pooling – River Brain @ Guithavon

GL	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.436	1.721	2.006	2.099	2.176	2.398	2.581	2.715	3.053	3.536	3.932
River Brain @ Guithavon Valley	3.9	5.6	6.7	7.8	8.2	8.5	9.3	10.0	10.6	11.9	13.8	15.3
River Brain @ the A12	4.5	6.5	7.8	9.0	9.5	9.8	10.8	11.6	12.2	13.8	15.9	17.7

**The River Brain @ Guithavon growth curve will be used as a donor for the target site the River Brain @ A12 which is located a short distance downstream of the gauge.*

The River Brain @ Guithavon (37009) is not included in the FEH Statistical pooling analysis due to the moderately urbanised catchment.

GEV (for comparison only)	Flood peak (m^3/s) for the following return periods (in years)											
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**200**	**500**	**1000**
Growth Factor	1.000	1.486	1.776	2.033	2.110	2.171	2.336	2.460	2.544	2.736	2.968	3.128
River Brain @ Guithavon Valley	3.9	5.8	6.9	7.9	8.2	8.4	9.1	9.6	9.9	10.6	11.5	12.2

5 Revitalised flood hydrograph (ReFH2.3) method

5.1 Parameters for ReFH2.3 model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible so it is not essential to enter them in the table.

Location	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
River Blackwater @ A12	CD	21.1	399	74.9	2.37
River Blackwater @ Appleford	CD	21.3	391	74.2	2.27
River Brain @ Guithavon Valley	CD	12.4	435	63.9	2.76
River Brain @ the A12	CD	13.4	438	65.5	2.74
Brief description of any flood event analysis carried out (further details should be given below or in a project report)		No flood event analysis carried out.			

5.2 Design events for calculating peak flow using ReFH2.3 method

Location	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
River Blackwater @ A12	Rural	Winter	33.0	Catchment area
River Blackwater @ Appleford	Rural	Winter	33.0	Catchment area
River Brain @ Guithavon Valley	Rural	Winter	18.0	Catchment area
River Brain @ the A12	Rural	Winter	22.0	Catchment area

5.3 Peak flood estimates from the ReFH2.3 method (based upon the sub-catchment specific critical storm durations)

Using ReFH2.3 Duration	Flood peak (m^3/s) for the following return periods (in years)											
	2	5	10	20	25	30	50	75	100	200	500	1000
River Blackwater @ A12	17.9	23.1	26.5	30.0	31.2	32.1	35.2	38.0	40.4	48.5	63.1	74.8
River Blackwater @ Appleford	20.2	26.0	29.9	33.8	35.1	36.2	39.6	42.8	45.5	54.6	71.1	84.3
River Brain @ Guithavon Valley	5.7	7.6	8.9	10.2	10.6	11.0	12.2	13.3	14.2	17.5	23.5	28.3
River Brain @ the A12	6.2	8.2	9.6	11.0	11.4	11.8	13.1	14.2	15.2	18.7	25.0	30.1

6 Discussion and summary of results

6.1 Comparison of results from different methods

This table compares peak flows from the two methods for two key return periods.

	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	FEH Statistical ESS	FEH Statistical Pooling	ReFH2.3	FEH Statistical ESS	FEH Statistical Pooling	ReFH2.3
River Blackwater @ A12	12.7	12.7	17.9	**37.2**	35.5	40.4
River Blackwater @ Appleford	13.2	13.2	20.2	**38.5**	36.8	45.5
River Brain @ Guithavon Valley	3.9	3.9	5.7	**14.3**	10.6	14.2
River Brain @ the A12	4.5	4.5	6.2	**16.6**	12.2	15.2

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	ReFH2.3 generally produces higher peak flows than the FEH Statistical methods (ESS and Pooling) for the River Blackwater and River Brain catchments. The River Blackwater @ Appleford gauge and the Brain @ Guithavon gauges are however assessed as suitable for estimating QMED. Both gauges have been assessed as also suitable for pooling however there is less confidence in higher flows estimates at the gauges. Therefore, it has been decided to take forward the higher FEH Statistical ESS peak flow estimated up to and including the 1% AEP event (100-year return period) and then use ReFH ratios for all flow estimates above the 1% AEP event. In order to be consistent with the current ReFH1 method based IED boundary units used within the Blackwater hydraulic model, the hydrograph shapes are derived from the application of ReFH1. The use of ReFH1 boundary units was assessed and judged to be acceptable for deriving hydrograph shape for these catchments. This was based on the assessment that there is limited floodplain storage within the modelled reach and as there appears to be limited catchment change post the 2017 study. This was also supported by the results of the ReFH2.3 hydrograph shape sensitivity analysis undertaken as part of the current

	study within the Middle Blackwater model. The results of the sensitivity analysis indicated that for 1% AEP (100-year return period) event the ReFH1 hydrograph shape (scaled to the final peak flow estimates) produced slightly conservative flood extent outline at the A12 crossing location of the River Blackwater in comparison to the corresponding flood extent when the ReFH2.3 flood hydrograph shape was adopted. For further details of the sensitivity analysis refer to Annex A.11. For the Middle Blackwater model one new ReFH1 boundary unit was created at the top of the Jacobs model extent just upstream of the Domsey Brook confluence with the River Blackwater (586750, 218850), as modelling is only required in the vicinity of the A12. The IEDs for the lateral inflows (DB1, B45, R1) from the original River Blackwater model were retained. For the Lower Blackwater and Brain model a new ReFH1 boundary unit was created at the top of the Jacobs model extent on the River Blackwater at the Appleford gauging station. The IEDs for the River Blackwater lateral inflows B60 and B80 were retained. A new ReFH1 boundary unit was further created for the River Brain to the Guithavon Valley gauging station with the CH2M BRA30 ReFH1 lateral inflow retained. The final design peak flows at the River Blackwater @ Appleford (37010) and River Brain @ Guithavon (37009) gauges are suitable to use as a target flow to reconcile the modelled flow at the gauges. A check on the peak flow at the target locations; the River Blackwater @ A12 and the River Brain @ A12 was also carried out and adjustments considered where necessary.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions used in this study are as follows: • ReFH1 hydrograph shapes are suitable to use in this investigation. • ReFH2.3 is likely to be slightly overestimating lower return periods design peak flows for the target catchments (based on gauged data for the River Blackwater @ Appleford and River Brain @ Guithavon gauges). • The River Blackwater @ Appleford and River Brain @ Guithavon gauges are suitable to estimate design peak flows up to the 1% AEP (100-year return period) event. • ReFH2.3 ratios are more appropriate from the 0.5% AEP (200-year return period) event.

Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH **3** 12.5 or the factorial standard error from Science Report SC050050 (2008).	QMED for the River Blackwater @ Appleford (37010) gauge and River Brain @ Guithavon (37009) gauge has been estimated using AMAX data from the gauges. It is assumed that the AMAX data at both gauges is suitable for QMED.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	The design peak flow estimates have been derived specifically for the River Blackwater and River Brain for modelling of potential impacts of A12 road improvements scheme. Before the flows are used in future studies there should be an assessment of their suitability for the issue in question. Flow derivation methods change in time and this should be taken into consideration.
Give any other comments on the study, for example suggestions for additional work.	If additional high flows spot gaugings become available, the ratings for the River Blackwater @ Appleford (37010) and River Brain @ Guithavon (37009) gauges should be reviewed. There is some uncertainty regarding the higher flow estimates at these gauges.

6.4 Checks

What do the results imply regarding the return periods of floods during the period of record?	River Blackwater @ Appleford (37010) gauge: the biggest recorded flow (22 Oct 2001) maps on to 0.2% AEP (500-year return period) event with an estimated flow of approximately 61m^3/s. Though the NFRA web site says "*Exceptionally high flow recorded in Oct 2001, magnitude suspect as likely to have been affected by drowning*". There are 56-years of AMAX data for this gauge. The second AMAX value (27.9 m^3/s recorded on 11th February 2009) has a rarity of approximately a 5% AEP (20-year return period) event. River Brain @ Guithavon (37009) gauge; the biggest recorded flow (21 Oct 2001) maps onto a 0.2% AEP (500-year return period) event with an estimated flow of 25.6m^3/s. There are 57-year's worth of AMAX data for this gauge. The second AMAX value (12.5 m^3/s recorded on 10th February 2009) has a rarity of approximately a 2% AEP (50-year return period) event. Caution: the findings of this study suggests that both the gauges struggle to provide robust flow estimates at the highest

	flows in the record. This suggests that the above rarities are treated with considerable caution.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	The 1% AEP (100-year return period) growth factor for the River Blackwater @ Appleford gauge is 2.93. The growth factor is quite high but within the expected range. The 1% AEP (100-year return period) growth factor for the River Brain @ Guithavon gauge is 3.67. This growth factor is very high and is considered to be towards the upper end of the possible range.
If 1000-year flows have been derived, what is the ratio for 1000-year flow over 100-year flow?	The ratio of the 0.1% AEP (1000-year return period) flow to the 1% AEP (100-year return period) flow for the River Blackwater @ Appleford gauge is 1.85. The ratio of the 0.1% AEP (1000-year return period) flow to the 1% AEP (100-year return period) flow for the River Brain @ Guithavon gauge is 1.99.
What specific runoff (l/s/ha) does the design flow equate to?	The corresponding 1% AEP (100-year) and 50% (2-year) specific runoffs for the River Blackwater @ Appleford catchment are 1.551 l/s/ha and 0.53 l/s/ha respectively. The corresponding 1% AEP (100-year) and 50% (2-year) specific runoffs for the River Brain @ Guithavon catchment are 2.37 l/s/ha and 0.65 l/s/ha respectively.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	Peak flow estimates are available from a previous study of the River Blackwater and River Brain catchments undertaken by CH2M (2017). The CH2M study derived a 1% AEP (100-year return period) flow of 32.9 m^3/s for the River Blackwater @ Appleford gauge. (The current study 38.5 m^3/s). The CH2M study derived a 1% AEP (100-year return period) flow of 20.2 m^3/s for the River Brain @ Guithavon gauge. (The current study 14.3 m^3/s). There is some uncertainty regarding the higher flow estimates at the River Blackwater @ Appleford and River Brain @ Guithavon gauges. In the CH2M (2017) study the rating for the River Brain @ Guithavon gauge was derived from their hydraulic model due to concerns regarding the rating. The resulting CH2M growth curve for the River Brain @ Guithavon gauge was however unconvincingly steep.
Are the results compatible with the longer-term flood history?	The modelled flows can be validated within the hydraulic model using readily available flood extents for the River Blackwater and River Brain where available.

Describe any other checks on the results	

6.5 Final results

Flood peak (m^3/s) for the following return periods (in years) – FEH Statistical ESS and ReFH2.3 Ratios above 1% AEP event													
	2	**5**	**10**	**20**	**25**	**30**	**50**	**75**	**100**	**100+38%CC Uplift**	**200**	**500**	**1000**
River Blackwater @ A12	12.7	18.2	22.1	26.2	27.6	28.7	32.1	35.0	37.2	51.3	44.5	58.0	68.8
River Blackwater @ Appleford	13.2	18.9	22.9	27.2	28.6	29.8	33.3	36.3	38.5	53.2	46.2	60.1	71.4
River Brain @ Guithavon Valley	3.9	6.2	7.8	9.5	10.1	10.6	12.1	13.3	14.3	19.7	17.6	23.6	28.5
River Brain @ the A12	4.5	7.2	9.0	11.1	11.7	12.3	14.0	15.5	16.6	22.9	20.4	27.4	33.0

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	Design peak flows have been derived for the target locations for the River Blackwater and River Brain hydraulic models. The hydrographs were created using ReFH1 inflow boundary units within the hydraulic models. Modelled flows have been reconciled to agree with the final peak flow estimates presented within this report at the River Blackwater @ Appleford and River Brain @ Guithavon gauges. The modelled peak flows at the target locations the River Blackwater @ A12 and River Brain @ A12 were further checked to ensure flows are appropriate and adjusted where necessary. It should be noted that the ReFH1 boundary units should be run using the design storm duration to the target locations (the A12 scheme). This results in a duration of 29.7 hours for the River Blackwater model and a design duration of 15.9 hours for the River Brain.

Acronyms

Term	Definition
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
ESS	Enhanced Single Site flood frequency analysis
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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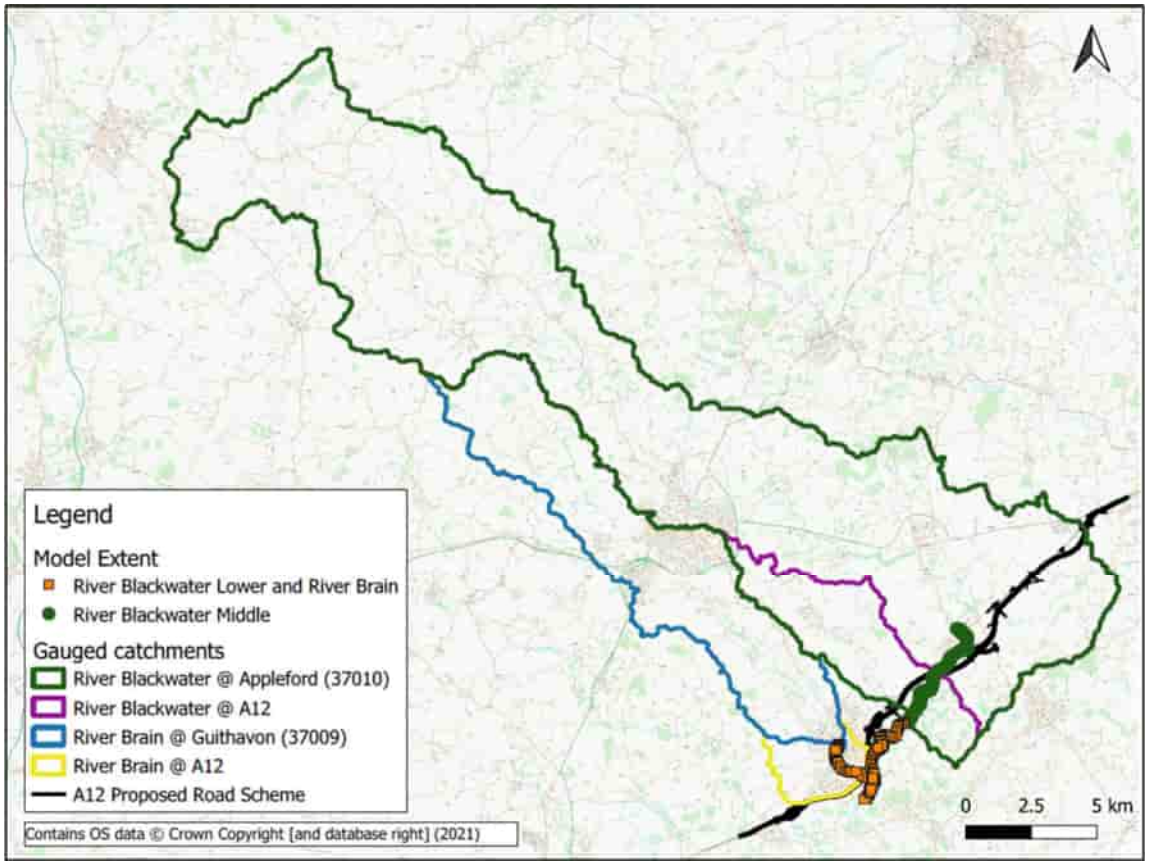
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Annex A - Supporting Information

A.1 Location of catchments



A.2 River Blackwater and River Brain Hydrology Update - A12 Road Widening Scheme (issued to the Environment Agency April 2020)

A review of the hydrology for the River Blackwater Middle hydraulic model and the River Blackwater Lower (including River Brain) hydraulic model has been undertaken to assess whether the CH2M (2017) design flows and hydrographs are suitable to use as input into the Jacobs hydraulic models for the A12 Road Widening Scheme DMRB Stage 3 assessment.

The rivers Blackwater and Brain both have well positioned flow gauges that appear within the Peak Flows dataset. The Appleford gauge on the Blackwater is located within the extent of the hydraulic model, as is the Guithavon gauge on the Brain. The use of these two gauges was central within the CH2M analysis and their use has been the focus of this review which has considered the following:

- the availability of additional new data to refine the understanding (both new spot gaugings and additional annual maximum data),
- the appropriateness of the estimated flows within the annual maximum (Amax) series,
- the methods used to estimate design peak flows,
- improvements in recent updates to software (WINFAP and ReFH) since the 2017 assessment.

The key findings of the review are:

i) No new high flow spot gaugings have been taken since the CH2M (2017) hydrological analysis that can help refine the understanding of the high flows performance of either the Blackwater @ Appleford or the Brain @ Guithavon gauges.

ii) Six additional years of annual maximum data now exist for both gauges (not available to the CH2M (2017) study). It is recommended that these be included in any analysis updating the design flows. However, this by itself is thought unlikely to greatly change the resulting design flows since none of the recent flows are particularly large.

iii) The CH2M study used an annual maximum series for the Appleford gauge that differs to the most recent version of the annual maximum series that now appears on the Peak Flows database. In the CH2M study two versions of the Amax series were actually considered: called the “Hiflows” and the “Hydro-Logic” Amax series. The Hiflows series is understood to have been the favoured series at the time that appeared in the Hiflows dataset which was the predecessor to the Peak Flows dataset. The Hydro-Logic series was one based on a revised provisional rating developed by Hydro-Logic following their

rating review (Hydrologic, 2014)[4]. Figure 1 compares the Gringorten plotting positions of these Amax series, together with the single site statistical analysis undertaken by CH2M and reported in 2017.

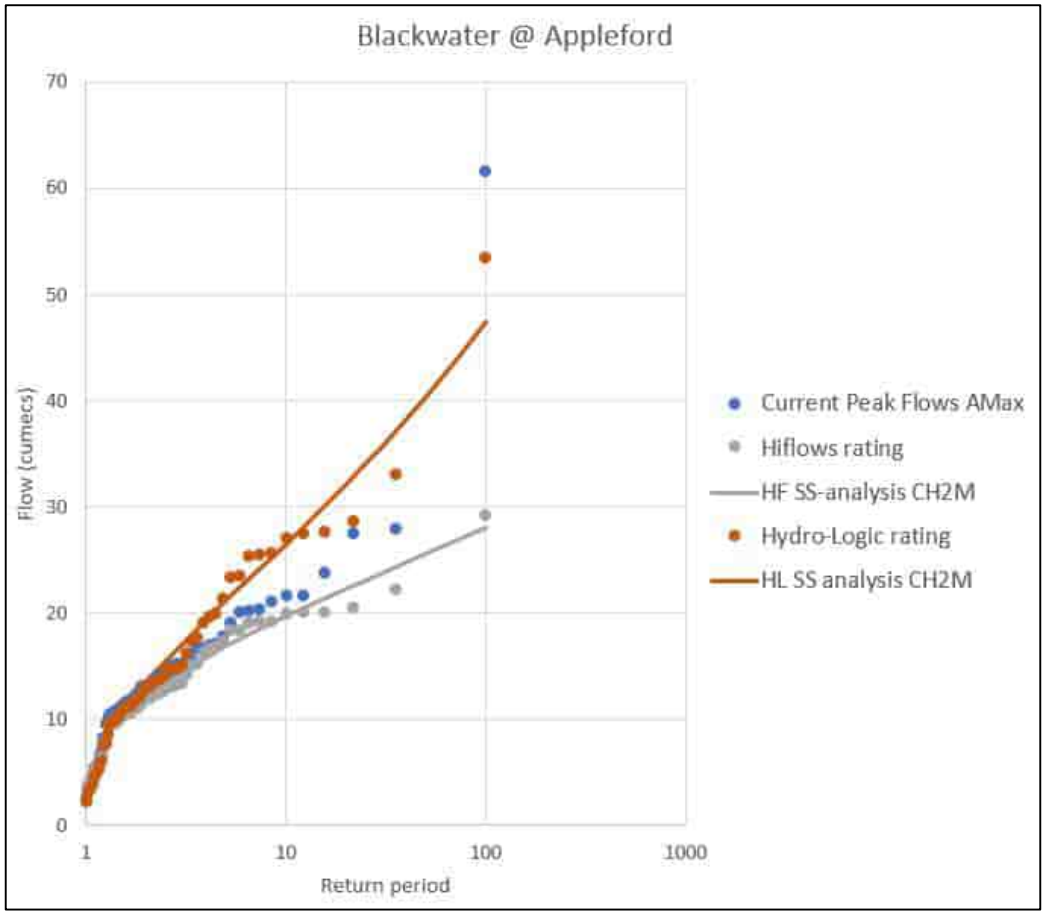


Figure 1 Comparison of the Gringorten plotting positions of the current Peak Flows Amax series to those of the Amax series considered by CH2M in the 2017 study

The Hydro-Logic rating review suggested that the gauge should provide reasonable estimates of Qmed, and that crest-tapping measurements were used to adjust for non-modular flow conditions, plus that an additional overflow spillway contribution should also be included at the highest of flows. However Hydro-Logic were at the time unable to review these aspects as the background information was unavailable. At the time they suggested a provisional revision to the general rating though this was not particularly convincing at higher flows when compared to the available spot gaugings. They recommended further spot gauging of high flows to help refine the understanding. However, the

[4] Hydro-Logic, 2014. Review of performance of gauging Station 2013-14 River Blackwater at Appleford Bridge. Report for Environment Agency, Anglian Region.

CH2M study rejected the Hydro-Logic rating as it resulted in a flood growth curve that they considered to be excessively steep (refer to Figure 1).

iv) The Guitavon Amax values that now appear in the Peak Flows database also differ to those that were available on the Hiflows database at the time of the CH2M (2017) study (Figure 2). The full period of record Amax series has recently been reviewed by the EA (released September 2019). The difference is in part thought to be based on the use of crest tappings to improve the estimate when the flow becomes non-modular. These amendments seem not to have been applied in the earlier Hiflows series.

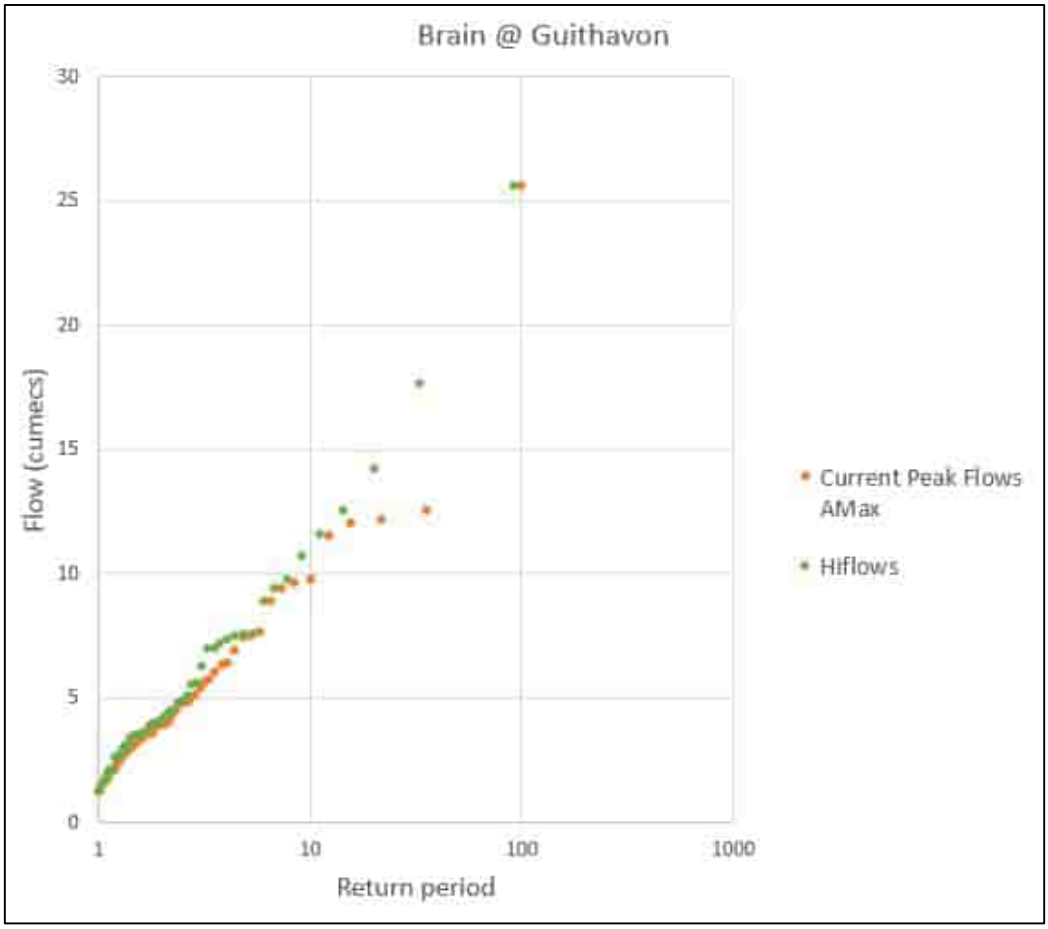


Figure 2 Comparison of the Gringorten plotting positions of the current Peak Flows Amax series to those of the Hiflows Amax series available to the CH2M (2017) study

The Hydro-Logic (2014) rating review[5] concluded that there were "insufficient high flow gaugings with which to draw any meaningful conclusions about the high flow rating i.e. greater than stage of 1 m". However, there are a reasonable number of spot gaugings for smaller floods and the gauge is therefore likely to provide a reasonable estimate of Qmed.

In the CH2M study it was noted that there was uncertainty about the adequacy of the station's rating curve (and hence the Amax series). They derived an alternative rating from their hydraulic model. They did however note: "the hydraulic model is a broadscale catchment model and therefore not ideally suited to deriving detailed rating curves". This lowered the highest Amax values by up to 15% whilst leaving the Qmed little changed and was preferred in the study to the Hiflows Amax series.

It is beyond the resources of the current project to undertake a further detailed gauge review. However, the indications are that the gauge will supply a reasonable estimate of Qmed. There was uncertainty at the time of the CH2M analysis as to what represented the most accurate Amax series (particularly for the higher flows). Since then the Amax series appearing in Peak Flows has been revised by the EA which makes allowances for non-modular flow. It is therefore recommended that this Peak Flows Amax series be used in preference to that relied upon in the CH2M study. However, we consider that residual uncertainty remains, and recommend that the analysis needs to be wary of placing too much reliance on the accuracy of the gauge's higher flows.

v) In the CH2M (2017) study the design peak flows for the Blackwater at Appleford gauge were based upon Enhanced Single Site (ESS) analysis (using the Hiflows rating) augmented above the 1% AEP by application of ReFH ratios.

   Based on ii and iii above this review recommends that the estimate of design peak flows at the Blackwater at Appleford gauge be estimated by both up-dated and revised ESS and pooling group in which the Qmed is taken directly from the gauge. Given there remains a degree of uncertainty concerning the higher Amax values it is considered prudent to take the higher of the two sets of estimates to be the design flows. Extension of the flood frequency curve above the 1% AEP using the ratios of ReFH is considered acceptable. We will in addition to the statistical estimates also provide single catchment estimates from the latest ReFH version (2.3) for comparison.

vi) The CH2M (2017) flood growth curve used for the River Brain was remarkably steep. For example: the1% AEP growth factor was marginally over 5 (yet the normal range tends to be between 2 and 3.5). This is an exceptionally steep relationship and one that merits further consideration. Based on ii and iv above this review recommends that the design peak flows at the Brain at

[5] Review of the performance of gauging stations 2013-14 Guithavon Valley. Report for Environment Agency, Anglian Region

Guithavon station be recalculated using the up-dated and revised annual maximum series via both ESS and pooling group analysis in which the Qmed is taken directly from the gauge. Given there remains a degree of uncertainty concerning the higher Amax values it is considered prudent to take the higher of the two sets of estimates to be the design flows. Extension of the flood frequency curve above the 1% AEP using the ratios of ReFH is considered acceptable. We will in addition to the statistical estimates also provide single catchment estimates from the latest ReFH version (2.3) for comparison.

Proposed analysis\approach:

Based upon the above key findings the following steps are proposed for revising the CH2M (2017) design flood hydrology at those locations where the A12 crosses or potentially lies on the flood plain of the rivers Blackwater and Brain.

Step 1: Revise the statistical estimates of the peak flows.

Following the recommendations given in v and vi above the design flows at the two gauges will be re-estimated. Design flows for target locations upstream or downstream of the gauges will be calculated using standard donor principals.

Step 2: Compare the runoff rates per unit area.

The resulting runoff rates (l/s/ha) over a range of return periods will be compared to each other and to those of adjacent similar catchments also subject to analysis on the A12 project. Any differences will be interpreted.

Step 3: Inflows to the hydraulic flood model.

The ReFH1 IEDs from the 2017 study will be retained in the flood model and will be the means of supplying the necessary inflow design hydrographs, with the exception that new ReFH1 IEDs will be created at the top of the Jacobs model extent to represent as a single catchment the upstream area. (Since the full extent of the River Blackwater and River Brain models do not require to be run for the specific locations of interest to the A12 Road Widening assessment).

Step 4: Design event runs

Critical storm durations will be established from the ReFH default storm duration equation for the location of the target site. The same design storm will be applied to all the catchments modelled. The modelled flows will be reconciled at the target locations of interest to the preferred statistical flows established in Step 1. This will be done by uniformly scaling the inflows such that the routed flow at the target location equals the design flow obtained in Step 1. The design flows and hence storm on the Brain above where the two rivers interact will be the subject of a separate storm event appraisal and flood model analysis.

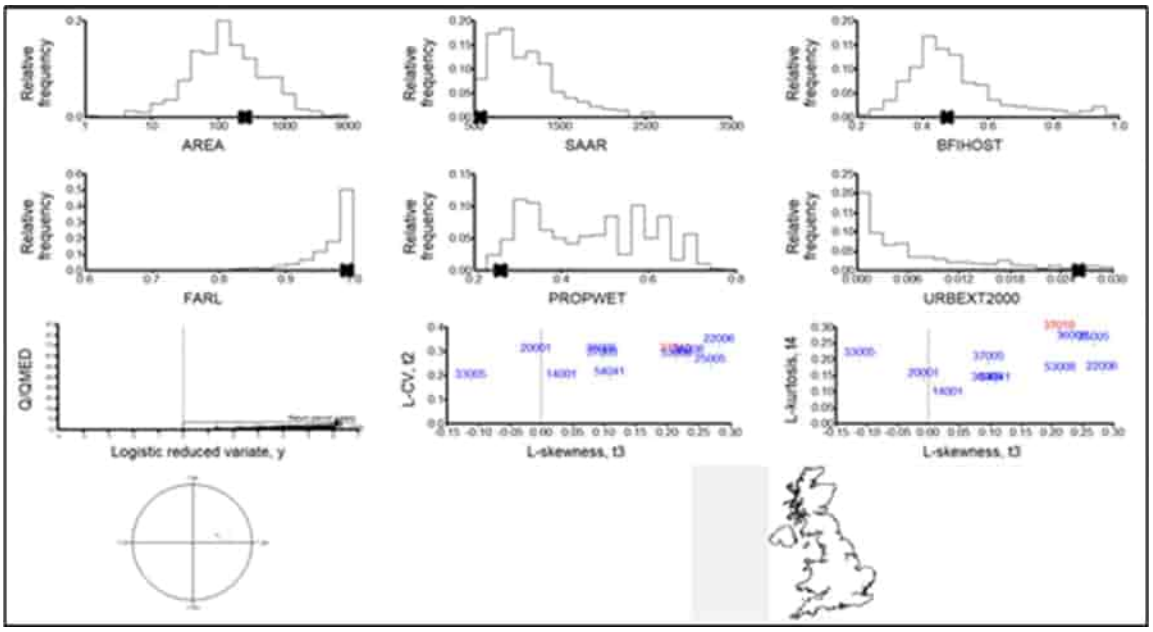
Step 5: Validation of the modelled design floods

Design levels and flood extents will be compared to observed flood levels and extents where readily available as a check upon the acceptability of the design runs.

Requested action:

The Environment Agency is requested to confirm they agree with the proposed hydrological approach for the River Blackwater and River Brain for the A12 Road Widening Scheme.

A.3 Pooling Group Details – Graphs – River Blackwater @ Appleford



A.4 Pooling Group Details – Table- River Blackwater @ Appleford

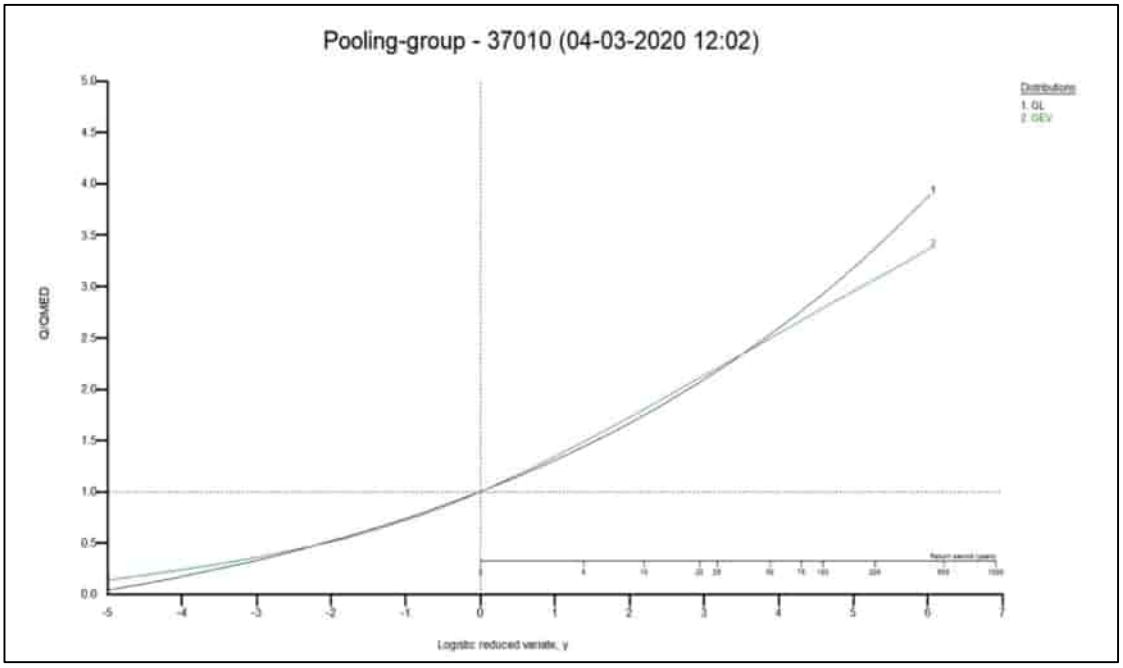
Initial Pooling Group Details: (show default selection) at River Blackwater @ Appleford

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37010 (Blackwater @ Appleford Bridge)	0	56	13.15	0.285	0.213	1.138	0	249.12	572	0.098	0.991	0.026
37005 (Colne @ Lexden)	0.287	58	12.4	0.268	0.097	0.351	0.287	235.88	566	0.076	0.97	0.022
36008 (Stour @ Westmill)	0.368	58	17.395	0.28	0.233	0.647	0.368	222.82	589	0.068	0.994	0.023
22006 (Blyth @ Hartford Bridge)	0.441	58	52.591	0.322	0.283	0.659	0.441	273.67	696	0.115	0.99	0.009
30003 (Bain @ Fulsby Lock)	0.499	56	16.198	0.287	0.083	0.992	0.499	199.48	667	0.081	0.963	0.009
25005 (Leven @ Leven Bridge)	0.583	48	43.54	0.241	0.269	1.318	0.583	194.15	726	0.107	0.994	0.014
54041 (Tern @ Eaton Upon Tern)	0.656	42	12.444	0.187	0.109	1.203	0.656	193.49	719	0.119	0.954	0.015
33005 (Bedford Ouse @ Thornborough Mill)	0.688	28	21.8	0.178	-0.112	2.253	0.688	387.67	655	0.111	0.983	0.014
36005 (Brett @ Hadleigh)	0.698	56	11	0.284	0.095	0.581	0.698	155.85	580	0.076	0.994	0.009
43005 (Avon @ Amesbury)	0.698	29	10.264	0.322	0.255	1.119	0.698	326.47	744	0.071	1	0.013
14001 (Eden @ Kemback)	0.711	39	40.417	0.176	0.032	1.513	0.711	308.74	800	0.104	0.992	0.011
53008 (Avon @ Great Somerford)	0.712	55	37.132	0.264	0.214	0.227	0.712	305.17	804	0.093	0.988	0.009
Total		583										
Weighted means				0.279	0.167							

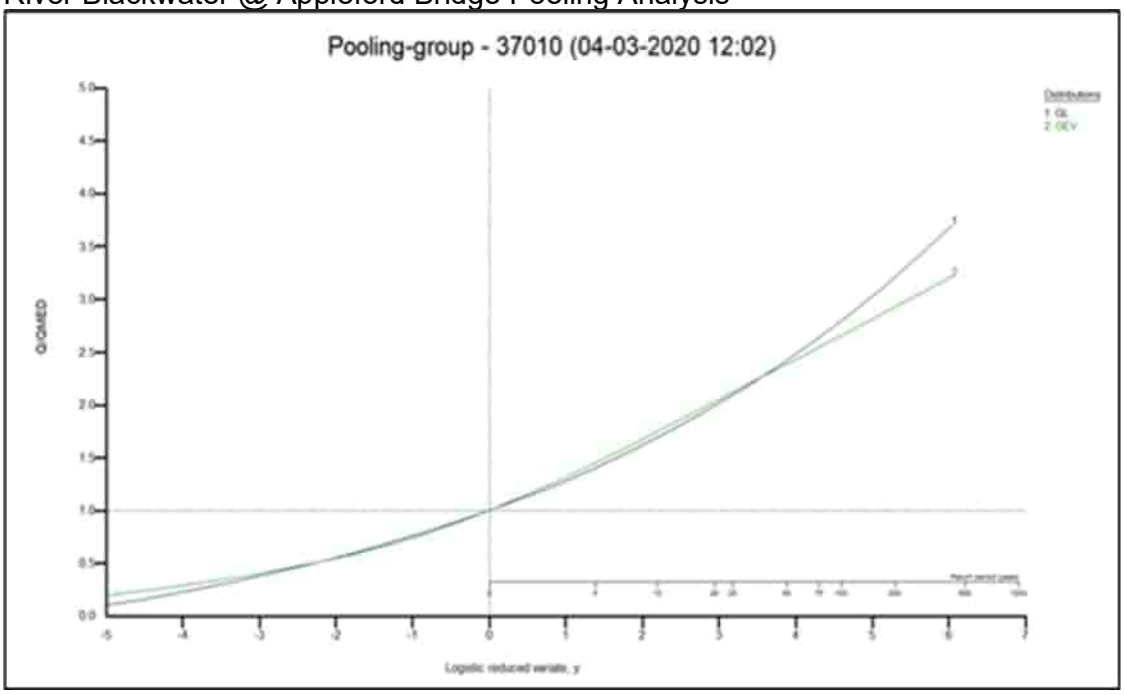
Final Pooling Group Details at River Blackwater @ Appleford

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37010 (Blackwater @ Appleford Bridge)	0	56	13.15	0.285	0.213	1.046	0	249.12	572	0.098	0.991	0.026
37005 (Colne @ Lexden)	0.287	58	12.4	0.268	0.097	0.17	0.287	235.88	566	0.076	0.97	0.022
36008 (Stour @ Westmill)	0.368	58	17.395	0.28	0.233	0.577	0.368	222.82	589	0.068	0.994	0.023
22006 (Blyth @ Hartford Bridge)	0.441	58	52.591	0.322	0.283	1.176	0.441	273.67	696	0.115	0.99	0.009
25005 (Leven @ Leven Bridge)	0.583	48	43.54	0.241	0.269	1.09	0.583	194.15	726	0.107	0.994	0.014
54041 (Tern @ Eaton Upon Tern)	0.656	42	12.444	0.187	0.109	1.01	0.656	193.49	719	0.119	0.954	0.015
33005 (Bedford Ouse @ Thornborough Mill)	0.688	28	21.8	0.178	-0.112	2.087	0.688	387.67	655	0.111	0.983	0.014
36005 (Brett @ Hadleigh)	0.698	56	11	0.284	0.095	0.681	0.698	155.85	580	0.076	0.994	0.009
14001 (Eden @ Kemback)	0.711	39	40.417	0.176	0.032	1.428	0.711	308.74	800	0.104	0.992	0.011
53008 (Avon @ Great Somerford)	0.712	55	37.132	0.264	0.214	0.348	0.712	305.17	804	0.093	0.988	0.009
20001 (Tyne @ East Linton)	0.738	22	51.91	0.284	-0.008	1.388	0.738	307.26	713	0.05	0.987	0.007
Total		520										
Weighted means				0.277	0.159							

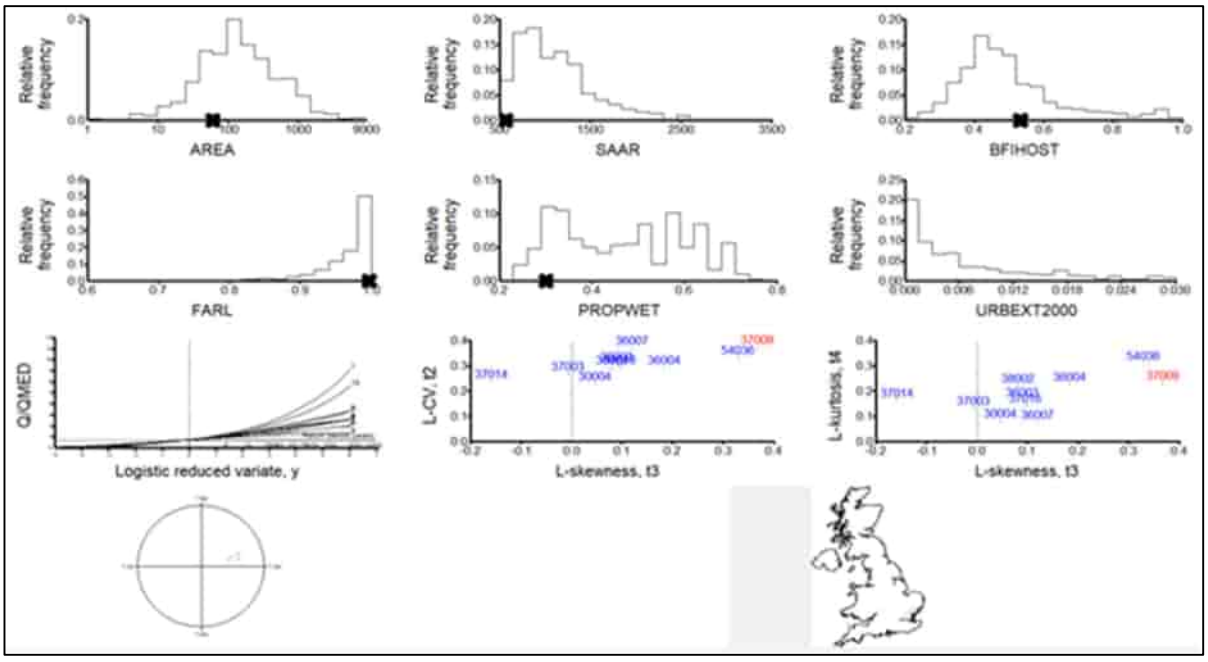
A.5 River Blackwater @ Appleford Bridge ESS Analysis



River Blackwater @ Appleford Bridge Pooling Analysis



A.6 Pooling Group Details – Graphs – River Brain @ Guithavon



A.7 Pooling Group Details – Table- River Brain @ Guithavon

Initial Pooling Group Details: (show default selection) at River Brain @ Guithavon

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
36007 (Belchamp Brook @ Bardfield Bridge)	0.085	53	4.63	0.371	0.119	2.12	58.16	560	0.079	0.996	0.004
36003 (Box @ Polstead)	0.141	57	3.91	0.305	0.089	0.172	56.72	566	0.093	0.993	0.012
37016 (Pant @ Copford Hall)	0.19	54	7.24	0.293	0.096	0.053	63.8	588	0.069	0.997	0.009
36004 (Chad Brook @ Long Melford)	0.334	51	5.186	0.294	0.182	1.811	50.33	589	0.065	1	0.006
26013 (Driffield Trout Stream @ Driffield)	0.411	8	2.78	0.29	0.218	2.603	53.33	690	0.093	0.997	0.006
30004 (Lymn @ Partney Mill)	0.45	56	6.983	0.229	0.046	0.953	60.09	686	0.06	0.979	0.006
26003 (Foston Beck @ Foston Mill)	0.46	57	1.76	0.248	-0.009	1.029	59.59	698	0.106	0.987	0.004
37003 (Ter @ Crabbs Bridge)	0.5	53	5.43	0.269	-0.007	0.739	77.76	570	0.115	0.994	0.012
33032 (Heacham @ Heacham)	0.52	50	0.442	0.304	0.124	0.084	56.16	688	0.116	0.983	0.006
38002 (Ash @ Mardock)	0.553	77	6.76	0.292	0.08	0.434	78	619	0.049	1	0.014
Total		516									
Weighted means		401		0.295	0.09						

*River Brain @ Guithavon not included in initial pooling group until ESS analysis selected as moderately urbanised catchment.

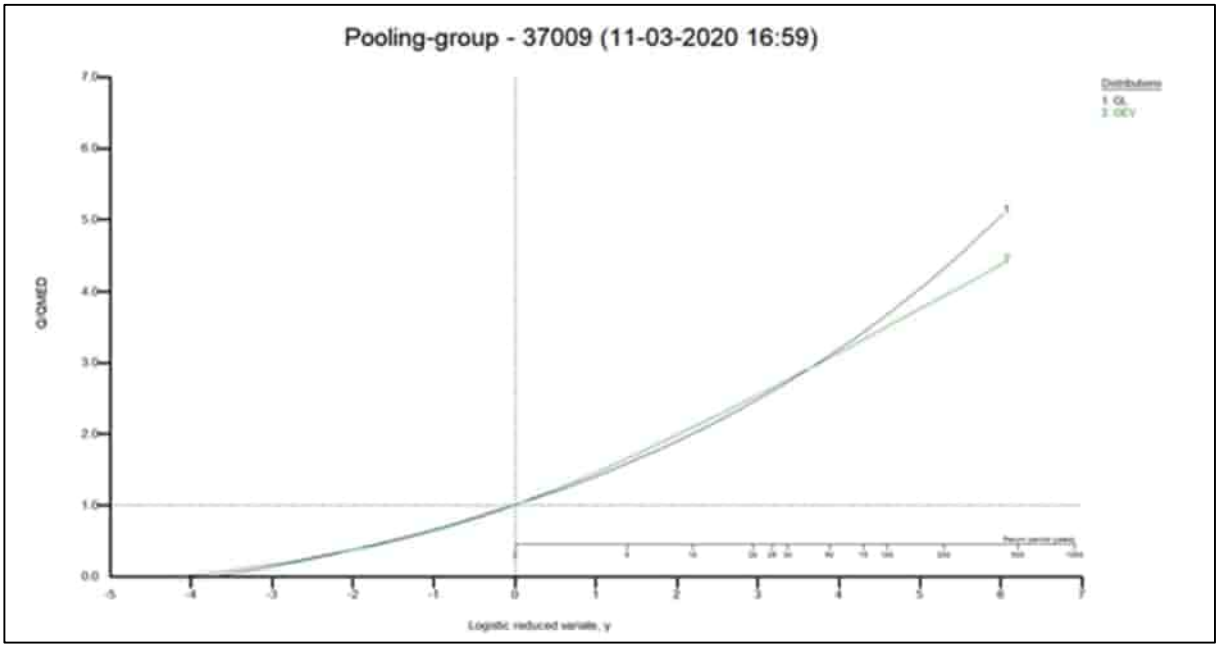
Final Pooling Group Details at River Brain @ Guithavon – ESS Analysis

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
37009 (Brain @ Guithavon Valley)	0	56	3.895	0.389	0.354	1.082	60.27	572	0.084	0.997	0.074
36007 (Belchamp Brook @ Bardfield Bridge)	0.085	53	4.63	0.371	0.119	2.26	58.16	560	0.079	0.996	0.004
36003 (Box @ Polstead)	0.141	57	3.91	0.305	0.089	0.059	56.72	566	0.093	0.993	0.012
37016 (Pant @ Copford Hall)	0.19	54	7.24	0.293	0.096	0.114	63.8	588	0.069	0.997	0.009
36004 (Chad Brook @ Long Melford)	0.334	51	5.186	0.294	0.182	0.298	50.33	589	0.065	1	0.006
30004 (Lymn @ Partney Mill)	0.45	56	6.983	0.229	0.046	2.435	60.09	686	0.06	0.979	0.006
37003 (Ter @ Crabbs Bridge)	0.5	53	5.43	0.269	-0.007	0.244	77.76	570	0.115	0.994	0.012
38002 (Ash @ Mardock)	0.553	77	6.76	0.292	0.08	0.413	78	619	0.049	1	0.014
37014 (Roding @ High Ongar)	0.663	54	10.928	0.239	-0.159	1.747	92.65	597	0.107	0.986	0.008
54036 (Isbourne @ Hinton on the Green)	0.737	46	13.578	0.331	0.329	1.348	92.83	701	0.07	0.99	0.012
Total		557									
Weighted means				0.368	0.17						

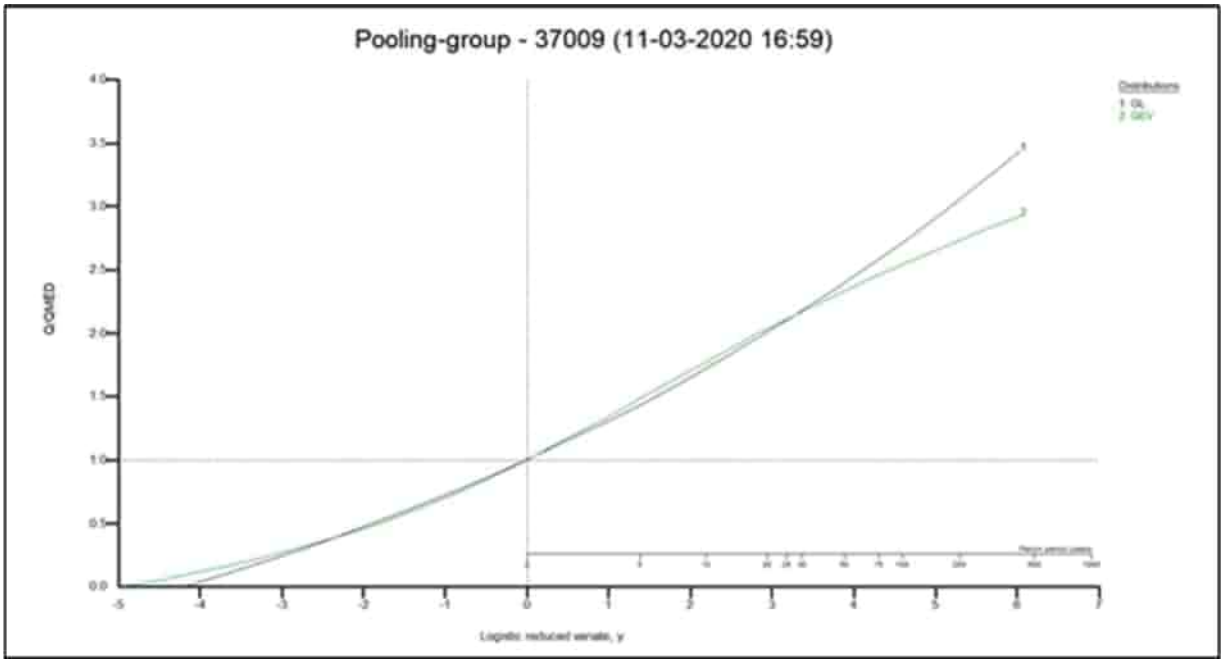
Final Pooling Group Details at River Brain @ Guithavon – Pooling Analysis

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
36007 (Belchamp Brook @ Bardfield Bridge)	0.085	53	4.63	0.371	0.119	2.261	58.16	560	0.079	0.996	0.004
36003 (Box @ Polstead)	0.141	57	3.91	0.305	0.089	0.056	56.72	566	0.093	0.993	0.012
37016 (Pant @ Copford Hall)	0.19	54	7.24	0.293	0.096	0.107	63.8	588	0.069	0.997	0.009
36004 (Chad Brook @ Long Melford)	0.334	51	5.186	0.294	0.182	0.344	50.33	589	0.065	1	0.006
30004 (Lymn @ Partney Mill)	0.45	56	6.983	0.229	0.046	2.16	60.09	686	0.06	0.979	0.006
37003 (Ter @ Crabbs Bridge)	0.5	53	5.43	0.269	-0.007	0.196	77.76	570	0.115	0.994	0.012
38002 (Ash @ Mardock)	0.553	77	6.76	0.292	0.08	0.346	78	619	0.049	1	0.014
37014 (Roding @ High Ongar)	0.663	54	10.928	0.239	-0.159	1.857	92.65	597	0.107	0.986	0.008
54036 (Isbourne @ Hinton on the Green)	0.737	46	13.578	0.331	0.329	1.673	92.83	701	0.07	0.99	0.012
Total		501									
Weighted means				0.297	0.09						
Weighted means				0.279	0.051						

A.8 River Brain @ Guithavon ESS Analysis

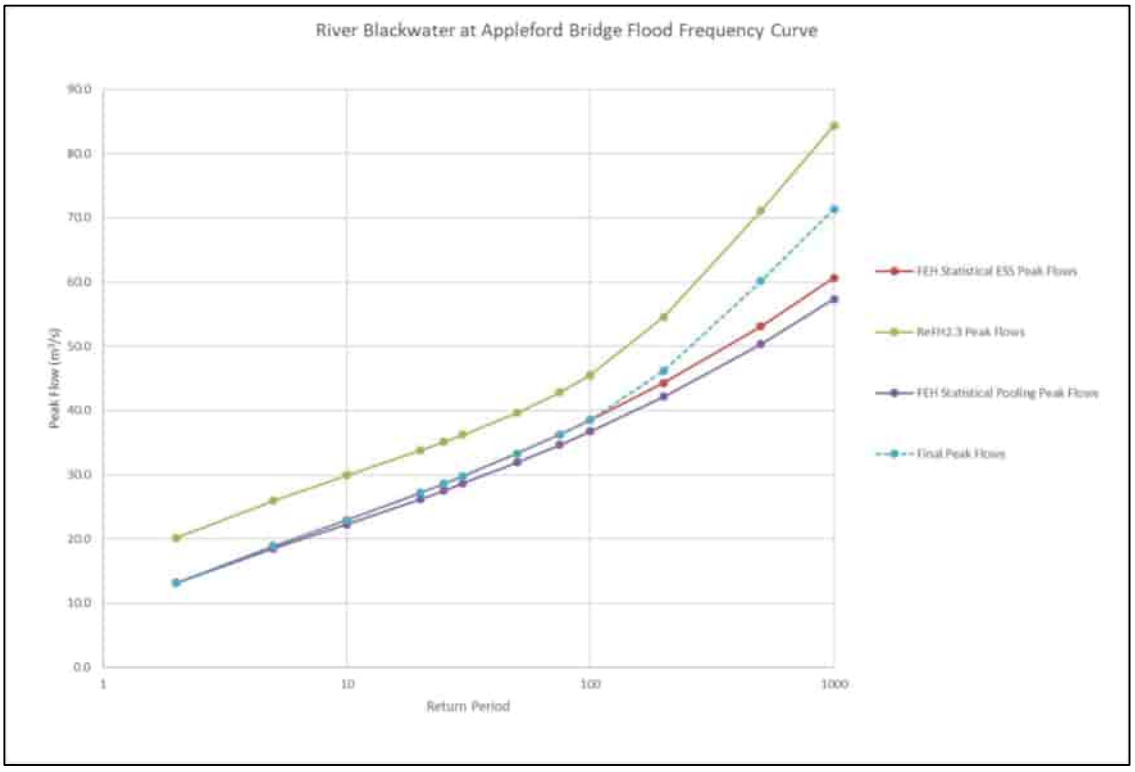


River Brain @ Guithavon Pooling Analysis

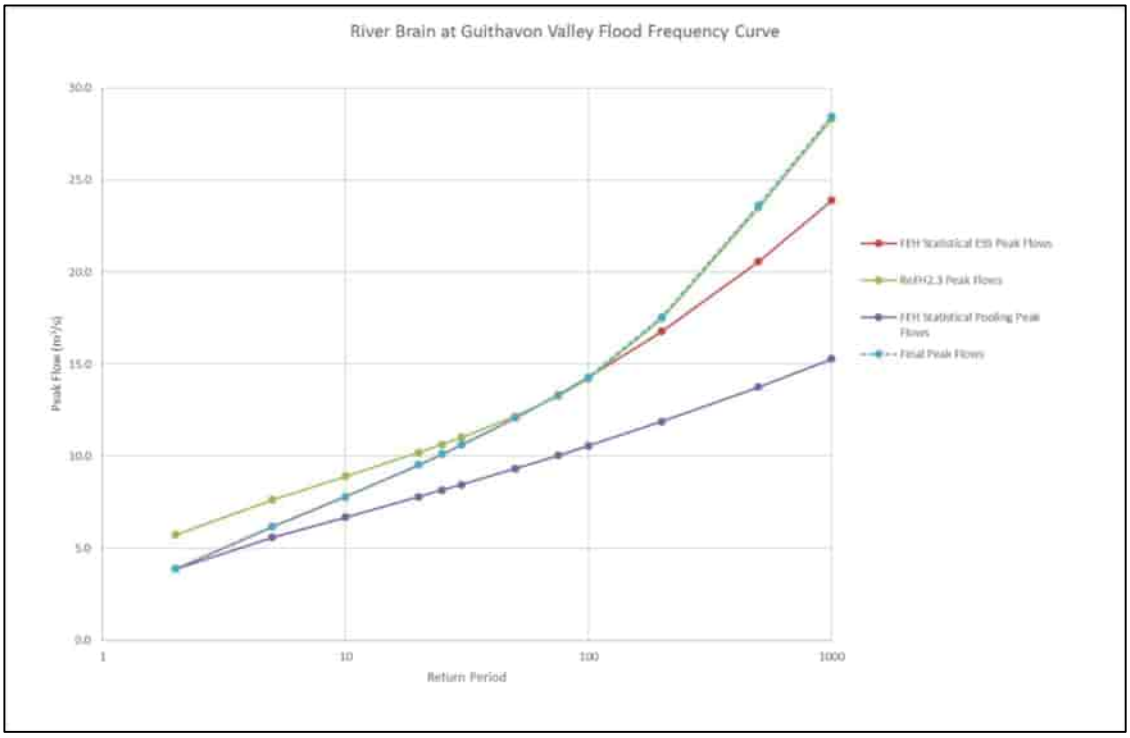


A.9 Flood Frequency Curve Comparison

River Blackwater at Appleford Bridge

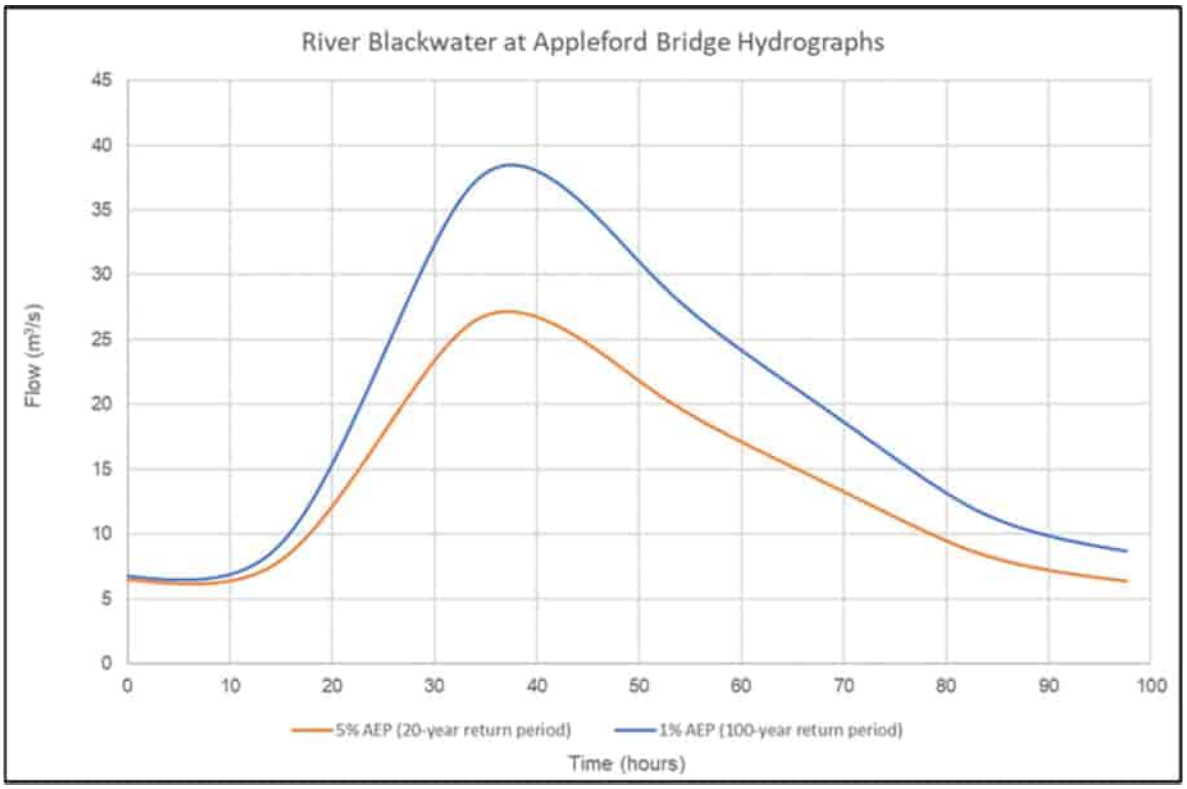


River Brain at Guithavon Valley



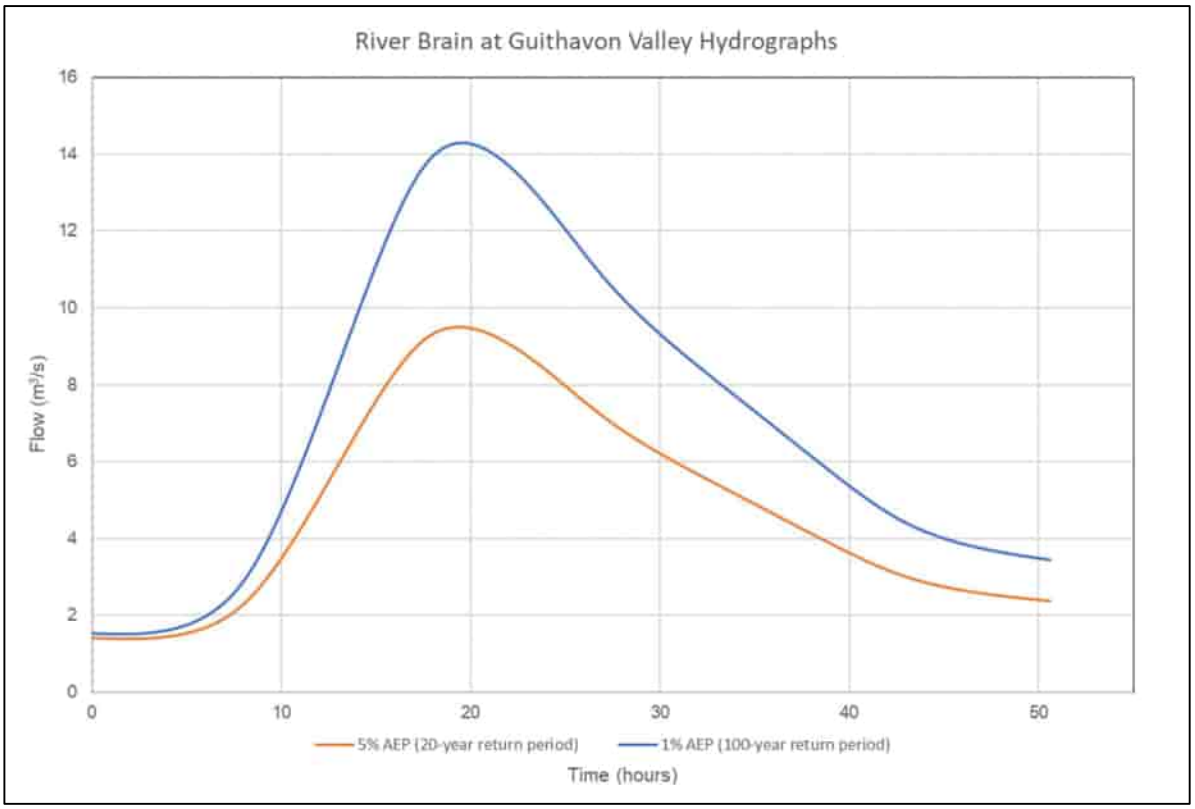
A.10 5% and 1% AEP (20 and 100-year return period) Hydrographs

River Blackwater @ Appleford Bridge 5% and 1% AEP (20 and 100-year return period) Hydrographs



ReFH1 hydrograph scalled to the final peak flow estimate. Duration 29.7 hours

River Brain @ Guithavon Valley 5% and 1% AEP (20 and 100-year return period) Hydrographs



ReFH1 hydrograph scalled to the final peak flow estimate. Duration 15.9 hours.

A.11 ReFH2.3 Hydrograph Shape Sensitivity Analysis

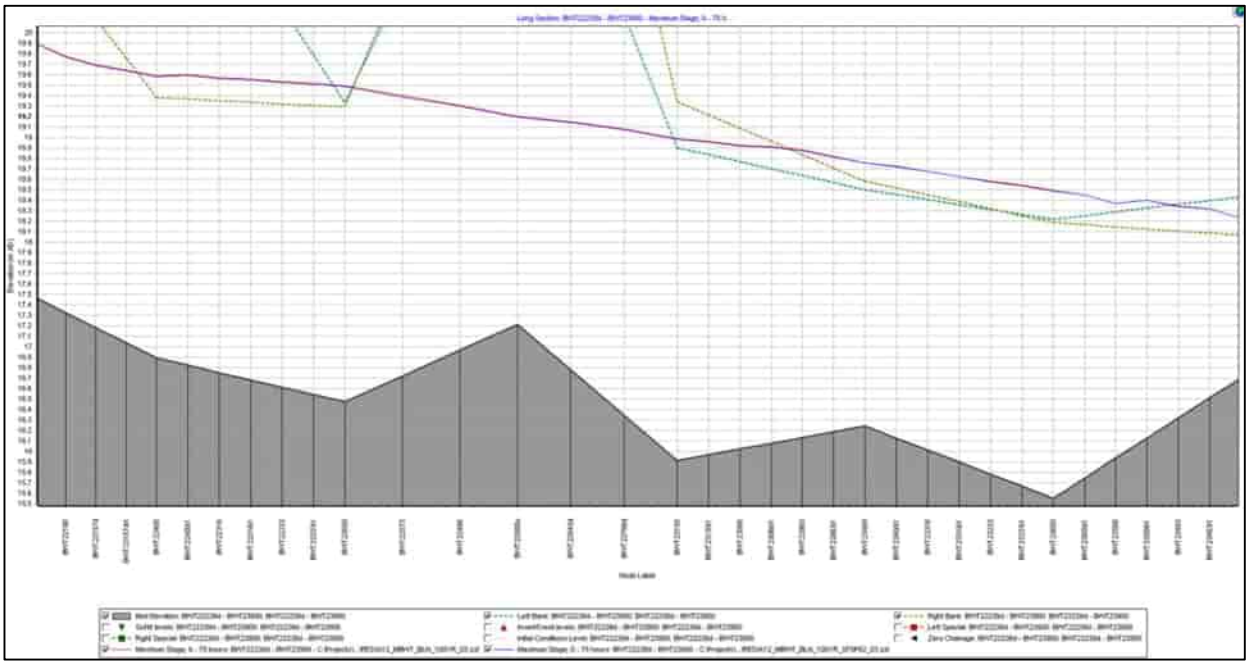
A sensitivity analysis was undertaken for the Middle Blackwater model for the 1% AEP (100-year return period) event to assess whether there is an impact on flood extent and depths by adoption the ReFH2.3 hydrograph shape opposed to the originally adopted ReFH1 hydrograph shape. For this purpose, the Middle Blackwater model ReFH1 IEDs for the baseline scenario were replaced with ReFH2.3 flows for the 1% AEP (100-year return period) event. The modelled flow using ReFH2.3 inflows were reconciled with the final peak flow estimate at the River Blackwater at Appleford Bridge (37010) gauging station, similar to that was undertaken for the ReFH1 modelled flows. The results from this sensitivity analysis indicated that the ReFH1 hydrograph shape produced more conservative flood extents (red outline in figure below) within the vicinity of the A12 crossing of the River Blackwater than the corresponding ReFH2.3 inflows scaled to the target peak flow estimate (blue shading in figure).

Figure showing 1% AEP (100-year return period) flood extents at the A12 crossing of the River Blackwater for the Baseline Scenario (ReFH1 – red - scaling factor 0.7, ReFH2.3 – blue - scaling factor 0.82)



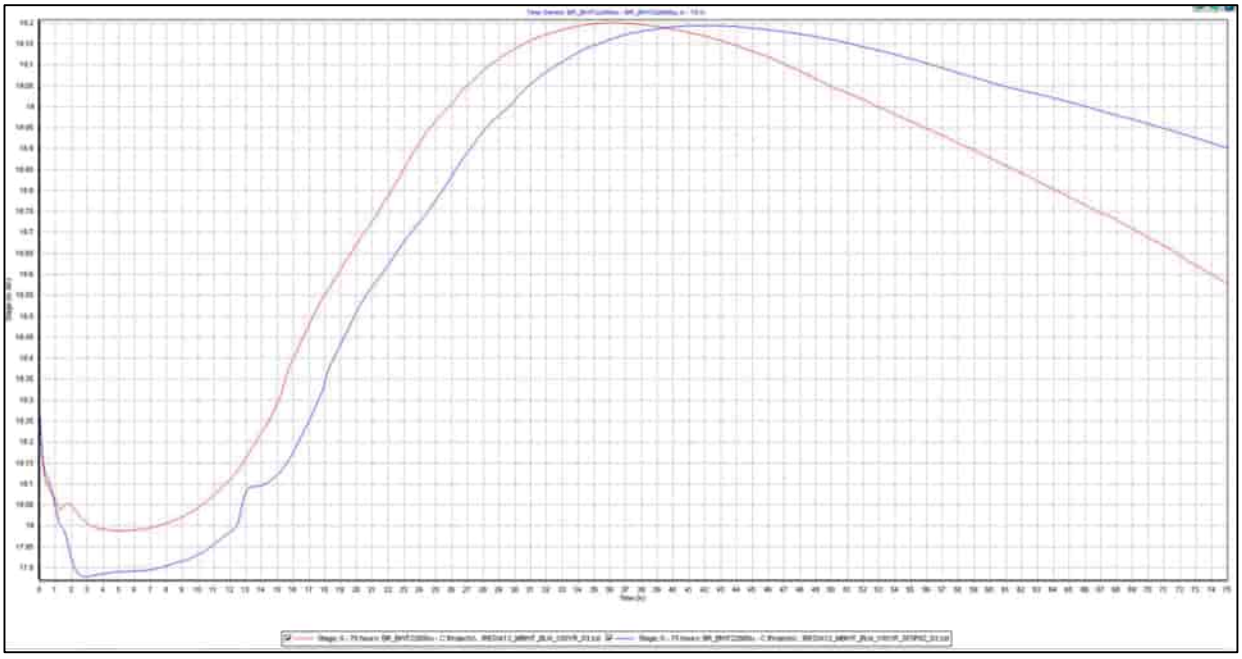
The water level in the River Blackwater in the vicinity of the A12 crossing was also assessed and estimated to be marginally higher when the ReFH1 hydrograph shape (red) is adopted opposed to the ReFH2.3 hydrograph shape (blue).

Longprofile in vicinity of A12 for baseline scenario (ReFH1 – red – scaling factor 0.7, ReFH2.3 – blue – scaling factor 0.82)



When comparing the hydrograph shapes produced using ReFH1 and ReFH2.3 inflows scaled to the final peak flow estimate at the A12 crossing (baseline scenario) the ReFH1 shape is relatively comparable with the ReFH2.3 shape, with peak from one method (ReFH1) occuring earlier than the other (ReFH2.3).

Stage at A12 for baseline scenario (ReFH1 – red – 0.7 scaling factor, ReFH2.3 – blue – 0.82 scaling factor)



In conclusion it has been assessed that the application of the ReFH1 hydrograph shape for model inflows results in slightly more conservative flood extents and depths within the vicinity of the proposed scheme for the tested baseline scenario and therefore the ReFH1 hydrograph shape has been accepted as appropriate to take forward for the Middle Blackwater and Lower Blackwater and Brain hydraulic models for the purposes of assessing any potential impacts of the A12 road improvements scheme.

A.12 Environment Agency Response to Consultation

Environment Agency

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WOKINGHAM
RG41 5TU

Our ref: AE/2020/125129/01-L01
Your ref: B229H130-TRN-00590

Date: 11 June 2020

Dear Mr. Symons

A12 CHELMSFORD TO A120 WIDENING SCHEME - JUNCTION 23 (KELVEDON SOUTH) TO JUNCTION 25 (MARKS TEY INTERCHANGE).
RIVER BLACKWATER & RIVER BRAIN FLOOD MODEL HYDROLOGY REVIEW

Thank you for sending over the technical note regarding the review of the hydrology for the River Blackwater Middle hydraulic model and the River Blackwater Lower (including River Brain) hydraulic model (Doc No. HE551497-JAC-EWE-3_S0-TN-LE-0001).

Following a review of that note we would confirm that we are largely in agreement with the approaches proposed. Carrying out a revised ESS and Pooling group approaches seems appropriate given the gauging uncertainty, despite the updates to the Peak Flow data. It would be useful to have rating reviews carried out on the two gauges, but we accept that it's outside of the resources of the project. It would be useful perhaps to add a note on this in the reporting, so as to highlight this requirement in future studies.

The only further point we would like to raise relates to Step 3: Inflows to the hydraulic flood model. For the new IED files being generated, will there be any consideration of applying the ReFH2 methods? The National River Flow Archive states that the Appleford gauge (37010) is groundwater influenced, ReFH2 should offer a major improvement on these types of catchments – so ideally a comparison of the hydrographs should be carried out.

Yours sincerely

[redacted]

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A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT DOMSEY BROOK MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

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0 Executive summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at Domsey Brook. The modelling and hydrology have been carried out in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1D (1 Dimensional) type using Flood Modeller software (Jacobs 2022) and was built from scratch. The A12 crosses Domsey Brook twice.

0.1.2 Three events were simulated: the 5%, 1% and 1% AEP with 38% climate change. Flow reconciliation was carried out to match the design hydrology. No data was available for calibration, but the model was verified using Environment Agency (EA) flood maps. The model was then used to simulate baseline and with-scheme scenarios for the above three design events. The with-scheme scenario involved a new culvert at the upstream crossing. At the downstream crossing, an extension to the A12 culvert and a change in the alignment of the channel local to the highway are required. Sensitivity analysis to roughness, inflows and the downstream boundaries was carried out to further validate the results.

0.1.3 The analysis shows that the proposed scheme has no significant impact on flooding along Domsey Brook and therefore no mitigation is proposed.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed scheme includes widening the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) is required for the proposed scheme to meet relevant local and national planning legislation and inform the design and planning process. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic models with associated catchment hydrology. Water levels for the baseline and with scheme scenarios both upstream and downstream of the proposed scheme and the associated flood extents were determined for a range of storm flood events at each watercourse crossing.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA are:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook (this report)
- Roman River
- Five minor watercourses were also modelled for the proposed scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for Domsey Brook to assess the baseline and “with scheme” scenarios. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 For the upstream A12 crossing of Domsey Brook, the highway will be upgraded to three lanes per carriageway with the addition of a new offline road alignment to the south of the existing carriageway. Where the A12 crosses the downstream end of Domsey Brook, the road will be widened to the east (upstream) and the channel will be realigned before entering the culvert.

1.1.6 This report should be read in conjunction with the Flood Risk Assessment report (Jacobs 2022).

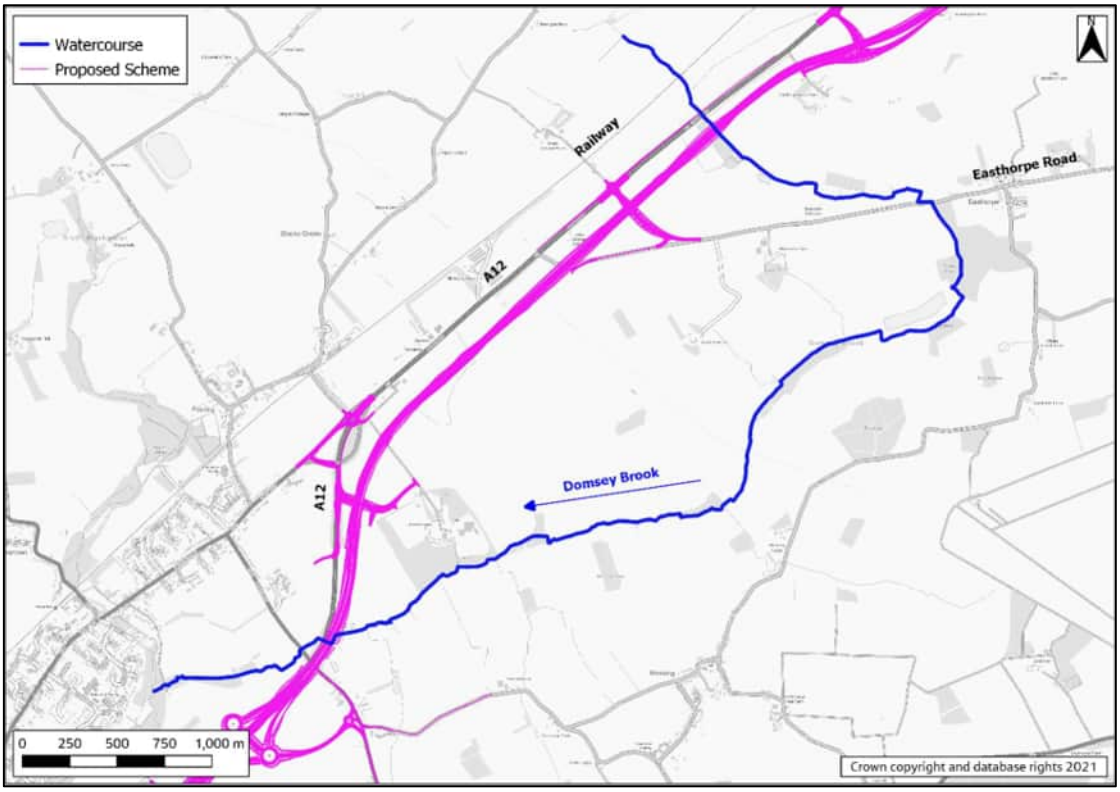
1.2 Methodology

1.2.1 The model was built using a one-dimensional (1D) schematisation, using the Flood Modeller (version 4.6) river modelling software. This approach was taken because Domsey Brook flows within a well-defined floodplain that remains consistent across the model domain, and where the predominant unidirectional flow path from upstream to downstream is maintained for all flows that are considered.

1.3 Study area

1.3.1 Domsey Brook flows to the west of Easthorpe village in Essex, from north to south-west and crosses the A12 twice to meet the River Blackwater as shown in Plate 1.1. The model domain includes a 580m reach of Domsey Brook upstream of the east A12 crossing, a 6.5km reach between the two A12 crossings, and a 1.0km reach downstream of the west A12 crossing down to the confluence with the River Blackwater.

Plate 1.1 Domsey Brook study area



2 Input data

2.1.1 The data used to construct the hydraulic model of Domsey Brook is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR	1m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) data. It was used to extend cross sections into the floodplain.	Environment Agency (EA) 2012
OS maps	Mastermap data	Highways England
Channel survey	In-channel cross sections and hydraulic structures	Jacobs March 2020
Hydrological analysis	Hydrological analysis carried out for Domsey Brook	Jacobs April 2021

3 Hydrology

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of the Domsey Brook for the 5% and 1% Annual Exceedance Probability (AEP) events (equivalent to the 20-year and 100-year return periods). Allowance for climate change (CC) on the 1% AEP event is also required. Full details of the flow derivation methods and background hydrology are available in Annex A.

3.2 Hydrological considerations

3.2.1 The Domsey Brook catchment is a lowland catchment in the South East of England covering an area of 26.6km^2 to the downstream model extent at its confluence with the River Blackwater. It is classified as essentially rural with small urban areas around Marks Tey, Easthorpe, Messing and Inworth. There is a small offline reservoir south east of Easthorpe but the attenuation of river flows is expected to be minimal across the catchment.

3.2.2 The catchment lies on a bedrock of sedimentary clay, silt, sand and gravel and has soils with slightly impeded drainage. The baseflow index is 0.365 with a relatively high standard percentage runoff of 46.4%. These do not indicate a highly permeable catchment. The catchment receives a low average annual rainfall of 565mm.

3.3 Peak flow derivations

3.3.1 The Domsey Brook catchment is just above the threshold to be classified as small (<25km^2). For such catchments where uncertainties may be large it is considered sensible to compare peak flows derived from two independent methods. Therefore, both the Flood Estimation Handbook (FEH) Statistical and ReFH2.3 methods were applied to estimate the design return period peak flows.

3.3.2 A total of seven inflow hydrographs were required as inputs to the hydraulic model, including one point inflow at the upstream model extent (DOM1), three lateral inflows from residual catchments (DOM3, DOM4 and DOM7) and three point inflows from three minor tributaries (DOM2, DOM5 and DOM6). The FEH statistical method is not recommended for deriving peak flow from the residual catchments, but for all sub-catchments considered in this study it was used to be able to compare peak flows from the two independent methods. An estimate of peak was also made at the downstream end of the model (target location) to provide comparisons of runoff from the entire catchment to enable reconciliation of modelled flow and the corresponding peak flow at this target location. The ReFH2.3 method resulted in higher design peak flows for the Domsey Brook than the FEH Statistical method at all locations.

3.3.3 The peak flow estimates at the four point inflow locations DOM1, DOM2, DOM5 and DOM6 as well as at the target location (CONF – which includes the runoff from the entire Domsey catchment) are presented in

Table 3.1. The location of the peak flow estimation points is shown in Plate 3.1, with the CONF at the same location as DOM7.

Table 3.1 Domsey Brook peak flows as a function of event rarity for corresponding critical storm. N.B. flows calculated with individual design storm duration for each catchment.

	Peak flows (m^3/s)									
AEP	**50%**	**20%**	**10%**	**5%**	**3.33%**	**2%**	**1.33%**	**1%**	**0.5%**	**0.1%**
Statistical										
CONF	3.1	4.7	5.6	6.6	7.1	7.9	8.5	9.0	10.2	17.7
DOM1	0.44	0.63	0.76	0.89	0.97	1.08	1.16	1.22	1.38	2.41
DOM2	0.76	1.10	1.32	1.54	1.68	1.86	2.01	2.12	2.39	4.2
DOM5	0.71	1.02	1.23	1.44	1.57	1.74	1.87	1.98	2.23	3.89
DOM6	0.35	0.51	0.61	0.71	0.77	0.86	0.92	0.97	1.10	1.92
ReFH2.3										
CONF	4.27	5.57	6.45	7.32	7.87	8.64	9.38	10.01	12.19	19.66
DOM1	0.61	0.81	0.95	1.08	1.17	1.29	1.41	1.50	1.83	2.97
DOM2	0.86	1.14	1.33	1.52	1.64	1.81	1.97	2.11	2.56	4.18
DOM5	1.16	1.55	1.81	2.07	2.23	2.46	2.67	2.85	3.47	5.77
DOM6	0.71	0.97	1.14	1.32	1.43	1.57	1.71	1.82	2.20	3.65

The AEP flows at CONF are for the full catchment runoff (target peak flow) used to reconcile the modelled flow at the model downstream extent (target location)

3.4 Inflow hydrograph shape

3.4.1 Given the Domsey Brook catchment is ungauged, it was assessed as appropriate to derive hydrograph shapes for input into the hydraulic model using ReFH2.3. The ReFH2.3 hydrograph shapes were derived for all sub-catchments and are all based on an 18-hour winter storm duration, which is equivalent to the design storm duration at the downstream modelling extent (CONF), that is, target location.

3.4.2 The critical storm duration for the upstream A12 crossing location was determined to be 15-hours. However following model testing it was seen that the use of the shorter storm duration made no significant difference to peak water levels or flows at the upstream location and the single storm duration of 18-hours was therefore utilised for both the upstream and downstream crossing locations.

3.5 Climate change

3.5.1 The climate change allowance for the River Blackwater Operational Catchment Area has been taken from the unpublished UKCP18 peak river

flow climate change allowances. The upper climate change allowance for the River Blackwater Operational River Basin is 38% in accordance with guidance (Environment Agency 2021) for climate change uplift for this area. For further details of the climate change uplift, see the Flood Risk Assessment (Environment Agency 2021).

3.6 Hydraulic model inflows

3.6.1 The Domsey Brook hydraulic model requires design hydrographs at seven locations as described below:

- DOM1 – point inflow at the upstream extent of the Domsey Brook;
- DOM2 –point inflow for the sub-catchment between the upstream model extent and the existing A12;
- DOM3 – lateral inflow from the residual catchment between the existing A12 and the initially proposed A12 crossing east of Easthorpe;
- DOM4 – lateral inflow from the residual catchment between the initially proposed A12 crossing to the river inflows at Messing and Inworth;
- DOM5 – point inflow from a combined sub-catchment of two watercourses at Feering (right bank) and Messing (left bank);
- DOM6 – point inflow from the left bank river sub-catchment at Inworth;
- DOM7 – lateral inflow from the residual catchment between DOM6 to the confluence with the Blackwater.

3.6.2 A map showing the Domsey Brook hydraulic model extent and inflow locations is provided in Plate 3.1.

3.6.3 The peak inflows before reconciliation are given in Table 3.2 and 1% AEP inflow hydrographs are presented in Plate 3.2.

Plate 3.1 Inflow locations

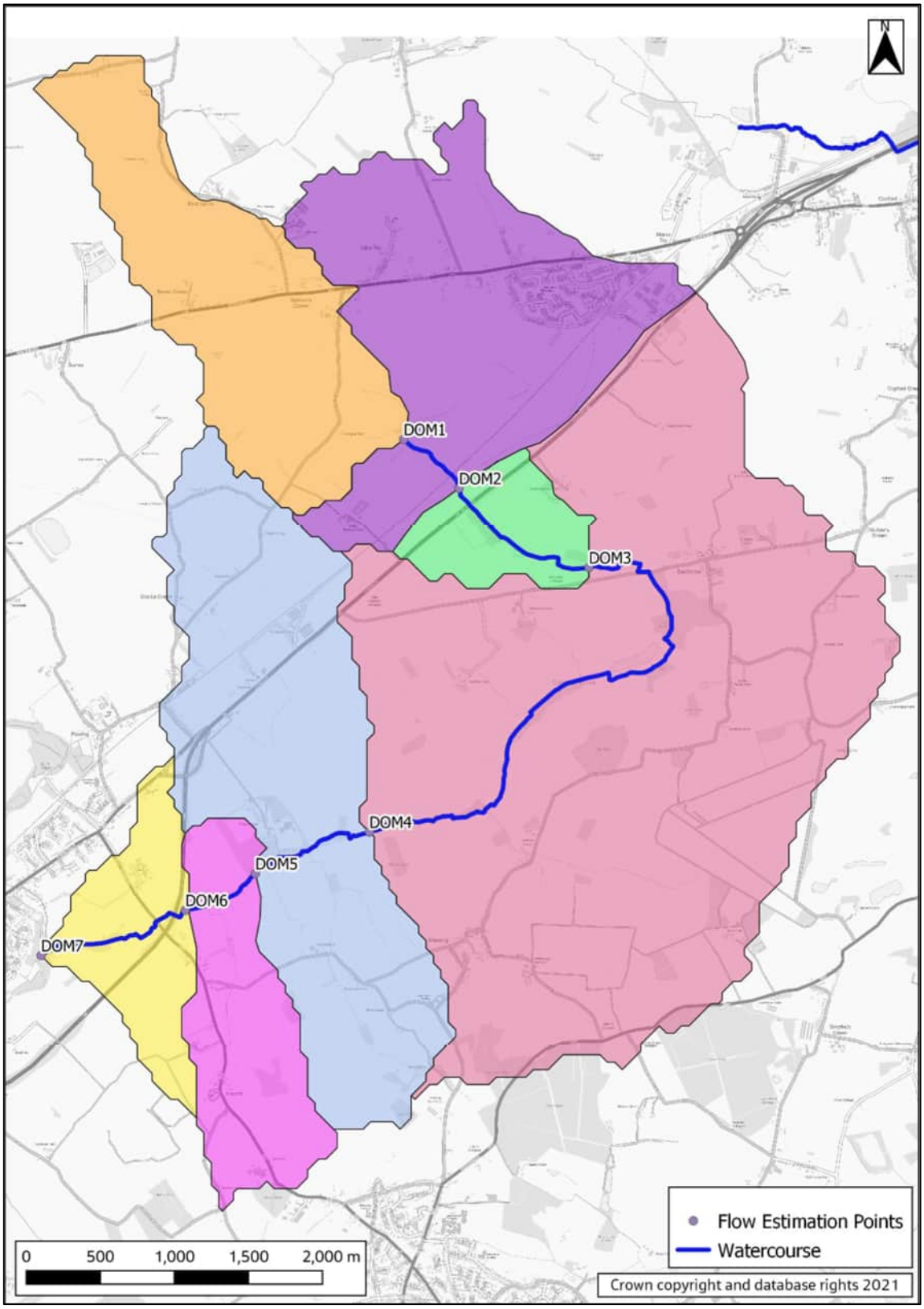
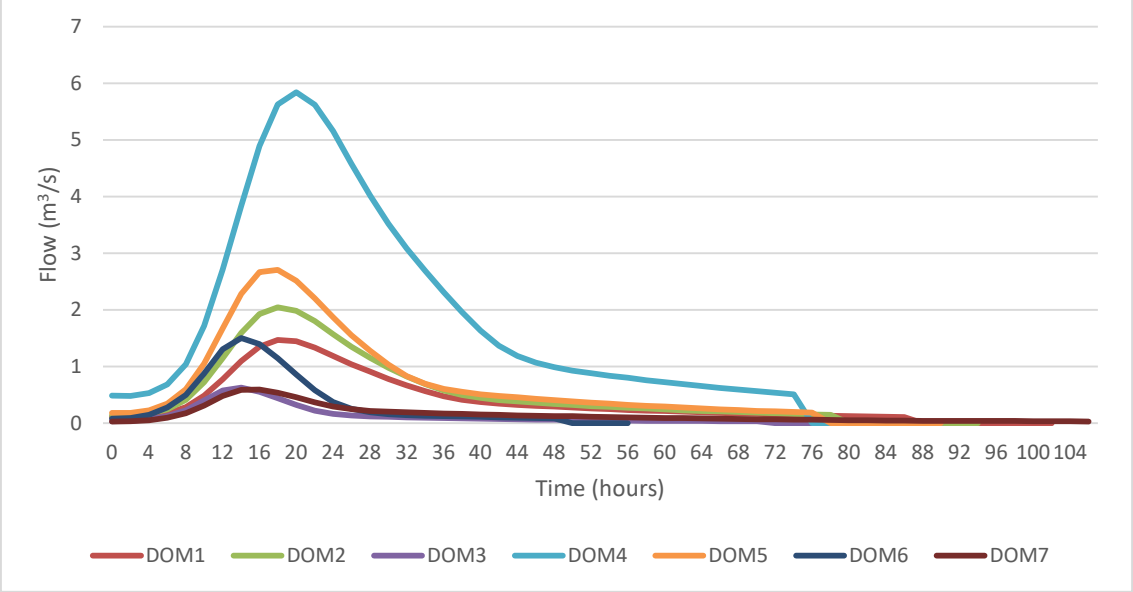


Table 3.2 Inflow peaks as a function of event rarity for the 18 hour storm duration

	Peak flows (m^3/s)										
AEP	**50%**	**20%**	**10%**	**5%**	**3.33%**	**2%**	**1.33%**	**1%**	**1% + 38% CC**	**0.5%**	**0.1%**
DOM1	0.60	0.80	0.93	1.06	1.14	1.26	1.37	1.47	2.02	1.79	2.88
DOM2	0.85	1.12	1.30	1.48	1.60	1.76	1.92	2.05	2.83	2.51	4.05
DOM3	0.25	0.33	0.39	0.45	0.48	0.54	0.58	0.63	0.86	0.77	1.25
DOM4	2.45	3.21	3.73	4.25	4.57	5.03	5.47	5.84	8.06	7.12	11.55
DOM5	1.11	1.48	1.72	1.96	2.12	2.33	2.53	2.71	3.74	3.32	5.49
DOM6	0.61	0.82	0.95	1.09	1.17	1.29	1.41	1.50	2.07	1.85	3.05
DOM7	0.24	0.32	0.37	0.43	0.46	0.51	0.55	0.59	0.82	0.73	1.20

Plate 3.2 1% AEP inflow hydrographs



3.7 Flow reconciliation

3.7.1 A flow reconciliation exercise was carried out by comparing the peak modelled flow at the target location, which is at the downstream end of the model (cross-section “DB_0000i”) against the whole catchment target flows (CONF). The model inflows were scaled (uniformly) down by 15% to achieve the best fit. Final flows used for the baseline simulations are given in Table 3.2, whilst Table 3.3 presents the reconciliation results obtained with those.

Table 3.3 Domsey Brook final flows following flow reconciliation

AEP	5%	1%	1% +38%CC
Labels	**(m^3/s)**		
DOM1	0.90	1.25	1.73
DOM2	1.26	1.74	2.41
DOM3	0.38	0.54	0.74
DOM4	3.61	4.96	6.85
DOM5	1.67	2.30	3.18
DOM6	0.93	1.28	1.76
DOM7	0.37	0.50	0.69

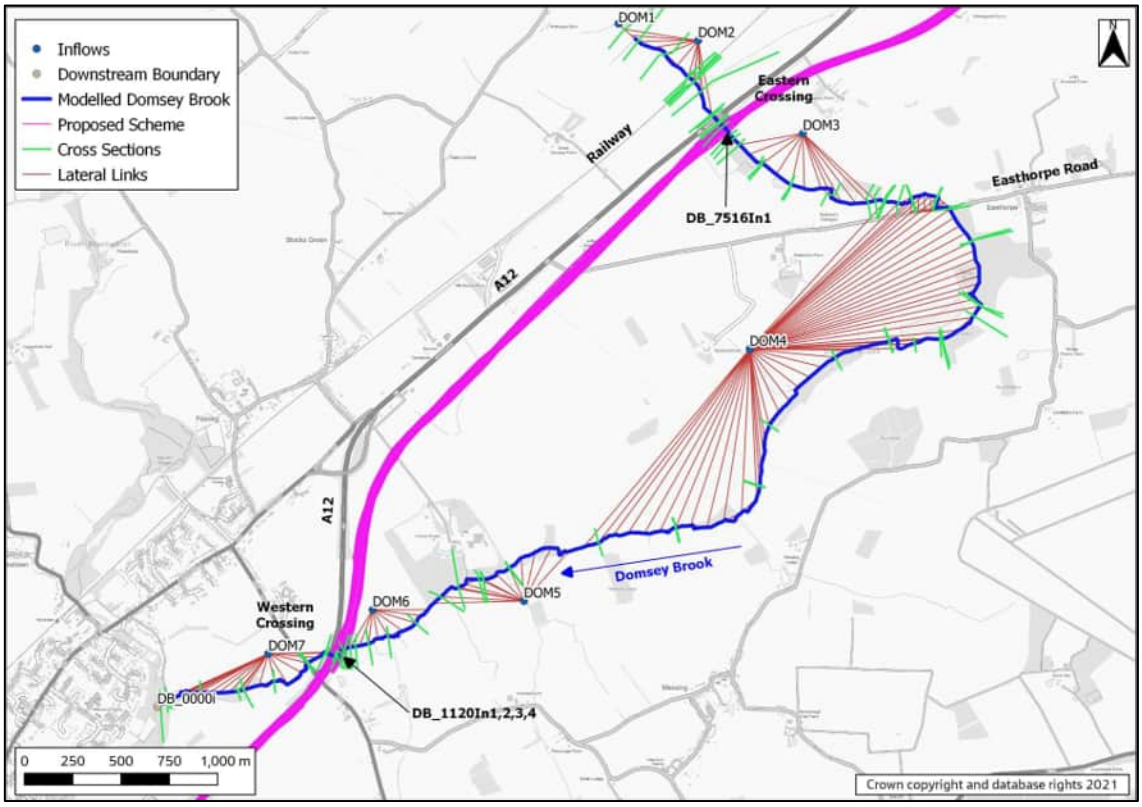
Table 3.4 Domsey Brook final peak flows

AEP	5%	1%	1% +38%CC
	(m^3/s)		
Downstream end, Target Flows (CONF)	7.32	10.01	13.81
Downstream end, Modelled Flows (DB_0000i)	7.36	10.38	13.68
Difference (%)	0.55	3.70	-0.94

4 Baseline modelling

4.1.1 Surveyed cross section data has been used to inform the in-channel geometry of the modelled watercourse. The locations of the surveyed cross-sections are shown in Plate 4.1.

Plate 4.1 Domsey Brook baseline model schematisation



4.1.2 To aid model performance, interpolated cross sections were added between the surveyed cross sections. The spacing of the interpolated cross sections ranged from 10m to 100m.

4.1.3 New river unit sections have been added at key locations and their out-of-bank elevations were extracted from the 1m DTM. In-bank dimensions were copied from the nearest surveyed cross-section. The key locations comprise:

- At the model upstream extent (DB_8235) to ensure full representation of the backwater effect from the railway embankment.
- Between the railway and the existing east A12 Road crossing (DB_7769In1) to ensure appropriate representation of the left-hand side floodplain storage.

- Upstream of the west A12 Road crossing (DB_1961i) to ensure appropriate representation of the floodplain storage.
- Upstream of the proposed offline carriageway locations (DB_7516In1, DB_1120In1, DB_1120In2, DB_1120In3 and DB_1120In4) to allow good comparison of baseline and subsequent "with scheme" models.

4.1.4 There is an inline weir immediately downstream of the existing east A12 Road crossing (Plate 4.2) which was not surveyed. It is represented in the model by assuming the weir crest is at 32.46mAOD, that is, above the surveyed water level of 32.36mAOD by 0.1m.

Plate 4.2 Online weir



4.1.5 The floodplain has been represented in the model with extended cross-sections. This was done by extending selected cross-sections using the 1m LiDAR DTM.

4.1.6 Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using the photographs taken during the survey. Generally, gravels are seen in the bed of the channel, with long grass and heavy vegetation on the floodplain. The Manning's 'n' coefficients used in the

model are shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow 1959). In addition to this, at selected cross-sections, out of bank areas were assigned Manning's 'n' values between 0.07 and 0.1 to represent woodland coverage.

Table 4.1 Manning's 'n' coefficient

Node Labels	Watercourse	Bed Manning's 'n'	Bed Material	Floodplain Manning's 'n'	Floodplain Material
DB_8141 to DB_0000i	Domsey Brook	0.035	Clean, straight channel with Gravels and Weeds	0.050	Scattered brush, heavy weeds

4.1.7 Twenty-one hydraulic structures are included in the model. A structure schedule is presented in Annex B. Several simple footbridge structures have been ignored as adding them would not make any difference to the model results and could potentially lead to unnecessary instabilities.

4.1.8 The upstream and downstream boundary conditions applied to the hydraulic model are described in Table 4.2.

Table 4.2 Boundary conditions

Type of Boundary	Node Labels	Description
QTBDY	DOM1	ReFH2 boundary. Applied at the upstream end of Domsey Brook.
QTBDY	DOM2	ReFH2 boundary. Applied laterally between DB_8141 (first cross-section in the model) and DB_7560i (upstream of the existing east A12 Road crossing) using a reach distribution method.
QTBDY	DOM3	ReFH2 boundary. Applied laterally between DB_7356 (downstream of the proposed offline carriageway) and DB_6513 using a reach distribution method.
QTBDY	DOM4	ReFH2 boundary. Applied laterally between DB_6460 and DB_2685In1 using a reach distribution method.
QTBDY	DOM5	ReFH2 boundary.

Type of Boundary	Node Labels	Description
		Applied laterally between DB_2685In2 and DB_1715In1 using a reach distribution method.
QTBDY	DOM6	ReFH2 boundary. Applied laterally DB_1715In2 and DB_1120 (upstream of the west A12 Road crossing) using a reach distribution method.
QTBDY	DOM7	ReFH2 boundary. Applied laterally between DB_0956i (downstream of the west A12 Road crossing) and DB_0198In3 (downstream model extent) using a reach distribution method.
NCDBDY (Normal depth downstream boundary)	DB_0000i	Normal depth boundary condition applied to the downstream end of Domsey Brook. Sensitivity testing was undertaken (Section 6.4 Sensitivity Analysis) which demonstrates that this assumption is appropriate to make.

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 38% uplift is in accordance with guidance (Environment Agency 2021) for climate change uplifts for this area. For further details of the climate change uplift, see the Flood Risk Assessment (Environment Agency 2021).

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope (see Section 6.4 Sensitivity Analysis).

Table 5.1 Modelled events

Scenario	AEP Event		
	5%	1%	1%+38%CC
Baseline	X	X	X
With Scheme	X	X	X
Roughness Sensitivity (in-channel, floodplain, and structures)		X	
Hydrological Inflow Sensitivity		X	
Downstream Boundary Sensitivity		X	

6 Model proving

6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution, within a specified number of iterations, for each model time step. The convergence of the 1D model was checked and is within tolerance as shown in Plate 6.1 for the baseline scenario and Plate 6.2 for the with scheme scenario. This convergence plot is typical for all events simulated. There are three instances of poor convergence in flow during the simulation. These were seen to occur at model nodes DB_2145o, DB_1020 and DB_1120i2 (upstream of the west A12 crossing at the downstream end). The effect on the model results is noted to be very small and occurs outside of the hydrograph peak and is therefore deemed acceptable. The 1D model mass error has also been checked and is within the recommended tolerance of 1% for all simulations carried out.

Plate 6.1 1D Model convergence for baseline scenario – 1% AEP +38%CC event

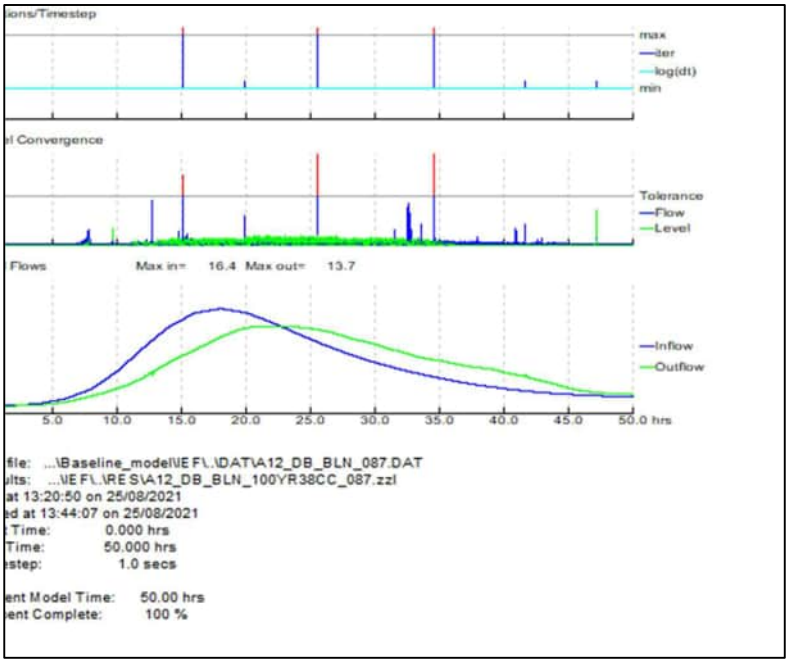
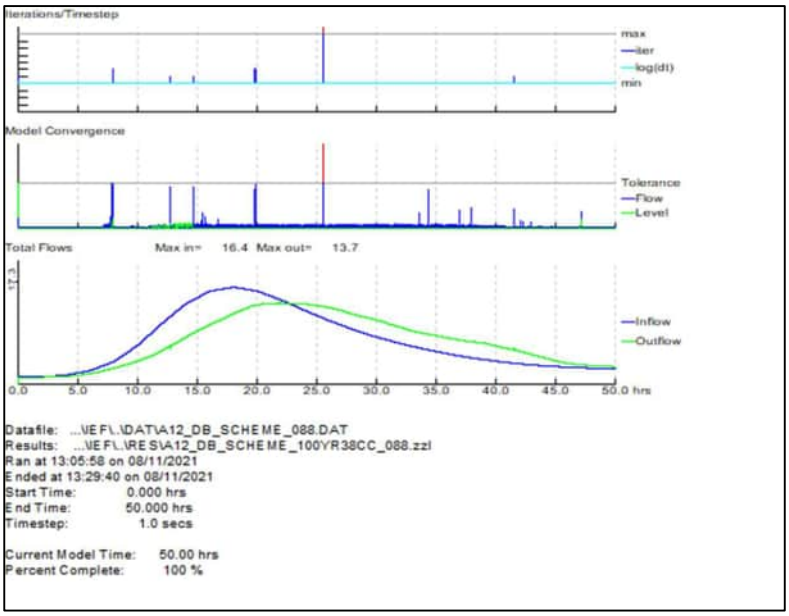


Plate 6.2 1D Model convergence for with scheme scenario – 1% AEP +38%CC event



6.3 Calibration and verification

Calibration

6.3.1 There are no gauges on Domsey Brook within the extent of the model. Therefore, it was not possible to calibrate the model. However, some data is available for a high-level verification of the model results.

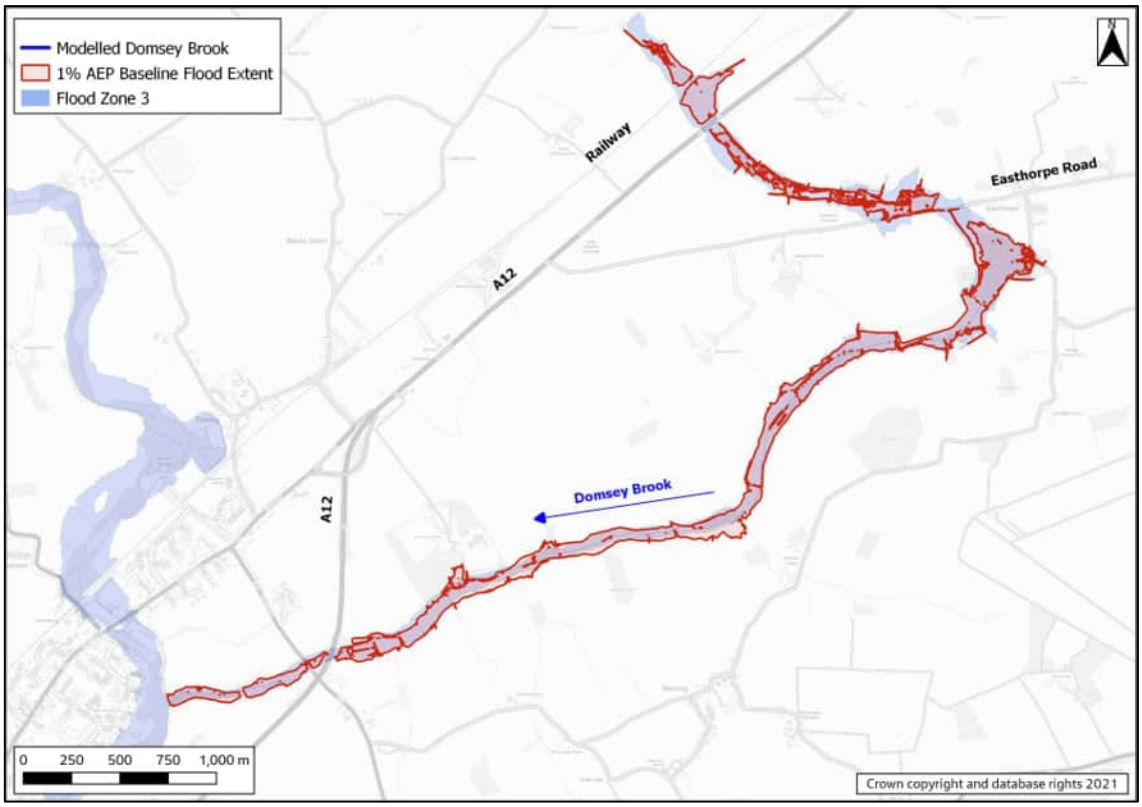
Verification using historic data

6.3.2 The Environment Agency published historic flood maps for UK watercourses show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. A review of this data showed that there is no historic flood data for Domsey Brook.

Verification using EA flood maps

6.3.3 Plate 6.3 shows the modelled 1% AEP existing situation flood extent and the Environment Agency published Flood Zone 3. The comparison shows that the Environment Agency flood map predicts a generally similar flood extent to the model results. There are some divergences along the watercourse length but this is expected as the EA map is generated based on the high-level J-Flow modelling system and the present model generates flood maps based on higher resolution and more up to date topographic data.

Plate 6.3 Modelled 1% AEP event flood extent vs. Environment Agency flood zone 3



Verification conclusion

6.3.4 In conclusion, there is limited data available for verification of the Domsey Brook model. The EA flood map shows a generally similar outline to the present work thereby validating the outputs.

6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness, hydrological inflows and downstream boundary conditions. These analyses were performed on the penultimate version of the baseline model. However, the changes made in the final version of the baseline model were small (the addition of a single cross-section to the baseline model to match the cross-section in the with-scheme). Therefore, the changes will not change the general outcomes of the sensitivity tests. All sensitivity tests were performed using the 1% AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') along with the structures such as bridges and culverts were changed by +20% and -20%. Table 6.1 shows the impact of changing the model roughness on predicted peak water levels. The results show that the water levels are moderately sensitive to changes in roughness.

Table 6.1 Roughness sensitivity results – 1 % AEP event

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	0.14	-0.09	0.04
-20% Roughness	-0.23	0.02	-0.05

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Table 6.2 shows the impact of changing the model inflows on predicted peak water levels. The results show that the water levels are moderately sensitive to changes in flow.

Table 6.2 Flow sensitivity results – 1 % AEP event

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.26	0.01	0.05
-20% Flow	-0.29	-0.02	-0.08

Downstream Boundary Condition sensitivity

6.4.4 The slope in the normal depth downstream boundary was adjusted by +20% and -20%. Table 6.3 shows
the response at the downstream end of the model (Flood Modeller node DB_0000i). The location at which there is no change in water level as a

result of changing the downstream boundary has been identified. Distances from this location, in relation to the downstream end of the model (tailwater distance) and in relation to the proposed scheme are also shown. The results of the downstream boundary sensitivity test show that the effect is remote from the scheme location.

Table 6.3 Downstream boundary sensitivity results – 1 % AEP event

Sensitivity	Water Level Difference at DB_0000i (m)	Tailwater distance (m)	Distance to proposed scheme (m)
+20% Downstream boundary slope	-0.016	49	962
-20% Downstream boundary slope	0.021	49	962

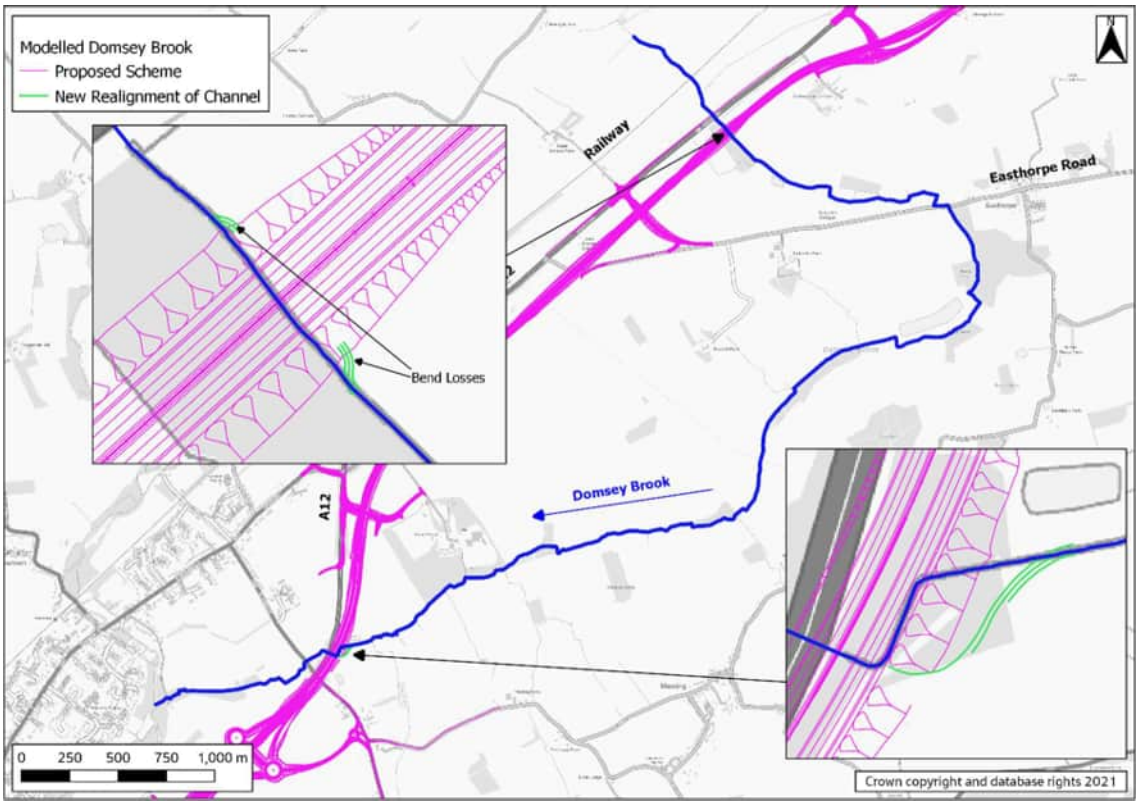
7 With scheme model build

7.1.1 The baseline model was updated in order to include representation of the proposed scheme. Changes to the model were as follows:

- For the upstream crossing (east crossing), a new box culvert was added where the proposed scheme will cross Domsey Brook. The dimensions of this culvert are 2.5m wide by 1.8m high and 68.5m long (chainage). The gradient of the culvert matches the existing channel bed which is slightly adverse. Bend losses were included to simulate the 22º bends at the inlet and outlet as well as wing walls. A Manning's 'n' of 0.015 was used for all walls and the soffit. Invert Manning's 'n' was kept the same as the riverbed. All design flows are required to pass through the new culvert structure, so no other model elements were required. The scheme arrangements are shown in Plate 7.1.

- For the downstream crossing (west crossing), the culvert under the A12 was extended from 42.75m to 66.5m using the same culvert dimensions as the existing culvert. Also, the channel was re-aligned as shown in Plate 7.1 and the first section was trimmed to fit in between the embankments.

Plate 7.1 Realignment of channel at the crossings



8 Model results

8.1 Baseline results

8.1.1 In-channel maximum water levels have been inspected at key locations in relation to the proposed scheme. Table 8.1 shows in-channel maximum water levels for the 1% AEP +38%CC event. The in-channel water levels at key locations for all modelled events are shown in Annex C.

8.1.2 Plate 8.1 shows the maximum flood extent for all modelled events for the baseline scenario. It shows the following:

8.1.3 Significant floodplain storage occurs upstream of the east A12 crossing alongside Domsey Brook, where flows are locally constricted by the A12 for all AEP events.

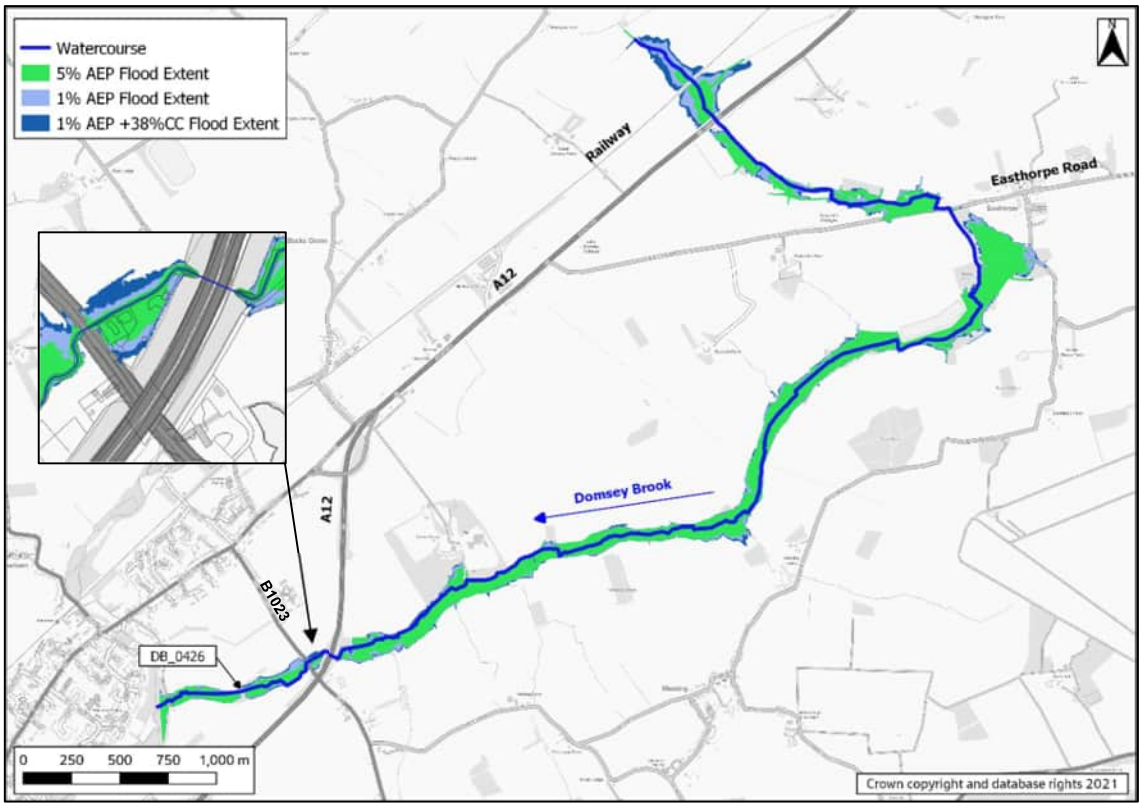
8.1.4 Between the east A12 crossing and Easthorpe Road, Domsey Brook comes out-of-bank at various locations for all modelled events, including some ponding directly upstream of Easthorpe Road.

8.1.5 Downstream of Easthorpe Road, Domsey Brook exhibits consistent out of channel flow for all modelled AEP events. However, the results show that no properties are flooded within the model domain for any of the events simulated, except a number of properties on the B1023, just downstream of the west A12 crossing (see Plate 8.1 inset), this site is inundated for all the flood events simulated.

8.1.6 It is evident when inspecting the full suite of flood event simulations, that in addition to the limited channel capacity, there are several culvert structures at river crossings which surcharge significantly in the modelled flood events.

- This is the case for the railway culvert and the east A12 Road twin culvert for events of 1% AEP and above.
- The structure between the west A12 crossing and the confluence with the River Blackwater also surcharges during the 1% AEP +38%CC event. Most notably the embankment at model node DB_0426 (which is thought to be a disused railway crossing) causes a head loss of more than 0.75m and a tailwater effect reaching into the floodplain upstream of the west A12 highway crossing.
- There is no flooding to the A12 highway at either crossing location.

Plate 8.1 Flood extents for all events for the baseline scenario. Insert shows property flooding.



8.2 With scheme model results

8.2.1 Table 8.1 compares the in-channel maximum water levels at key locations in the model between the baseline and with scheme scenarios for the largest event. The levels for all modelled events can be found in Annex C. It shows that changes to the water level due to the implementation of the scheme are negligible throughout the model except immediately upstream of the new east crossing where a 70mm increase in water level is noted for the 1%AEP +38%CC event. This increase is restricted to the 33m length of channel between the existing A12 and the new scheme crossing. Water levels remain in bank for this section of channel.

8.2.2 Plate 8.2 shows the water level difference for the 1% AEP +38%CC event. The impacts on maximum flood level difference between baseline and with scheme are categorised into levels of impact due to the scheme and it is shown that there is no significant impact to flooding due to the proposed scheme except the betterment immediately upstream from the west crossing. The modelled flood extent is not otherwise affected. The impact of the scheme for all other flood events can be found in Annex D.

8.2.3 The new culvert at the scheme east crossing has capacity for all modelled events. In order to comply with DMRB (Design Manual for Roads and

Bridges) guidance, the design height of this culvert incorporates 0.6m freeboard to the soffit above the 1% AEP +38%CC event maximum water level.

8.2.4 Pass forward flows were checked at the downstream end of the model. The flow hydrographs at the downstream end of the model show up to 0.016 m^3/s increase in the 1% AEP +38%CC event. This is no more than 0.15% increase in peak flow. This increase has been investigated and is not due to the implementation of the scheme and is therefore attributed to hydraulic modelling accuracy. It should be noted that within the model domain there is no significant increase in flood depth as mentioned previously. Immediately downstream from the modelled Domsey Brook reach, is the River Blackwater which is a considerably higher order of watercourse and the modelled flow increase is even more insignificant for this watercourse.

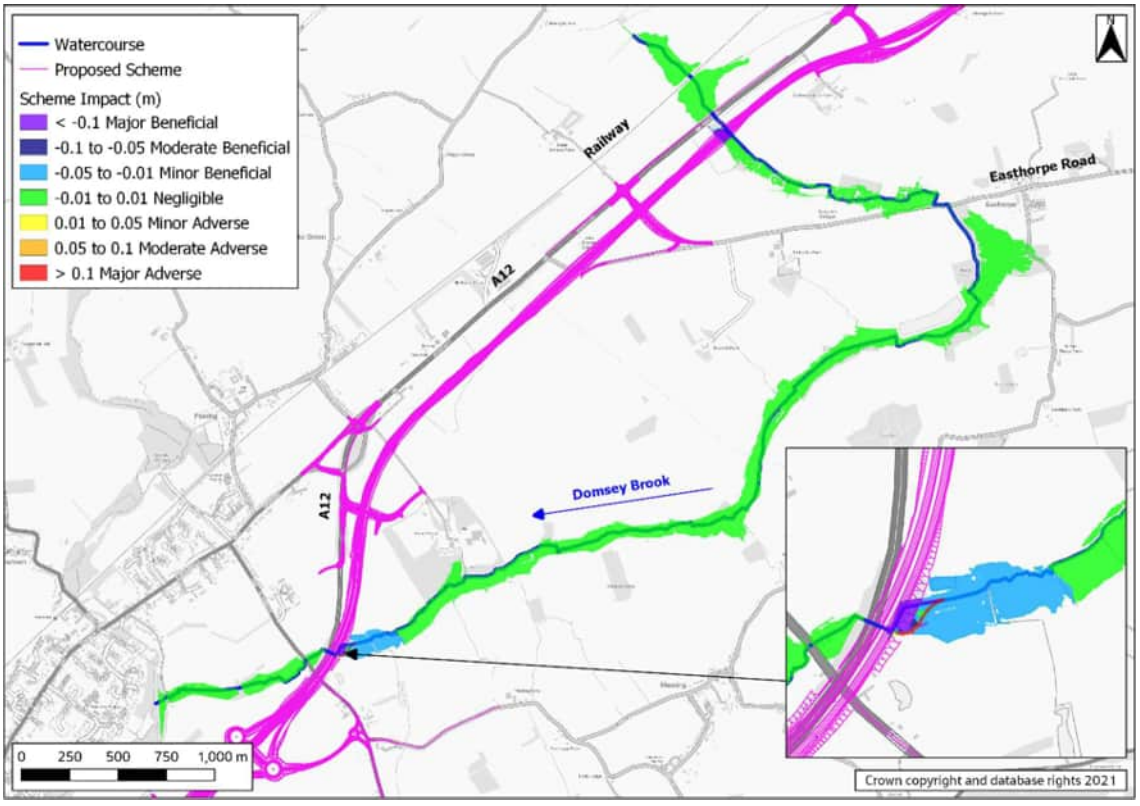
8.2.5 At the downstream crossing (west), the water levels remain the same for the existing and the newly extended culvert. The structure has more than 2m freeboard to the soffit above the 1% AEP + 38%CC event maximum water level. Upstream from the culvert, the channel realignment has improved the hydraulic performance of the channel and this results in decreased flood risk in the flood plain immediately upstream from the west crossing. The betterment is evident in all events simulated.

Table 8.1 In-channel maximum water levels at key locations for the 1 % AEP +38% CC event

Node Labels	Description	Baseline	Scheme	Difference
DB_7809i	Upstream of the railway culvert	33.77	33.77	0.00
DB_7769i	Downstream of the railway culvert	33.69	33.69	0.00
DB_7560	Upstream of the east A12 Road culvert	33.67	33.67	0.00
DB_7516	Downstream of the east A12 Road culvert	33.05	33.05	0.00
DB_7516i2	Upstream of the proposed offline carriageway culvert	32.82	32.89	0.07
DB_7356	Downstream of the proposed offline carriageway culvert	32.59	32.59	0.00

Node Labels	Description	Baseline	Scheme	Difference
DB_1120In1	62m upstream of the west A12 Road culvert	24.00	24.00	0.00
DB_1144	Upstream of the west A12 Road culvert	24.10	24.05	-0.05
DB_0956i	Downstream of the west A12 Road culvert	23.59	23.59	0.00
DB_0426	(Suspected) Disused railway located 540m downstream of the proposed carriageway culvert	22.75	22.75	0.00

Plate 8.2 Water level difference (with-scheme minus baseline) for the 1% AEP +38%CC event



9 Model assumptions and limitations

9.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological, surveyed, and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

9.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed scheme and are therefore, appropriate for the flood risk assessment.

9.2 Hydrology

9.2.1 The catchment areas from the FEH web service were considered inaccurate in some places and have therefore been more accurately determined using OS mapping.

9.2.2 The River Ter at Crabbs Bridge has been selected as a donor catchment in the statistical method with the assumption that it is hydrologically similar to the Domsey Brook and will respond to rainfall in the same manner.

9.2.3 The 68% and 95% confidence limits for the QMED calculation, using the statistical method, at the confluence are 2.3 – 5.0m^3/s and 1.6 – 7.0m^3/s respectively.

9.3 Hydraulic modelling

9.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for Domsey Brook are as follows:

- Surveyed cross sections have been extended using LiDAR data.
- Channel roughness and the floodplain roughness has been assigned across the model using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Some structures have been simplified to aid their representation and achieve stability. This is the case for two small culverts for which an orifice unit has been used. The disused railway crossing has been represented with a bridge unit rather than a full culvert arrangement.
- Hydraulic coefficients for structures have been applied using available guidance within the Flood Modeller software. The dimensions for structures have been based on detailed survey measurements.
- No calibration was carried out as the Domsey Brook catchment is ungauged.

10 Conclusion

10.1.1 This report has detailed the modelling carried out to assess the baseline and with scheme flood risk for Domsey Brook with respect to the location of the proposed A12 highway scheme.

10.1.2 The results of the baseline modelling have shown that there is significant flooding in the immediate floodplain for all events simulated. There are a small number of properties predicted to be flooded. However, the A12 highway remains free from flooding up to the 1% AEP +38% CC event.

10.1.3 Model results demonstrates that the proposed scheme has no impact on the existing flood risk in the vicinity of the A12 crossings.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

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Chow, Ven Te (1959). Open-Channel Hydraulics. McGraw-Hill.

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Environment Agency (2017b). Flood Estimation Guidelines. Technical Guidance 197_08.

Environment Agency (2017). Spatial Data Catalogue. [online] Available at: http://environment.data.gov.uk/ds/catalogue/#/catalogue [Accessed October 2017].

Jacobs (2022). Environmental Statement - Appendix 6.6 - Flood Risk Assessment. Ref: TR10060/APP/6.6.

Annex A **Domsey Brook Flood Estimation Calculation Record**

HE551497-JAC-EWE-SCHW-RP-LE-0080.docx

Annex B **Structure Schedule**

Table B.1 Model Structure Schedule

Structure Name	Node Labels	XY Coordinates	Modelling Approach
Culvert under Railway, 06.065	DB_7809	589550.0, 222042.8	Represented as symmetrical culvert without any spill unit in parallel. Inverts and dimensions of the culvert (2m wide x 1.8m high) are estimated from survey photographs. Survey for the structure not carried out due to access restriction. CW roughness of 0.004 m used to represent the brick culvert. Spill was ignored as railway embankment is never overtopped. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all parameters.
East A12 Road Culvert, 06.063	DB_7560	589668.0, 221846.2	Represented as twin circular culvert of 1m diameter without any spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill was ignored as road embankment is never overtopped. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Weir, Assumed	DB_7516wu	589686.7, 221824.5	Represented as spill unit. Weir crest determined to be 0.1m above the surveyed water level. No survey for the structure was carried out due to access restriction. Spill coefficient of 1.4 is used to represent the rough concrete crest.
Culvert under Road, 06.060	DB_7207	589906.0, 221617.0	Represented as twin circular culvert of 0.75m diameter with spill unit in parallel. Data from detailed survey.

			Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.0 is used to represent the track. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Culvert under Road, 06.058	DB_7014	590057.1, 221507.0	Represented as twin circular culvert of 0.7m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.0 is used to represent the country road crossing. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Road Bridge, 06.051	DB_6469	590542.5, 221421.1	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill coefficient of 1.0 is used to represent the track. FM default coefficients applied for all other parameters.
Culvert under Road, 06.048	DB_6308	590682.6, 221435.0	Represented as twin circular culvert of 0.775m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.0 to represent the country road. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.

Culvert under Road, 06.046	DB_6167	590816.1, 221441.7	Represented as circular culvert of 0.85m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.0 is used to represent the country road crossing. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Culvert under Easthorpe Road, 06.044	DB_6026	590881.0, 221384.6	Represented as twin circular culvert of 0.8m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.2 to represent the road crossing. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Culvert under Road, 06.042	DB_5817	590991.9, 221211.7	Represented as twin circular culvert of 0.775m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.0 to represent the track cover. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Culvert under Road, 06.040	DB_5429	591041.6, 220879.7	Represented as twin circular culvert of 0.77m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material.

			Spill coefficient of 1.0 is used to represent the country road crossing. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Weir, 06.037	DB_5152s	590841.24, 220707.51	Represented as spill unit. Data from detailed survey. Spill coefficient of 1 is used to represent the concrete.
Culvert under Road, 06.034	DB_4824	590568.0, 220695.0	Represented as twin circular culvert of 0.775m diameter with spill unit in parallel. Data from detailed survey. Manning's n = 0.015 is used for mid-concrete material. Spill coefficient of 1.0 is used to represent the country road. Inlet and outlet loss coefficients are 0.5 and 1, respectively. FM default coefficients applied for all other parameters.
Road Bridge, 06.030	DB_3699	589897.0, 219946.3	Represented as an arch bridge unit with spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill coefficient of 1.0 to represent the concrete slab. FM default coefficients applied for all other parameters.
Road Bridge, 06.028	DB_3116	589462.1, 219728.6	Represented as an arch bridge unit with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill coefficient of 1.0 to represent the concrete slab. FM default coefficients applied for all other parameters.

Road Bridge, 06.025	DB_2145	588599.9, 219522.3	Represented as a rectangular orifice with a spill unit in parallel. Data, including bore area of 3.371m^2, is taken from the detailed survey. Spill coefficient of 1.0 to represent the concrete road. FM default coefficients applied for all other parameters.
Road Bridge, 06.023	DB_1985	588463.4, 219463.1	Represented as an arch bridge with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill coefficient of 1.0 to represent the country road. FM default coefficients applied for all other parameters.
West A12 Road Culvert, 06.012	DB_1020	587677.2, 219071.9	Represented as an asymmetrical culvert without any spill unit in parallel. Data from detailed survey. Manning's n = 0.015 for sides, and 0.035 for the bed material. Spill was ignored as road embankment is never overtopped. Inlet and outlet loss coefficients are 0.2 and 0.3, respectively. FM default coefficients applied for all other parameters.
B1023 Road Bridge, 06.010	DB_0850	587555.4, 219030.1	Represented as an arch bridge with twin opening with a spill unit in parallel. Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill coefficient of 1.0 to represent Inworth Road. FM default coefficients applied for all other parameters.
Culvert under Manmade Embankment, 06.006	DB_0426	587210.9, 218888.6	Represented as an arch bridge with twin opening with a spill unit in parallel.

			Lower and upper transition distance of 0.1m is applied. Data from detailed survey. Spill coefficient of 1.0 to represent the crest. FM default coefficients applied for all other parameters.
Bridge on World's End Lane, 06.002	DB_0004	586832.1, 218839.9	Represented as a rectangular orifice with a spill unit in parallel. Data, including bore area of 6.39m^2 is taken from the detailed survey. Spill coefficient of 1.2 to represent World's End Lane. FM default coefficients applied for all other parameters.

Annex C **Maximum Water Level Tables and Long Sections**

Table C.1 Baseline and With Scheme Modelled Water Levels

Model Node	Description	Water Level (mAOD)								
		5% AEP			1% AEP			1% AEP +38%CC		
		Baseline	Scheme	Difference	Baseline	Scheme	Difference	Baseline	Scheme	Difference
DB_7809i	Upstream of the railway culvert	33.25	33.25	0.00	33.49	33.49	0.00	33.77	33.77	0.00
DB_7769i	Downstream of the railway culvert	33.21	33.21	0.00	33.43	33.43	0.00	33.69	33.69	0.00
DB_7560	Upstream of east A12 Road culvert	33.17	33.17	0.00	33.41	33.41	0.00	33.67	33.67	0.00
DB_7516	Downstream of the east A12 Road culvert	32.94	32.94	0.00	33.00	33.00	0.00	33.05	33.05	0.00
DB_7516i2	Upstream of the proposed carriageway culvert	32.72	32.71	-0.01	32.78	32.80	0.02	32.82	32.89	0.07
DB_7356	Downstream of the proposed offline carriageway culvert	32.49	32.49	0.00	32.55	32.55	0.00	32.59	32.59	0.00
DB_1120In1	87m upstream of the west A12 Road culvert	23.60	23.56	-0.04	23.82	23.79	-0.03	24.00	24.00	0.00

Model Node	Description	Water Level (mAOD)								
		5% AEP			1% AEP			1% AEP +38%CC		
		Baseline	Scheme	Difference	Baseline	Scheme	Difference	Baseline	Scheme	Difference
DB_1144	Upstream of the west A12 Road culvert	23.76	23.60	-0.16	23.95	23.83	-0.12	24.10	24.05	-0.05
DB_0956i	Downstream of the west A12 Road culvert	23.19	23.19	0.00	23.39	23.40	0.00	23.59	23.59	0.00
DB_0426	540m downstream of the proposed carriageway culvert	21.88	21.88	0.00	22.25	22.25	0.00	22.75	22.75	0.00

Note: No long profiles are provided as the data would be incomparable between baseline and with-scheme scenarios since the chainage is different between the two scenarios.

Annex D **Flood Maps**

Plate D.1 Flood Extent Comparison between Baseline and With Scheme Scenarios for the 5% AEP Event

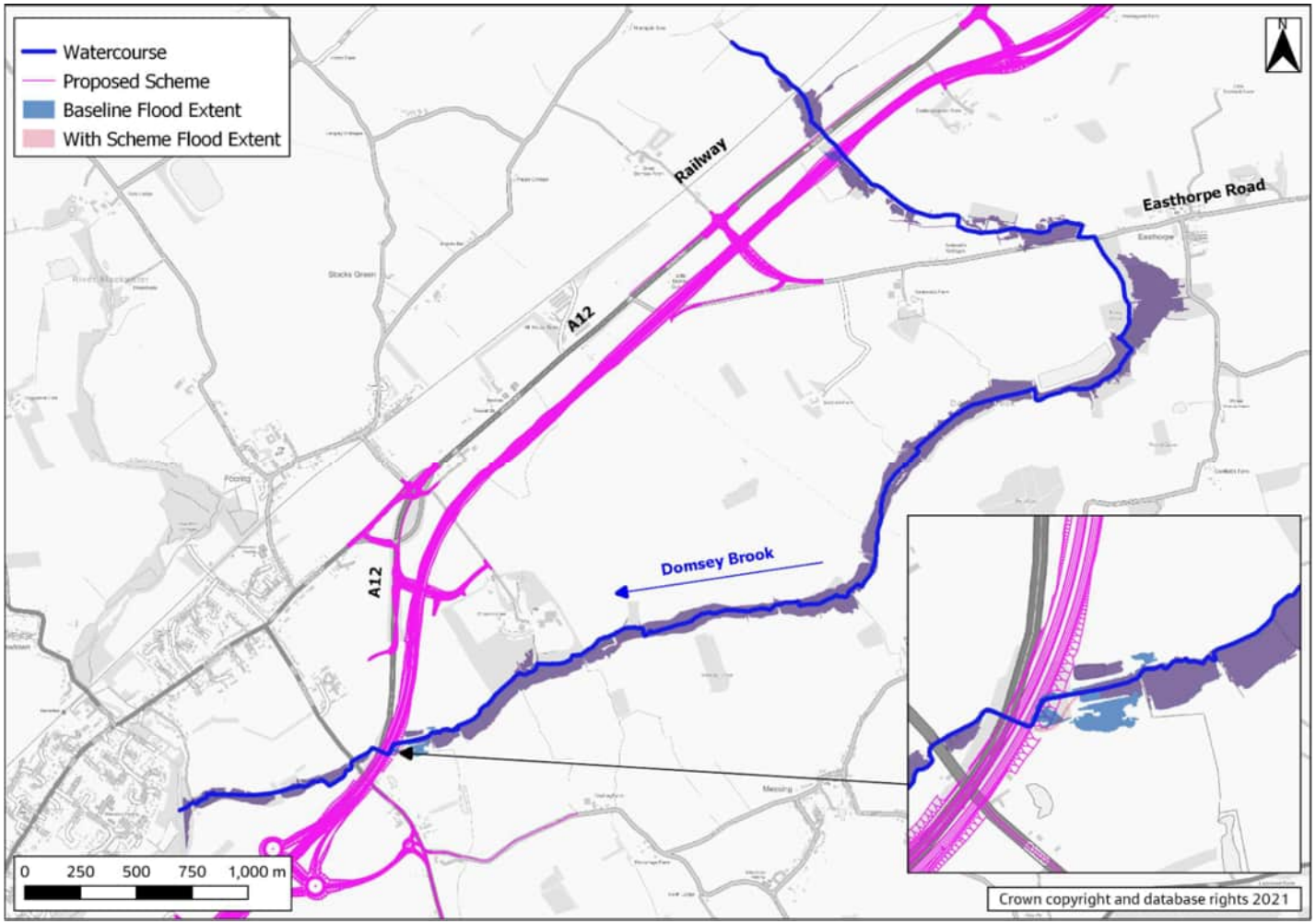


Plate D.2 Flood Extent Comparison between Baseline and With Scheme Scenarios for the 1% AEP Event

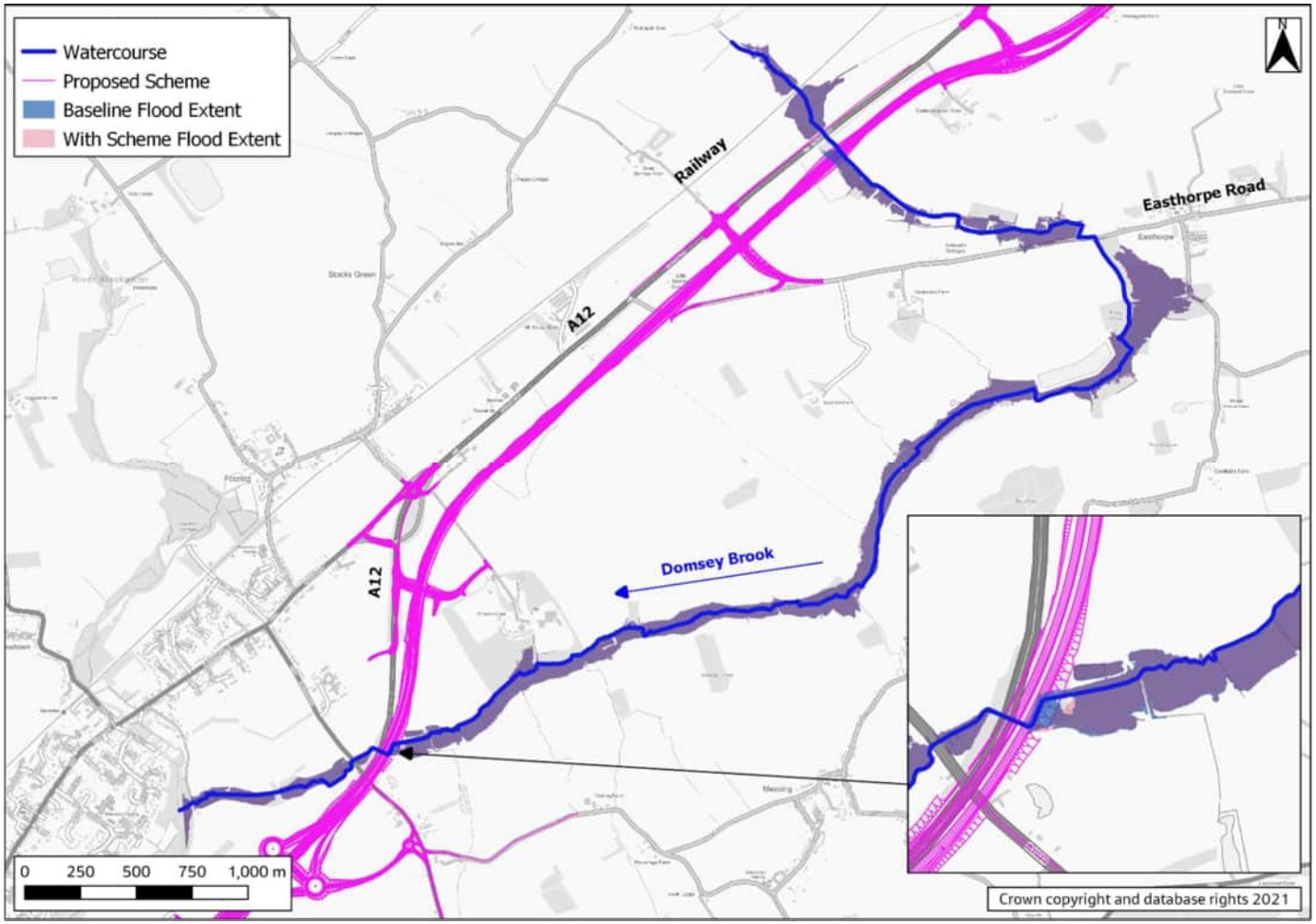


Plate D.3 Flood Extent Comparison between Baseline and With Scheme Scenarios for the 1% AEP + 38%CC Event

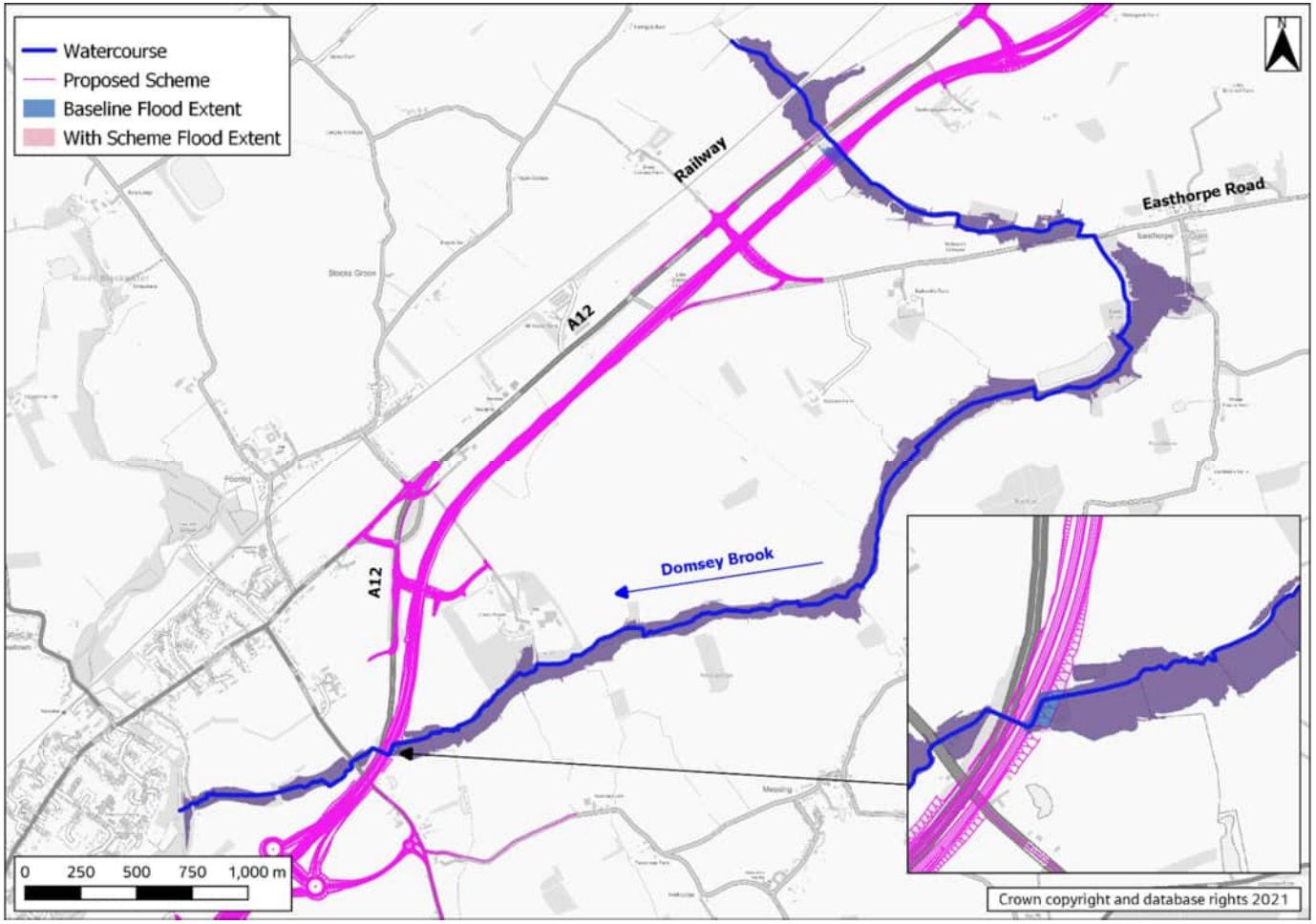


Plate D.4 Water Level Difference (With Scheme Minus Baseline) for the 5% AEP Event

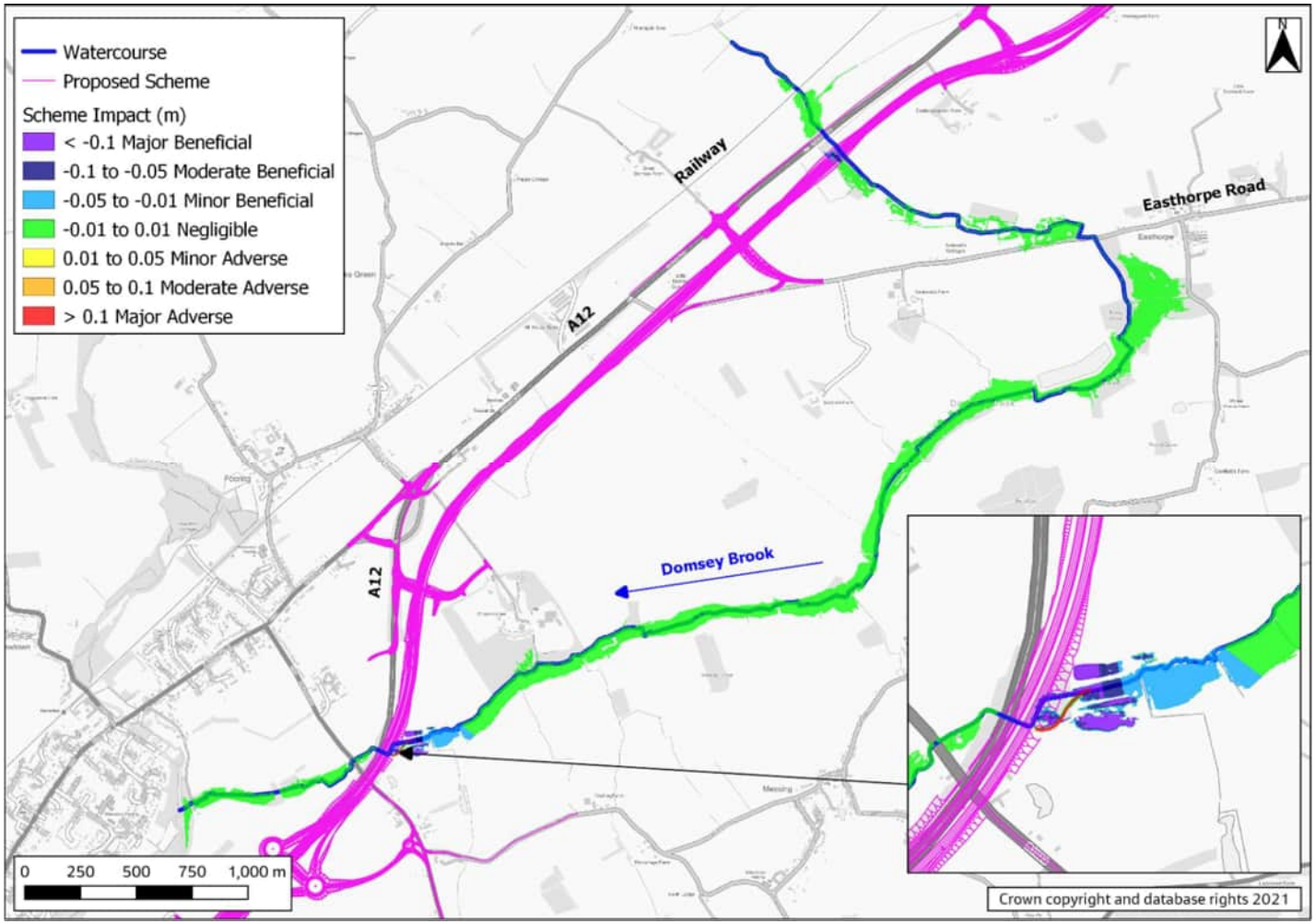


Plate D.5 Water Level Difference (With Scheme Minus Baseline) for the 1% AEP Event

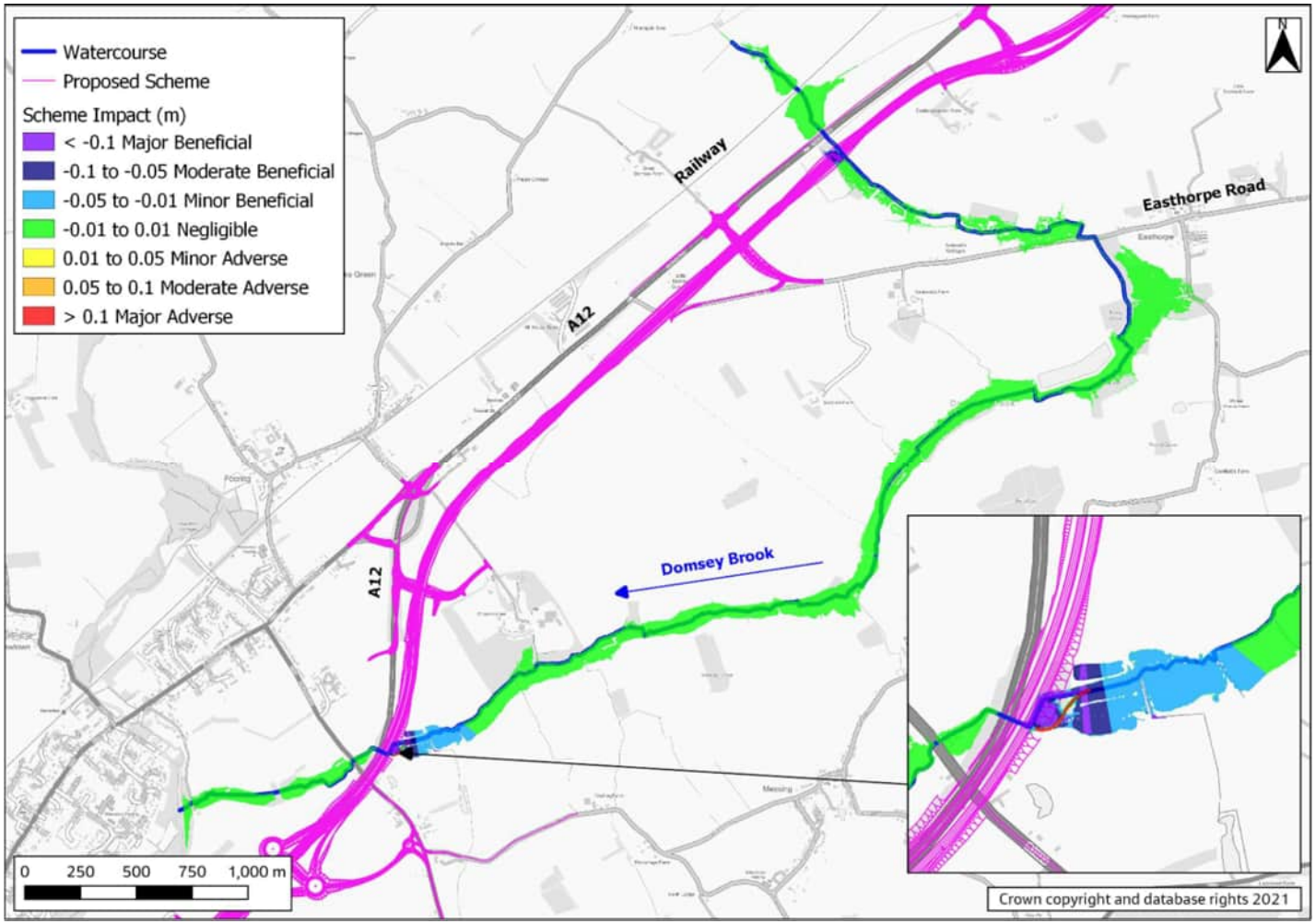
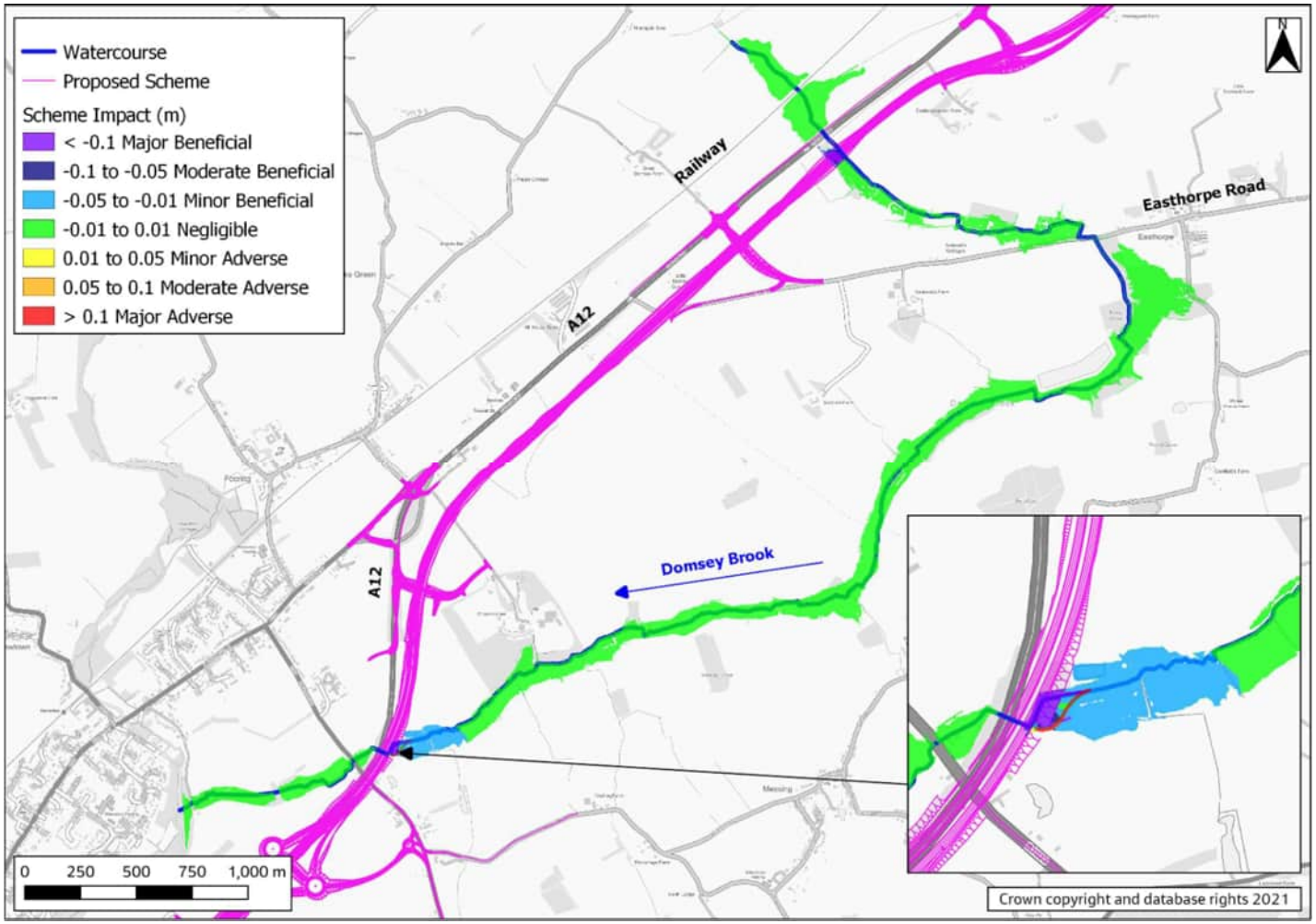


Plate D.6 Water Level Difference (With Scheme Minus Baseline) for the 1% AEP + 38%CC Event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT DOMSEY BROOK HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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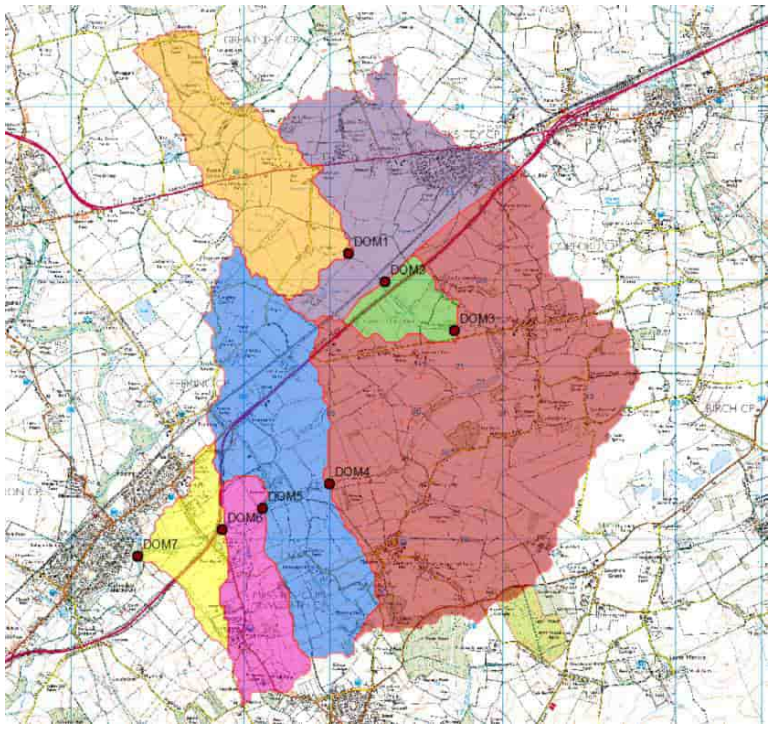
1 Introduction

1.1.1 This document is to provide a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This record sheet is to be used where flood estimates are needed at a single location.

2 Method Statement

2.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Approx. no. of flood estimates required • Peak flows or hydrographs? • Range of return periods and locations • Approx. time available	Highways England are planning Highway Improvements on the A12 from junction 19 to junction 25. A number of potential routes have been proposed. The current route of the road crosses the Domsey Brook in 2 locations, to the east of Kelverdon where the road is on embankment, and midway between Kelverdon and Marks Tey where the road is at ground level. The objective of the proposed study (hydrological assessment component) are: to produce baseline and design hydrology for a river hydraulic models, namely of Domsey Brook. 7 locations along the watercourse have been requested by the hydraulic modellers for inputs into the hydraulic model. These reflect where there are river confluences or locations of crossings. A full hydrograph is required for each of the events 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 200 and 1000-year design return periods). Climate change allowances – The climate change allowance for the River Blackwater Operational Catchment Area has been taken from the UKCP18 peak river flow climate change allowances. The upper climate change allowance for the River Blackwater Operational River Basin is 38% to the 2080s.



2.2 Overview of the catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The catchment of the Domsey Brook covers as area of 26.6km2 to the downstream model extent at its confluence with the River Blackwater. It lies on a bedrock of sedimentary clay, silt, sand and gravel and has soils with slightly impeded drainage. The baseflow index is 0.365 with a relatively high standard percentage runoff of 46.4. The catchment is essentially rural and receives a low average annual rainfall of 565mm.

2.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – Version 8, September 2019

2.4 Gauging stations (flow or level)

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
Blackwater	Stisted		37017	TL792242	139.2	flat v crump weir	01/1969 - present
Blackwater	Appleford Bridge		37010	TL844158	247.3	trapezoidal flume	01/1962 - present
Roman River	Bounstead Bridge		37021	TL984205	52.6	flat v crump weir	01/1965 - 08/1991
River Ter	Crabbs Bridge		37003	TL786107	77.8	trapezoidal flume	01/1932 - present

2.5 Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling ?	Data quality check needed ?	Other comments on station and flow data quality – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Stisted	1969 - 2017	N	Y	N		AMAX data corrected for non-modular flows
Appleford Bridge	1962 - 2017	N	Y	Y		Irregular record. Exceptionally high flow in 2001 considered suspect
Bounstead Bridge	N/A					
Crabbs Bridge	1963 - 2017	Y	Y	Y	N	AMAX data corrected for non-modular flows
Give link/reference to any further data quality checks carried out						

2.6 Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Reasons – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Stisted	Velocity area at high flows	N	
Appleford Bridge	High flow rated spillway	N	Rating accounts for mon-modular flow
Bounstead Bridge	N/A	N	
Crabbs Bridge	Theoretically rated	N	
Give link/reference to any rating reviews carried out		N/A	

2.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data availabl e?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)					
Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies					
Other data or information (e.g. groundwater, tides)					

2.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes – there is nothing unusual about the catchment that would mean that the methods were not acceptable
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Locations have been selected at tributary inflows and road crossings
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	None
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	Both the statistical method and ReFH2 have been selected and the peak flow compared at the target location(s), i.e. at the River Blackwater confluence, The catchment has been broken down into seven sub-catchments, with 5 point inflows and 2 lateral catchments. An additional flow estimate has been calculated for the downstream end to allow flow consolidation.

Software to be used (with version numbers)	Peak flow data set v8 September 2019WINFAP-FEH v4[1] / ReFH 2.3

3 Locations where flood estimates required

3.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on web service (km^2)	Revised AREA if altered
CONF	Domsey Brook	To confluence with Blackwater	586800	218800	26.62	26.6
DOM1	Domsey Brook	Upstream model extent	589200	222300	3.03	2.97
DOM2	Left bank tributary	Marks Tey	590350	222300	2.86	3.81
DOM3	Lateral catchment	Contributing area downstream of DOM2	590416	221443	N/A	0.80
DOM4	Lateral catchment	Contributing area downstream of DOM3	589000	219650	N/A	11.56
DOM5	Left and right bank tributaries	Messing and Ferring	588300	219550	N/A	4.70
DOM6	Left bank tributary	Inworth	588050	219150	1.25	1.60
DOM7	Domsey Brook	Lateral catchment from A12 to Blackwater confluence	586800	218800	N/A	1.20
Reasons for choosing above locations						

3.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	BFIHOST19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
CONF	1.00	0.23	0.365	0.348	5.94	14.8	565	46.4	0.0176	0.173
DOM1	1.00	0.23	0.395	0.372	1.91	8.6	564	46.8	0.0045	0.274
DOM2	1.00	0.23	0.380	0.352	1.67	6.6	569	46.5	0.0780	0.336
DOM3	1.00	0.23	0.380	0.352	0.90	21.9	566	56.7	0.0000	-

[1] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

Site code	FARL	PROPWET	BFIHOST	BFIHOST19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
DOM4	0.99	0.23	0.332	0.319	3.67	14.9	566	46.3	0.0260	-
DOM5	1.00	0.23	0.363	0.347	0.36	1.67	565	46.2	0.0230	0.181
DOM6	1.00	0.23	0.257	0.268	0.26	1.39	565	49.2	0.0060	0.042
DOM7	1.00	0.23	0.533	0.474	1.11	23.2	565	40.1	0.0010	-

3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Additional catchment area has been measured from Ordnance Survey 1:25,000 maps. Where there is little change to the catchment area, other descriptors have not been changed from those provided on the FEH web service. Lateral catchments have been using area weighting - calculated based on FEH catchments to the study location and the next one upstream.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	Reference was made to the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html and Soilscapes http://www.landis.org.uk/soilscapes/
Source of URBEXT	FEH web viewer
Method for updating of URBEXT	Calculation of UAF is based on the new UAF model implemented in WINFAPv4 and described in WINFAP 4 Urban Adjustment Procedures Issue 1.01

4 Statistical Method

4.1 Search for donor sites for QMED (if applicable)

Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	The Domsey Brook is not gauged. The River Blackwater is located to the west of the Domsey Brook and is gauged both upstream (Stisted 37017) and downstream (Appleford Bridge 37010) of the confluence. Both of these catchments are larger than the study catchment at 139 and 247km^2 respectively. The upper Blackwater catchment is chalk. The River Ter is further away from the study catchment and is gauged at Crabbs Bridge (37003). The catchment area is more similar to the study catchment at 77.8km^2. The geology differs to the study catchment being underlined by London Clay but has a similar SPRHOST 41.76.

4.2 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)
37017	Rejected due to artificial transfer from Ely Ouse scheme, predominantly chalk in the upper catchment.	Not applicable				
37010	Rejected - groundwater-influenced catchment with major artificial influences.					
37017	No flow data					
37003	Accepted – nearby rural catchment with data suitable for QMED calculation.	AM	N	5.430	6.509	0.83
Which version of the urban adjustment was used for QMED at donor sites, and why? Note: The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8).						

4.3 Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average	
DOM1	DT	0.48	37003	15.6	0.337	0.941	N/A	N/A	0.44
DOM2	DT	0.81	37003	17.5	0.324	0.943	N/A	N/A	0.76
DOM5	DT	0.75	37003	15.1	0.341	0.940	N/A	N/A	0.71
DOM6	DT	0.37	37003	14.8	0.343	0.940	N/A	N/A	0.35
CONF	DT	3.29	37003	16.4	0.332	0.942	N/A	N/A	3.10
Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?									
Which version of the urban adjustment was used for QMED, and why?						Calculation of UAF is based on the new UAF model implemented in WINFAPv4 and described in WINFAP 4 Urban Adjustment Procedures Issue 1.01			

Notes
Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.
When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added.
When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why.
The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.
The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.
If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

4.4 Derivation of pooling groups

4.4.1 The composition of the pooling groups is given in the Annex. Several subject sites may use the same pooling group.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons Note also any sites that were investigated but retained in the group.	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
Domsey at Blackwater	CONF	N	33054, 33032, 26013, 26003 removed as BFIHOST>0.65 (SPRHOST < 20%); 37016, 35017, 30004 added to increase stations years to 500.	L-CV = 0.285 L-SKEW = 0.114
Notes Pooling groups were derived using the revised procedures from Science Report SC050050 (2008). The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.				

4.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (Error! Reference source not found.)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustments	Growth factor for 100-year return period
CONF	P	Domsey Brook at River Blackwater confluence	GL – best fit to countrywide data Z value = 1.9697	Urban adjustment made to growth curve	Location = 1.000 Scale = 0.295 Shape = -0.114	2.783
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters. Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010). Growth curves were derived using the revised procedures from Science Report SC050050 (2008).						

4.6 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following events									
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	0.5% (200-yr)	0.1% (1000yr) *
CONF	3.1	4.7	5.6	6.6	7.1	7.9	8.5	9.0	10.2	17.7
DOM1	0.44	0.63	0.76	0.89	0.97	1.08	1.16	1.22	1.38	2.41
DOM2	0.76	1.10	1.32	1.54	1.68	1.86	2.01	2.12	2.39	4.2
DOM5	0.71	1.02	1.23	1.44	1.57	1.74	1.87	1.98	2.23	3.89
DOM6	0.35	0.51	0.61	0.71	0.77	0.86	0.92	0.97	1.10	1.92
1000 year flows derived by applying the ratio of the 1000:100 from the ReFH method. Ratio = 1.97										

5 Revitalised flood hydrograph (ReFH) method

5.1 Parameters for ReFH model

Site code	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
CONF	CD	12.039	298.859	47.171	1.09
DOM1	CD	7.473	318.084	38.350	1.322
DOM2	CD	7.527	301.981	36.016	1.140
DOM3	CD	3.218	301.981	31.473	1.177
DOM4	CD	9.128	277.173	40.298	0.866
DOM5	CD	6.457	298.084	35.705	1.097
DOM6	CD	3.931	242.782	29.331	0.541
DOM7	CD	3.978	414.582	39.389	2.538
Brief description of any flood event analysis carried out (further details should be given below or in a project report)		N/A			

5.2 Design events for ReFH method

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
CONF	Rural	Winter	18.0	
DOM1	Rural	Winter	11.0	
DOM2	Rural	Winter	11.0	
DOM3	Rural	Winter	5.5	
DOM4	Rural	Winter	15.0	
DOM5	Rural	Winter	11.0	
DOM6	Rural	Winter	6.5	
DOM7	Rural	Winter	6.5	
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?				

5.3 Flood estimates from the ReFH method

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+38% (100yr +cc)	0.5% (200-yr)	0.1% (1,000-yr)
CONF	4.27	5.57	6.45	7.32	7.87	8.64	9.38	10.01	13.81	12.19	19.66
DOM1	0.61	0.81	0.95	1.08	1.17	1.29	1.41	1.50	2.07	1.83	2.97
DOM2	0.86	1.14	1.33	1.52	1.64	1.81	1.97	2.11	2.91	2.56	4.18
DOM3	0.29	0.40	0.48	0.56	0.60	0.67	0.73	0.78	1.08	0.93	1.55
DOM4	2.46	3.24	3.77	4.29	4.62	5.09	5.53	5.91	8.16	7.20	11.71
DOM5	1.16	1.55	1.81	2.07	2.23	2.46	2.67	2.85	3.93	3.47	5.77
DOM6	0.71	0.97	1.14	1.32	1.43	1.57	1.71	1.82	2.51	2.20	3.65
DOM7	0.26	0.35	0.41	0.48	0.52	0.57	0.62	0.67	13.81	0.81	1.35

6 Discussion and summary of results

6.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	ReFH	Statistical	Ratio	ReFH	Statistical	Ratio
CONF	4.3	3.2	1.34	10.0	9.0	1.11
DOM1	0.6	0.4	1.50	1.5	1.2	1.30
DOM2	0.9	0.6	1.50	2.1	1.8	1.17
DOM5	1.2	0.6	2.00	2.9	1.6	1.81
DOM6	0.7	0.4	1.75	1.8	1.0	2.90

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The statistical method is not considered appropriate for use on lateral catchment inflows. The ReFH2 has therefore been used to provide a consistent approach to flow estimation across the Domsey Brook catchment. Comparison of flow estimates by the 2 approaches show greater flows using the ReFH2 methods across all catchments except DOM2 which is close to unity.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The catchment areas have been more accurately determined using the OS mapping.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	Larger return periods become less reliable due to the level of extrapolation required. Recorded river flow records used in the development of the estimation methods are generally much shorter than the return period of interest.
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).	Confidence limits for statistical estimates of QMED at CONF are 68% lower 2.3, upper 5.0 95% lower 1.6, upper 7.0
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	
Give any other comments on the study, for example suggestions for additional work.	

6.4 Checks

Are the results consistent, for example at confluences?	The comparison of 100 year flows at the downstream end of the model CONF. ReFH2 Direct calculation – 10.0m^3/s Sum of inflows –15.5m^3/s
What do the results imply regarding the return periods of floods during the period of record?	No recorded flows to compare
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	100 year growth factor for the whole catchment to confluence is 2.783 from the statistical method 2.366 from ReFH2 This is within the guidance
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	Ratio of 1,000 to 100 year flow at CONF the downstream of the model at the Blackwater confluence is 19.7/10 = 1.97
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	QMED specific runoffs are 1.62 from ReFH2 1.20 from statistical
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No earlier studies have been provided
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	N/A

6.5 Final results

Target flows

6.5.1 Below are the target flows for the downstream end of the model using the critical catchment duration of 18 hours.

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+38% (100yr +cc)	0.5% (200-yr)	0.1% (1,000-yr)
CONF	4.27	5.57	6.45	7.32	7.87	8.64	9.38	10.01	13.81	12.19	19.66

Model inflows

6.5.2 The ReFH2 model was run for each catchment with a critical catchment duration of 18 hours. The results below show the resultant peak flows for each sub-catchment.

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+CC (100yr +cc)	0.5% (200-yr)	0.1% (1,000-yr)
DOM1	0.60	0.80	0.93	1.06	1.14	1.26	1.37	1.47	2.03	1.79	2.88
DOM2	0.85	1.12	1.30	1.48	1.60	1.76	1.92	2.05	2.83	2.51	4.05
DOM3	0.25	0.33	0.39	0.45	0.48	0.54	0.58	0.63	0.87	0.77	1.25
DOM4	2.45	3.21	3.73	4.25	4.57	5.03	5.47	5.84	8.06	7.12	11.55
DOM5	1.11	1.48	1.72	1.96	2.12	2.33	2.53	2.71	3.74	3.32	5.49
DOM6	0.61	0.82	0.95	1.09	1.17	1.29	1.41	1.50	2.07	1.85	3.05
DOM7	0.24	0.32	0.37	0.43	0.46	0.51	0.55	0.59	0.81	0.73	1.20

Acronyms

Abbreviation	Term
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1989
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

Annex A

A.1 Supporting Information

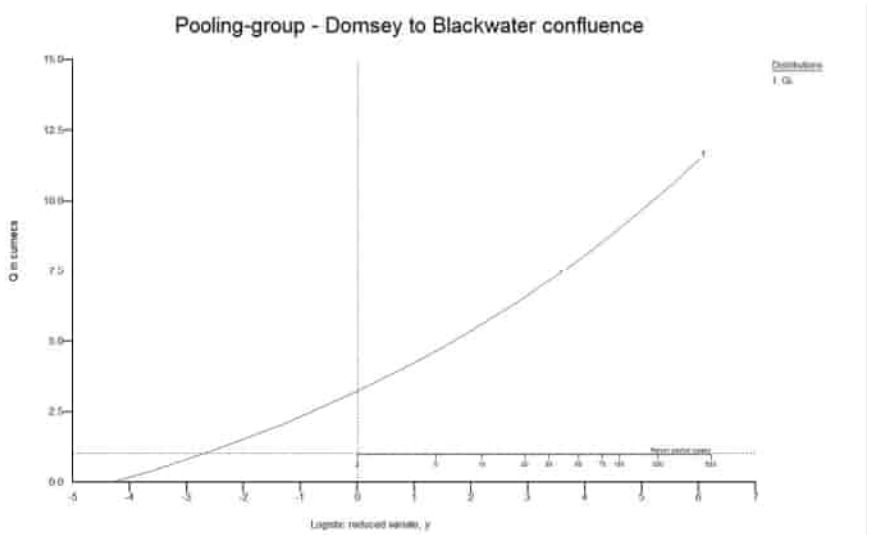
A.1.1 Pooling group composition

Table A.1 Pooling Group Composition

FINAL POOLING GROUP COMPOSITION – DOMSEY BROOK TO BLACKWATER CONFLUENCE

Station	Distance	Years of data	QMED AMAX	L-CV	L-SKEW	Discordancy
36003 (Box @ Polstead)	1.381	57	3.91	0.305	0.089	0.453
34005 (Tud @ Costessey Park)	1.436	57	3.146	0.269	0.168	0.895
36010 (Bumpstead Brook @ Broad Green)	1.437	51	7.5	0.372	0.184	1.139
41020 (Bevern Stream @ Clappers Bridge)	1.443	49	13.66	0.203	0.181	1.734
36004 (Chad Brook @ Long Melford)	1.502	51	5.186	0.294	0.182	0.826
36007 (Belchamp Brook @ Bardfield Bridge)	1.517	53	4.63	0.371	0.119	1.344
37003 (Ter @ Crabbs Bridge)	1.631	53	5.43	0.269	-0.007	1.831
37016 (Pant @ Copford Hall)	1.687	54	7.24	0.293	0.096	0.06
53017 (Boyd @ Bitton)	1.733	45	13.87	0.241	0.088	0.682
30004 (Lymn @ Partney Mill)	1.74	56	6.983	0.229	0.046	1.036
Total		526				
Weighted means				0.285	0.114	

Plate A.1 Domsey Brook Growth Curve



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT ROMAN RIVER MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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0 Executive summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at the Roman River. The modelling and hydrology have been carried out in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 1D-2D linked type using Flood Modeller software (Jacobs 2022) for the 1D domain and TUFLOW Classic (WBM 2022) for the 2D domain and was built from scratch.

0.1.2 Three events were simulated: the 5%, 1% and 1% AEP with 65% climate change. Flow reconciliation was carried out to match the design hydrology. No data was available for calibration, but the model was verified using Environment Agency (EA) flood maps. The model was then used to simulate baseline and with-scheme scenarios for the above three design events. The with-scheme scenario involved an extension to the A12 culvert and an offset of the channel alignment immediately downstream from the A12 crossing. Sensitivity analysis to roughness, inflows and the downstream boundaries was carried out to further validate the results.

0.1.3 The analysis shows that the proposed scheme has no significant impact on flooding at the Roman River and therefore no mitigation is proposed.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed scheme includes widening the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) is required for the proposed scheme to meet relevant local and national planning legislation and inform the design and planning process. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic models with associated catchment hydrology. Water levels for the baseline and with scheme scenarios both upstream and downstream of the proposed scheme and the associated flood extents were determined for a range of storm flood events at each watercourse crossing.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA are:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River (this report)
- Five minor watercourses were also modelled for the proposed scheme

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out for the Roman River, to assess the baseline and “with scheme” scenarios. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 For the A12 crossing of the Roman River, the highway will be upgraded to three lanes per carriageway. This will require widening of the highway embankment on its south side. Changes include extension of the culvert under the highway and realignment of part of the channel.

1.1.6 This report should be read in conjunction with the Flood Risk Assessment report (Jacobs 2022).

1.2 Methodology

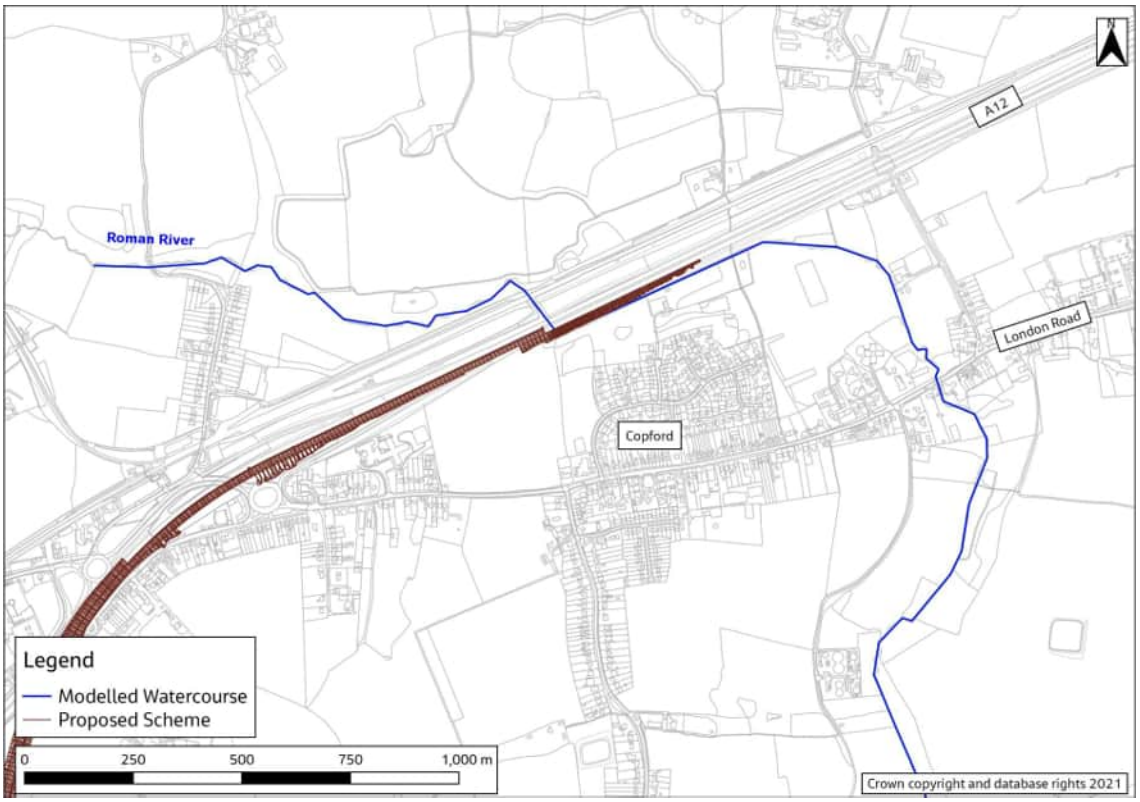
1.2.1 The hydraulic model uses a linked one-dimensional/two-dimensional (1D/2D) schematisation, where the river channel is represented as a 1D component and is linked to the floodplain, which is represented by a 2D domain. The 1D component was constructed using the Flood Modeller Pro (version 4.6) river modelling package, and the 2D component was constructed using TUFLOW (version 2020-01-AA-iDP-w64).

1.3 Study Area

1.3.1 The Roman River is situated to the west of Colchester town in Essex, flowing from north to south under the A12 (Plate 1.1).

1.3.2 The model domain includes a 1km reach of the Roman River upstream of the A12, where the river flows mostly through agricultural area. After crossing the A12, the river flows alongside it, and then crosses the eastern outskirts of Copford town.

Plate 1.1 Roman River study area



2 Input Data

2.1.1 The data used to construct the hydraulic model of the Roman River is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR	1m and 2m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) data. Used to define 2D model grid	Environment Agency 2020
OS maps	Mastermap data	Highways England
Channel survey	In-channel cross sections and hydraulic structures	Jacobs March 2020
Hydrological analysis	Hydrological analysis carried out for Roman River	Jacobs 2020
Culvert under A12	HADDMS* records for Culvert dimensions and location	Highways England 2020

*HADDMS - Highways Agency Drainage Data Management System

3 Hydrology

3.1.1 The details of the hydrological analysis carried out to produce design inflows for the hydraulic model are provided in Annex A of this report. A summary of this analysis is presented below.

3.1.2 The catchment of the Roman River covers an area of 19.7km^2 to the downstream model extent to the west of Colchester. It lies on a bedrock of London Clay Formation - Clay, Silt and Sand and has soils that are slightly acid loamy and clayey soils with impeded drainage. The baseflow index (BFIHOST) is 0.545 with a standard percentage runoff (SPRHOST) of 42.1. The catchment is essentially rural and receives a low average annual rainfall of 563mm. Although the Roman River is gauged at Bounstead Bridge (37017), hydrometric data is not available on the National River Flow Archive (2019) after 1991 due to continuous problems with drowning thereby producing poor quality data. As this is not a peak flow rated gauge and no suitable hydrometric data is available, the Roman River catchment has been treated as an ungauged catchment for the purpose of flood flow analysis.

3.1.3 Peak design flows were estimated using the FEH (Flood Estimation Handbook) Statistical method and ReFH2.3 method. From the comparison of the two methods, the peak flows from the ReFH2.3 method were found to be slightly higher than those from the statistical method and hence the ReFH2.3 peaks were adopted. The ReFH2.3 method-based hydrograph shapes were used to derive the model inflow hydrographs.

3.1.4 The FEH catchment descriptors of the Roman River have been purchased from FEH Web Service at five locations along the modelled reach of the river, namely;

- one at the upstream model extent (termed hereafter as ROM1; easting: 591450, northing: 224400),
- second for the residual catchment upstream of left bank tributaries (termed hereafter as ROM2; easting: 592450; northing: 224350),
- third – left bank tributaries (termed hereafter as ROM3; easting: 592969; northing: 224568),
- fourth – residual catchment downstream of A12 up to the downstream model extent (termed hereafter as ROM4; easting: 593450; northing: 223700); and
- and finally - at the downstream modelling extent (termed hereafter as TOT; easting: 593450, northing: 223700).

3.1.5 The above sub-catchment schematisation is shown in Plate 3.1.

3.1.6 Four inflow hydrographs (ROM1, ROM2, ROM3, ROM4) have been produced for the following Annual Exceedance Probabilities (AEP) events (see Plate 3.2):

- 5% AEP event (1 in 20 years); and
- 1% AEP event (1 in 100 years).

3.1.7 Additionally, inflow hydrographs were also produced for the 1% AEP plus Climate Change assuming a 65% uplift.

3.1.8 Table 3.1 below present the estimated peak flows for ROM1, ROM2, ROM3, ROM4 and TOT for an 18-hour winter storm duration, which is the critical storm duration at the downstream modelling extent.

3.1.9 The modelled flow at the downstream modelling extent (target location - TOT) was reconciled with the target flow estimated at that location, using the scaling factors presented in the last row of Table 3.1.

3.1.10 All inflows were scaled with the same scaling factor as shown in the last row of Table 3.1 for a given AEP event to achieve flow reconciliation with the target flow at the downstream modelling extent (TOT; model node: ROMR_0000).

Table 3.1 Design peak inflows (m³/s) - unscaled

AEP event	5%	1%	1% + 65%CC
	Target peak flow at the downstream modelling extent – ReFH2.3		
TOT (Storm Duration = 18hrs, Winter)	3.46	4.78	7.89
	Model inflows – ReFH2.3 pre-reconciliation		
ROM1 (Storm Duration = 18hrs, Winter)	3.05	4.22	6.96
ROM2 (Storm Duration = 18hrs, Winter)	0.31	0.45	0.74
ROM3 (Storm Duration = 18hrs, Winter)	0.49	0.71	1.17
ROM4 (Storm Duration = 18hrs, Winter)	0.77	1.07	1.77
Scaling factor	0.78	0.78	0.82

Plate 3.1 Roman River hydrology schematisation

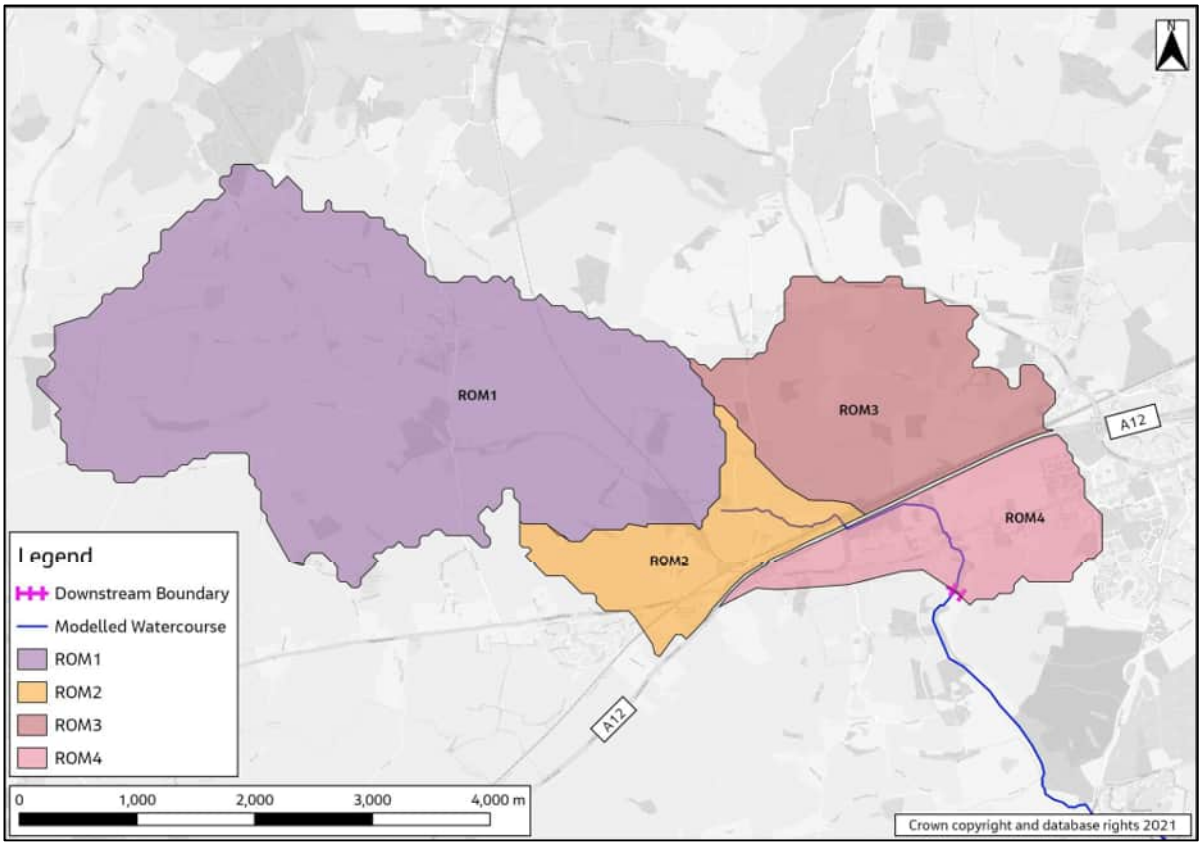
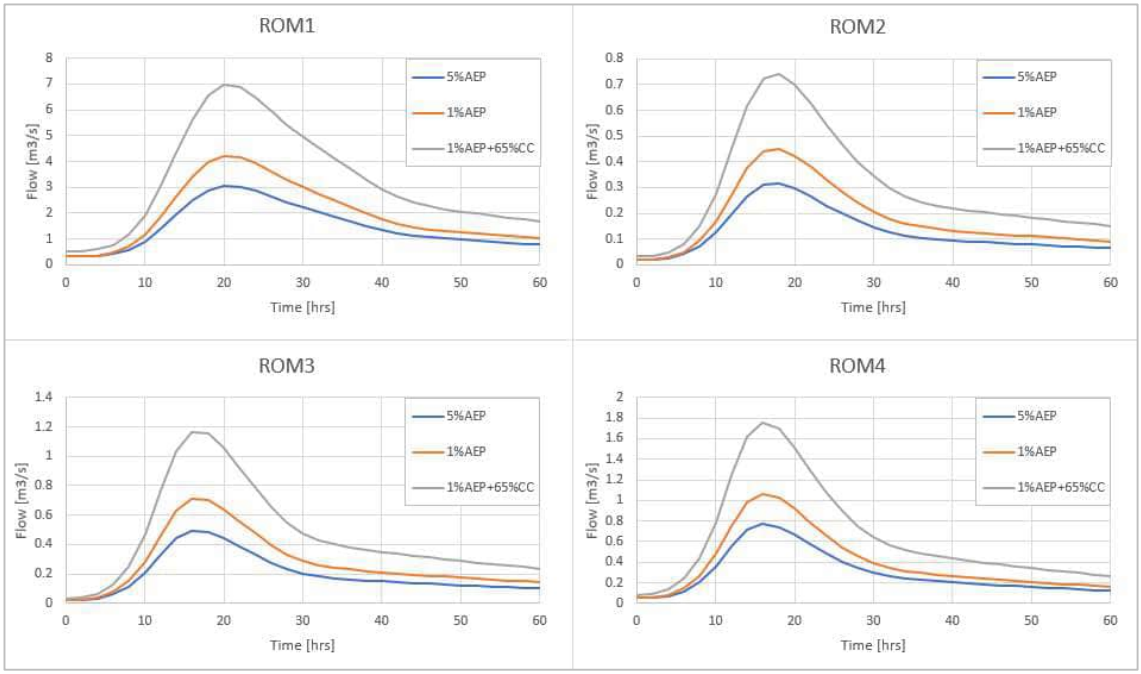


Plate 3.2 Inflow hydrographs (unscaled)



4 Baseline model build

4.1 Watercourse schematisation – 1D Domain

4.1.1 Surveyed cross section data has been used to inform the in-channel geometry of the modelled watercourses. To aid model performance interpolated cross sections were added between the surveyed cross sections. The spacing of the interpolated cross sections ranged from 30m to 100m.

4.1.2 The locations of key features in the model are shown in Plate 4.1, including hydraulic structures in the model. A structure schedule is presented in Annex B.

4.1.3 Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using site visit information. Generally, gravel can be seen in the bed of Roman River and grass and heavy vegetation on the floodplain. The Roman River channel itself is vegetated over at some locations. Roughness values adopted were taken from standard guidance (Chow 1959) and are presented in Table 4.1.

4.1.4 The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4.2.

Plate 4.1 Roman River baseline model schematisation

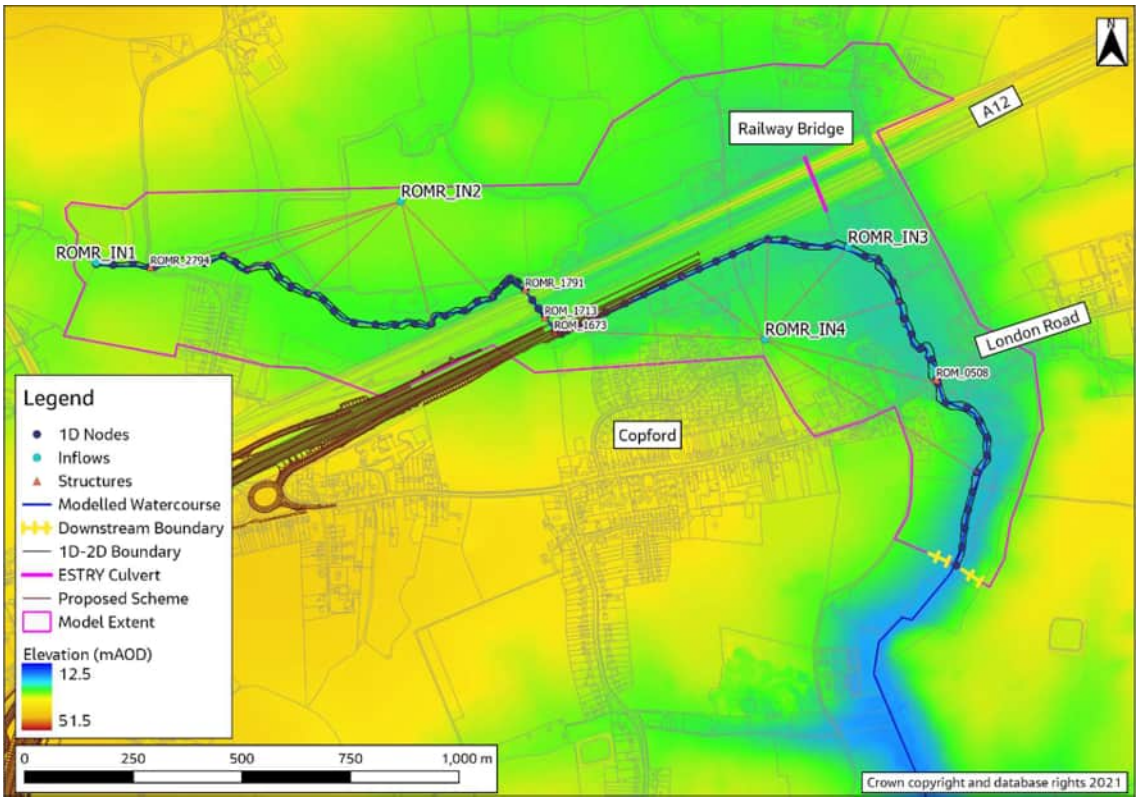


Table 4.1 Manning's 'n' coefficients – 1D domain

Flood Modeller Nodes	Bed Manning's 'n'	Bed Material	Bankside Manning's 'n'	Bankside Material
ROMR_2925	0.038	Gravel - with vegetation and debris	0.06	Long grass Heavy vegetation/trees
ROMR_2794 to ROMR_2783	0.035	Gravel	0.04	Grass
ROMR_2618	0.043	Vegetation - with weeds and stone	0.06	Long grass Heavy vegetation/trees
ROMR_2556 to ROMR_2385	0.040	Vegetation	0.06	Long grass Heavy vegetation/trees
ROMR_2067	0.035	Gravel	0.06	Long grass Heavy vegetation/trees
ROMR_1895	0.038	Gravel - with vegetation and debris	0.06	Long grass Heavy vegetation/trees
ROMR_1791 to ROMR_1713	0.035	Gravel	0.06	Long grass Heavy vegetation/trees
ROMR_1673 to ROMR_1638	0.038	Gravel - with vegetation and debris	0.06	Long grass Heavy vegetation/trees
ROMR_1347	0.035	Gravel	0.06	Long grass Heavy vegetation/trees
ROMR_1131	0.038	Gravel - with vegetation and debris	0.06	Long grass Heavy vegetation/trees
ROMR_0941C to ROMR_0941	0.040	Vegetation	0.06	Long grass Heavy vegetation/trees
ROMR_0749 to ROMR_0000	0.035	Gravel	0.06	Long grass Heavy vegetation/trees

Table 4.2 Boundary conditions – 1D domain

Flood Modeller Node	Boundary Type	Description
ROMR_IN1	QT Boundary	ReFH2.3 hydrograph. Applied at the upstream end of Roman River
ROMR_IN2	QT Boundary	ReFH2.3 hydrograph. Applied laterally between the upstream end of Roman River and the tributary
ROMR_IN3	QT Boundary	ReFH2.3 hydrograph. Applied at the small tributary, downstream of the A12 crossing
ROMR_IN4	QT Boundary	ReFH2.3 hydrograph. Applied laterally from downstream of the A12 and the downstream end of the model on Roman River
ROMR_0000	Normal depth downstream boundary	Normal depth boundary condition applied to the downstream end of Roman River. Sensitivity testing has shown that this boundary assumption does not affect model results in the area of interest (see Section 6.4 Sensitivity Analysis)

4.2 Flood plain schematisation – 2D domain

4.2.1 The 2D domain covers an area of 1.2km^2, shown in Plate 4.1.

4.2.2 The topography of the floodplain is represented using a 2m resolution square grid. The model grid has been oriented to align with the main flow direction along the scheme (highway embankment). The ground levels for the grid cells are based on a Digital Terrain Model (DTM) derived from 1m and 2m resolution LiDAR data.

4.2.3 A review of the floodplain using available aerial and OS mapping has shown that there is a culvert underneath the A12, located 700m east of the Roman River crossing. This required representation in the model. The structure was implemented in the model domain as a 1D ESTRY element (see Plate 4.1). The location and dimensions of the culvert were confirmed and applied based on HADDMS (Highways Agency Drainage Data Management System) records (130m length, 0.9 m diameter). Culvert invert elevations were not available in HADDMS database, so the LiDAR ground levels at the entrance and exit of the culvert were applied for the upstream and downstream culvert invert levels. No other modifications were required within the 2D domain.

4.2.4 Hydraulic roughness coefficients are applied over each grid cell of the 2D domain, as shown in Table 4.3, depending on land use taken from OS

Mastermap data. Roughness values adopted were taken from standard guidance (Chow, 1959).

Table 4.3 Manning's 'n' coefficients – 2D domain

Land use	Manning's n
Buildings	1.000
General surface, residential yards	0.040
General surface, grass, parkland	0.050
Glasshouse	1.000
Inland water	0.035
Natural environment (coniferous/non coniferous trees) heavy woodland and forest	0.100
Roads tracks and paths, tarmac or dirt tracks	0.025
Rail	0.500
Roads tracks and paths, tarmac or dirt tracks	0.020
Roads tracks and paths (roadside) pavement	0.020
Structures, roadside structure	0.030
Land (unclassified), industrial yards, car parks	0.035
Slope	0.040

4.2.5 No inflow has been applied directly in the 2D domain. Any flow within the 2D domain is as a result of the 1D channel being overtopped. Table 4.4 describes the downstream boundary condition used in the 2D domain. Its location is shown in Plate 4.1.

Table 4.4 Boundary conditions – 2D domain

Type of Boundary	Flood Modeller Node	Description
Stage-Discharge	HQ Boundary	Free flow normal depth boundary applied at the downstream extent of the model with slope calculated using LiDAR.

4.2.6 The link between the 1D and the 2D domains was defined along the left and right banktops of the Roman River. To ensure that the correct level is used for the 1D/2D link, banktop lines have been digitised, using levels from the LiDAR data.

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 65% uplift is in line with the climate change guidance for this area available at the time of the model analysis (Environment Agency 2017a). Based on the model analysis outcomes there was no requirement to update the climate change allowance to the new 2021 guidance (Environment Agency 2021).

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope.

Table 5.1 Modelled events

Scenario	AEP Event		
	5%	1%	1% +65%CC
Baseline	X	X	X
With Scheme	X	X	X
Roughness Sensitivity (1D and 2D)		X	
Hydrological Inflow Sensitivity		X	
Downstream Boundary Sensitivity (1D and 2D)		X	

6 Model Proving

6.1 Introduction

6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. Convergence refers to the ability of the modelling software to arrive at a stable solution, within a specified number of iterations, for each model time step. The convergence of the 1D model was checked and is within tolerance as shown in Plate 6.1 for the baseline scenario, and Plate 6.2 for the with scheme scenario, for the 1% AEP +65%CC event. This convergence plot is typical for all events simulated. The 1D model mass error has also been checked and is within the recommended tolerance of 1% for all simulations carried out.

Plate 6.1 Model Convergence for the 1% AEP (+65%CC) event for the Baseline Scenario

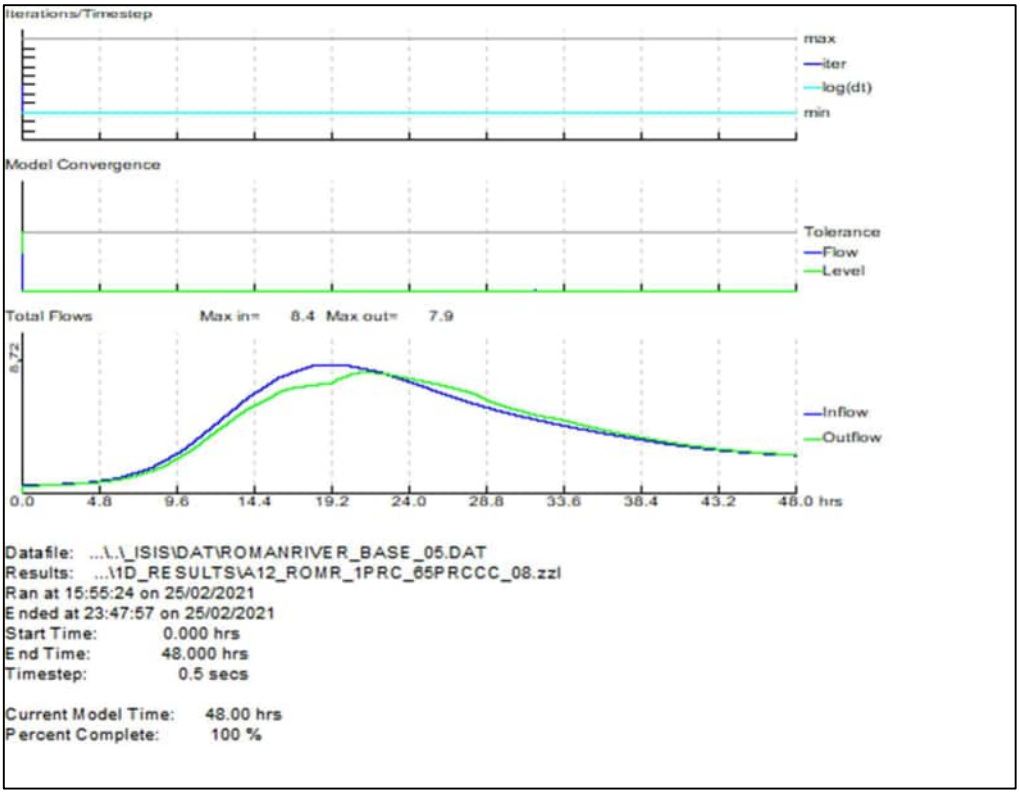
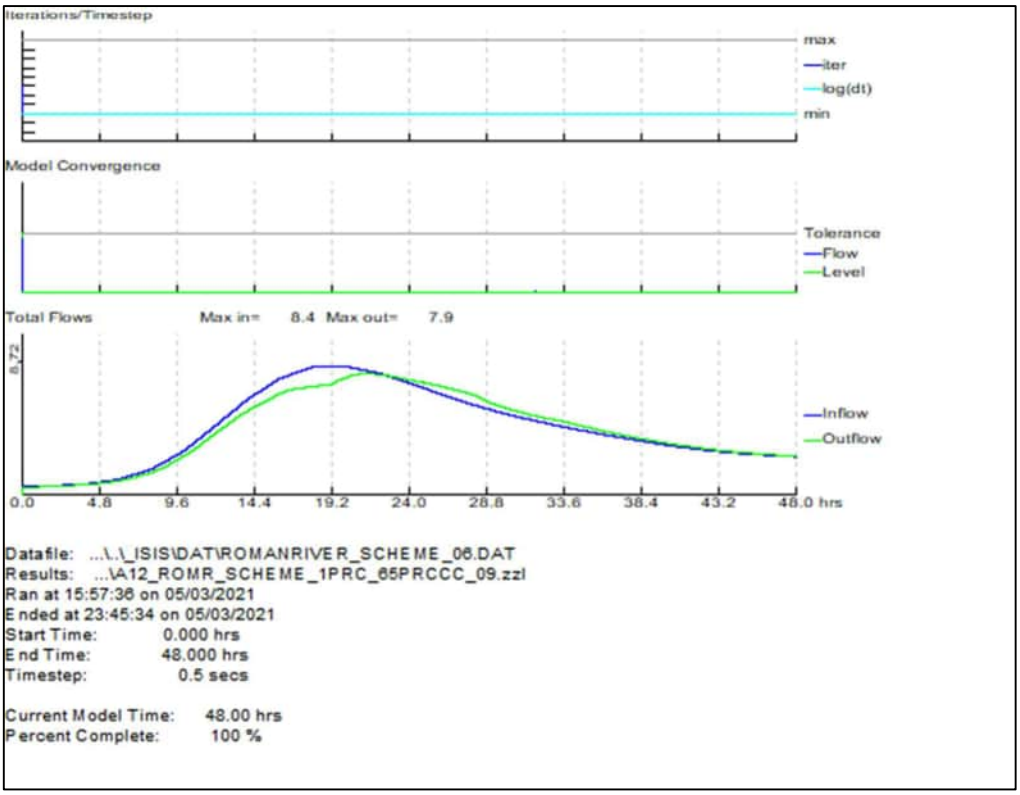


Plate 6.2 Model Convergence for the 1% AEP (+65%CC) event for the With Scheme Scenario



6.2.2 The cumulative mass error diagnostics output from the TUFLOW 2D model have also been checked for each model simulation. The accepted tolerance range recommended by the software manual is +/- 1% mass balance error. Plate 6.3 and Plate 6.4 show that for the 1% AEP (+65%CC) flood event, for the baseline and with scheme scenarios respectively, the cumulative mass error tolerance is exceeded somewhat at the onset of the floodplain wetting but stabilises to less than the 1% error very early during the simulation, before there is any significant volume of water in the floodplain. Plate 6.3 and Plate 6.4 show the change in volume (dVol) in the 2D domain during the model simulation. It has a smooth profile, which is another indicator of good numerical model performance.

Plate 6.3 2D Model Performance for the 1% AEP +65%CC event for the Baseline Scenario

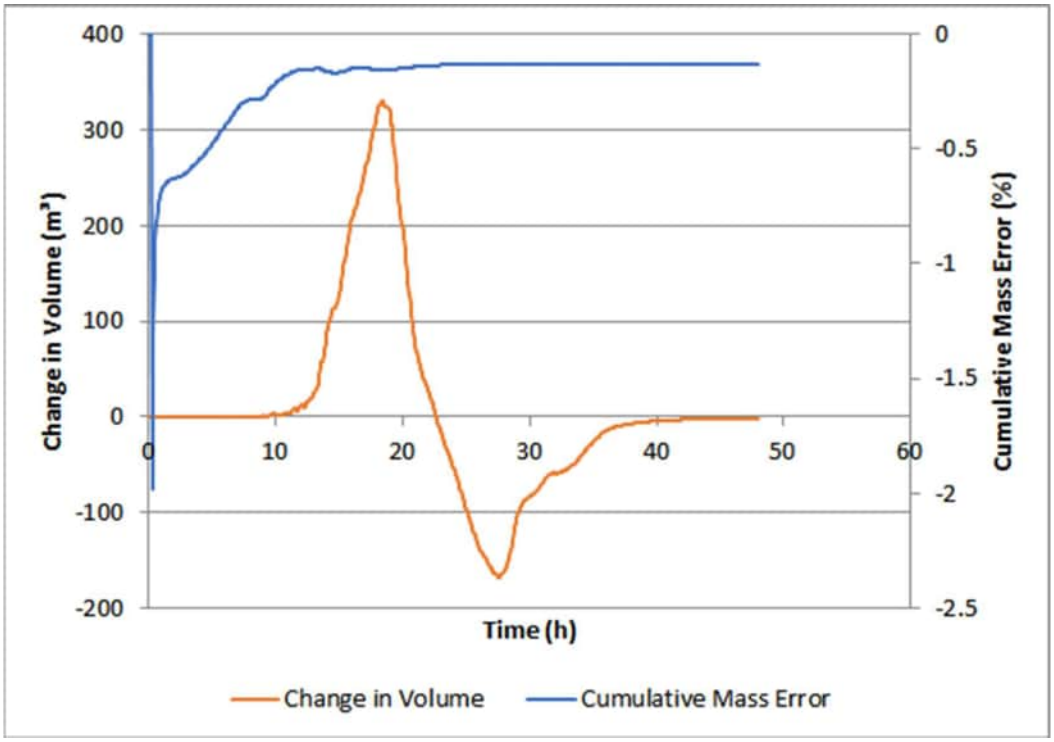
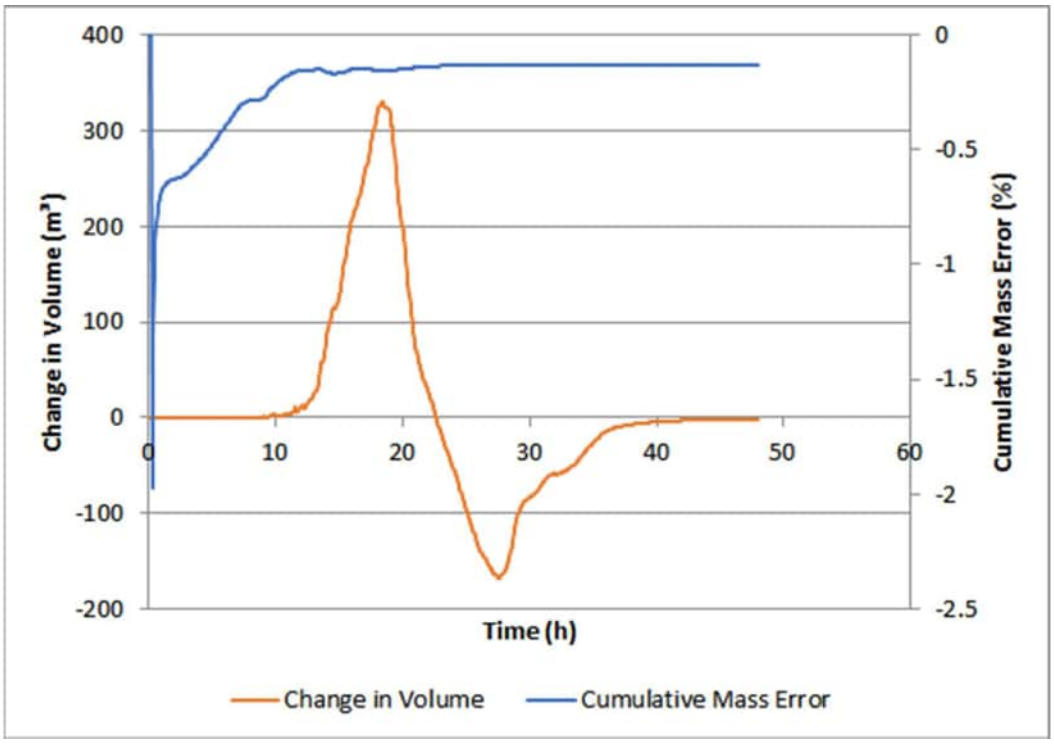


Plate 6.4 2D Model Performance for the 1% AEP +65%CC event for the With Scheme Scenario



6.3 Calibration and verification

Flow reconciliation

6.3.1 The modelled flows at the downstream end of the model domain were checked against the overall catchment hydrology analysis. To ensure a good match with the design hydrology a flow multiplier of 0.78 was the applied for the 5% and 1%AEP events. The climate change simulations required a multiplier of 0.82.

Calibration

6.3.2 There are no gauges on the Roman River within the extent of the model. Therefore, it was not possible to calibrate the model. However, some data is available for a high-level verification of the model results.

Verification using historic data

6.3.3 The Environment Agency Publish historic flood maps for UK watercourses show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. However, a review of this data showed that there is no historic flood data for Roman River.

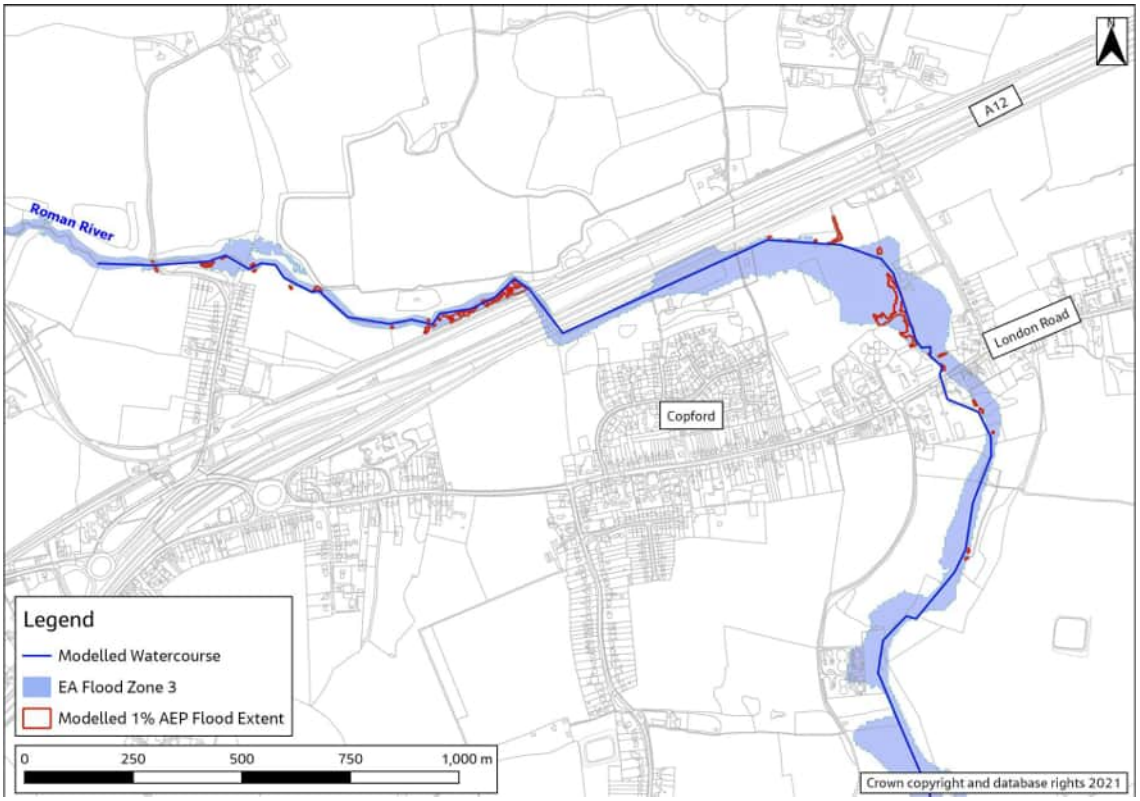
Verification using EA flood maps

6.3.4 Plate 6.5 shows the modelled 1% AEP baseline scenario flood extent and the Environment Agency published flood zone 3. The comparison shows that the Environment Agency flood map has a much larger extent than the model results show. This is expected as the Environment Agency flood map is generated based on the high-level J-Flow modelling system which does not explicitly account for the in-channel capacity of the watercourse.

Verification Conclusion

6.3.5 In conclusion, there is limited data available for verification of the Roman River model. The Environment Agency flood map shows a larger flood extent than the model (Plate 6.5). This is due to the present study being a more detailed assessment of flooding than that for the Environment Agency flood map.

Plate 6.5 Modelled 1% AEP Event Flood Extent vs. Environment Agency Flood Zone 3



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness, hydrological inflows and downstream boundary conditions. All sensitivity tests were performed using the 1% AEP event.

Roughness Sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Table 6.1 shows the impact of changing the model roughness on predicted peak water levels. The results show that the in-channel water levels are not very sensitive to changes in roughness. Decreased roughness caused a general decrease of flood extent down to just small areas of ponding in local land depressions. Increased roughness by 20% caused some additional flooding upstream of the railway bridge and London Road.

Table 6.1 Roughness sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	0.27	0.00	0.09
-20% Roughness	-0.14	0.00	-0.07

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Table 6.2 shows the impact of changing the model inflows on predicted peak water levels. The results show that the in-channel water levels are moderately sensitive to changes in flow. Increased flow by 20% caused significant increase in flood extent at two locations, first just upstream of the railway bridge, and second upstream of London Road. The flood extent for the simulation with flow decreased by 20% is very small and is found only in local land depressions.

Table 6.2 Flow sensitivity results

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	0.30	0.06	0.11
-20% Flow	-0.33	-0.07	-0.13

Downstream boundary condition sensitivity

6.4.4 The slope in the normal depth downstream boundary was adjusted by +20% and -20%. Table 6.3 shows the response at the downstream end of the model (Flood Modeller node ROMR_0000). Increased slope at downstream boundary did not cause any changes in the flood extent. Decreased slope by 20% caused additional very small (approximately 60m^2) flooding local to the downstream boundary. The results indicate that the model downstream boundary is appropriately removed from the area of interest at the A12 crossing.

Table 6.3 Downstream boundary sensitivity results

Sensitivity	Water Level Difference at ROMR_0000 (m)	Tailwater distance (m)	Distance to proposed scheme (m)
+20% Downstream boundary slope	-0.09	170	1600
-20% Downstream boundary slope	0.14	220	1600

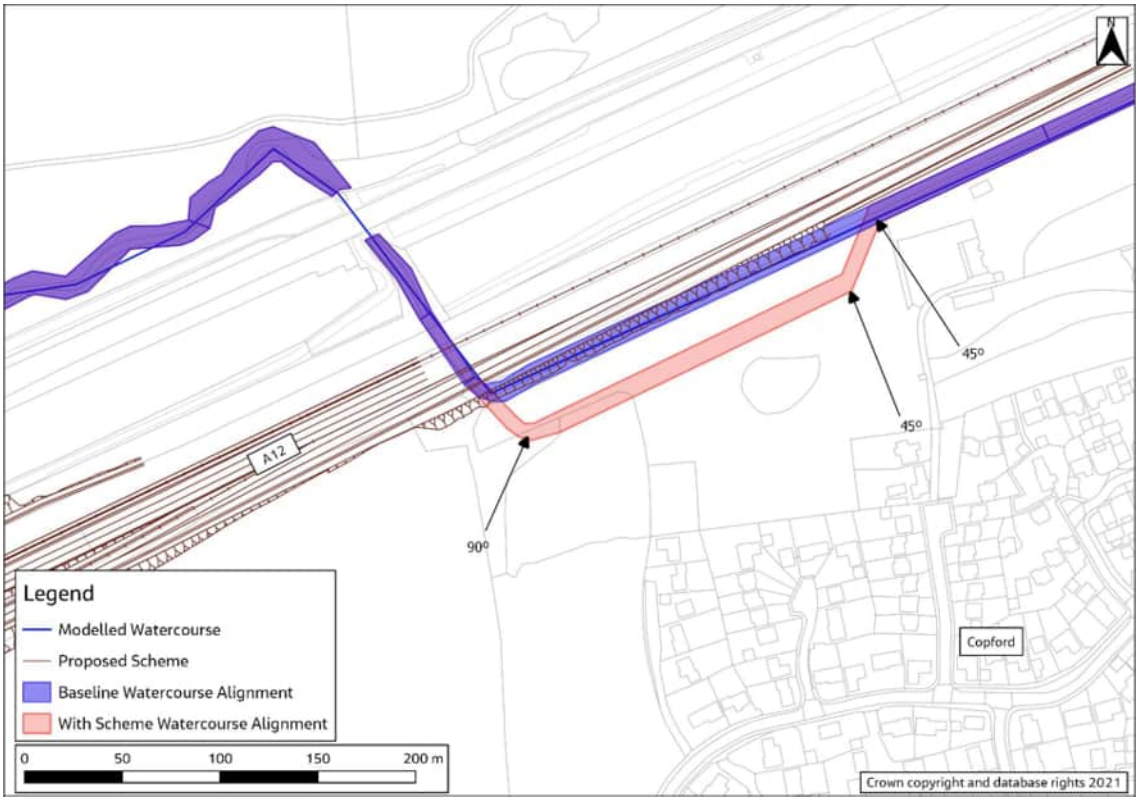
7 With Scheme model build

7.1.1 The baseline model was updated in order to include representation of the proposed scheme. Changes to the model were as follows:

- In the 1D domain, the culvert under the A12 road was extended by 20m.
- The channel was offset to the south east by 30m as shown in Plate 7.1.
- Bend losses were applied to simulate one 90º and two 45º bends in the new channel alignment.
- In the 2D domain, the original channel's DTM was filled with a polygon at the location of the offset in order to ensure correct representation, particularly for the ground levels of the 1D-2D boundary cells.

Note: Representation of the scheme embankment was not required to be included in the 2D domain as there is no flooding within the model at this location.

Plate 7.1 Realignment of the Roman River to accomodate the scheme



8 Model results

8.1 Baseline results

8.1.1 Plate 8.1 shows the maximum flood extents for all simulated events for the baseline scenario. Tabulated 1D model results are presented in Annex C. The results show that in the baseline scenario, there is no flood risk to the A12.

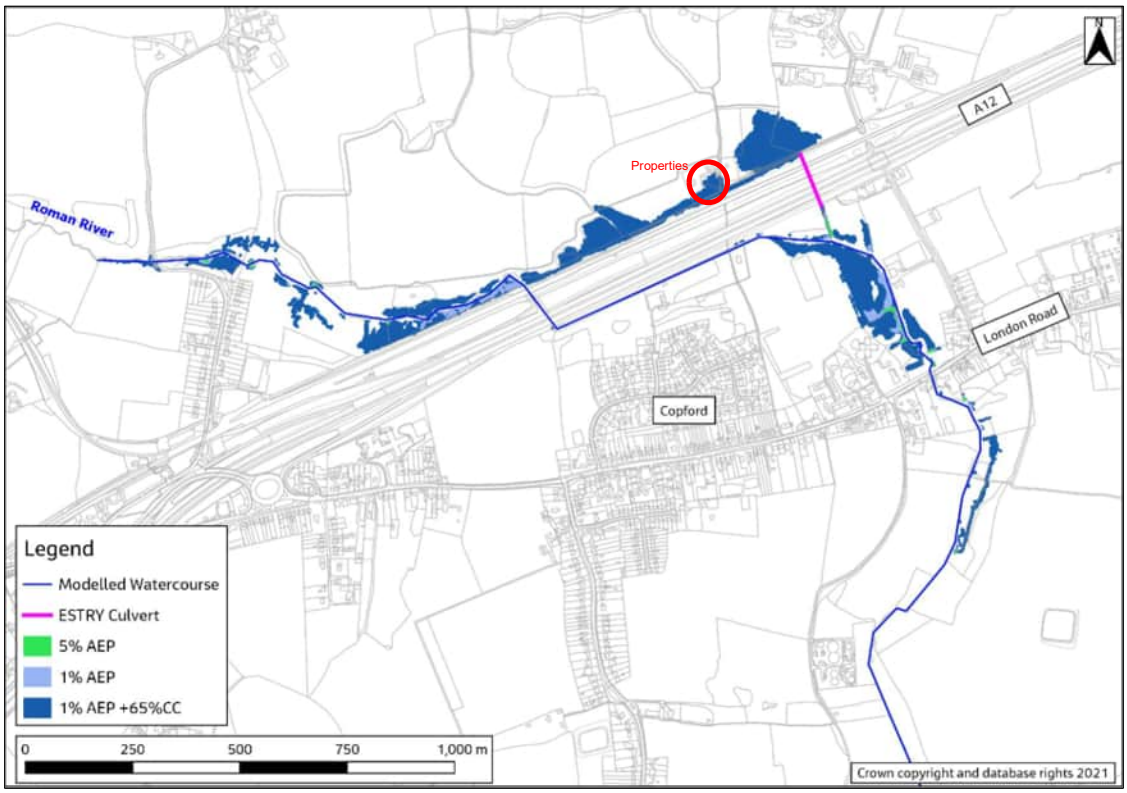
8.1.2 For the 5% AEP event no flooding occurs.

8.1.3 During the 1% AEP event, most of the modelled channel reach has the capacity to convey flood waters. Minor flooding occurs directly upstream of both the A12 and London Road. However, the flood extent is limited to the immediate floodplain and flood water fills only terrain depressions local to the watercourse, with no properties affected.

8.1.4 During the 1% AEP +65%CC event, flooding is more significant with flood mechanisms as follows:

- In the upstream part of the model several separated patches of flooding occur.
- Due to limited capacity of the railway culvert, the river backs up and spreads north east along the upstream toe of the railway embankment, as far as the drainage culvert located 700m from the Roman River crossing. There is also out of channel ponding along the stretch of the river which runs adjacent to the upstream rail embankment.
- Water passing through the drainage culvert under the A12, returns to the main river channel.
- There is no flooding alongside the downstream A12 embankment since both the river channel and the road bridge have sufficient capacity.
- A wider flood extent occurs from around 600m upstream of the B1408, London Road (ROMR_0508BrU), where the channel switches from engineered to natural form.
- Further small patches of flooding occur near the downstream extent of the model.
- Five buildings within the area upstream of London Road are flooded during this event (see Plate 8.1).

Plate 8.1 Flood extents for all events for the baseline scenario



8.2 With Scheme model results

8.2.1 Table 8.1 shows the peak water levels at key locations along the modelled reach. It shows that in-channel water levels for the largest event simulated are not affected by the presence of the scheme. There is no flood risk to the existing and proposed A12 road arrangements and the culvert does not surcharge in any of the events modelled. The properties affected in the baseline scenario show no change in flood depth in the with scheme scenario. Summary tables for all other modelled events are shown in Annex C. Plate 8.2 is a comparison of the flood extent between the with scheme and baseline results for the 1% AEP (+65%CC) event. It shows no change in flood extent due to the scheme for the largest event simulated. The same comparison is made with the water level differences, shown in Plate 8.3, showing only negligible impact due to the scheme. The same comparison for all other events can be found in Annex D.

Table 8.1 Maximum water level for the 1% AEP (+65%CC) event

Model Node	Description	Max. water level [mAOD]		Difference (m)
		Baseline	With Scheme	
ROMR_2925	Upstream extent of modelled Roman River. 1.2km upstream of the scheme	29.88	29.88	0.00
ROMR_1713	Immediately upstream of the scheme	26.38	26.38	0.00
ROMR_1673	Immediately downstream of the scheme	26.26	26.25	-0.01
ROMR_0000	Downstream end of Roman River. 1.6km downstream of the scheme	21.63	21.63	0.00

Plate 8.2 Flood extent comparison between baseline and with scheme for the 1% AEP (+65%CC)

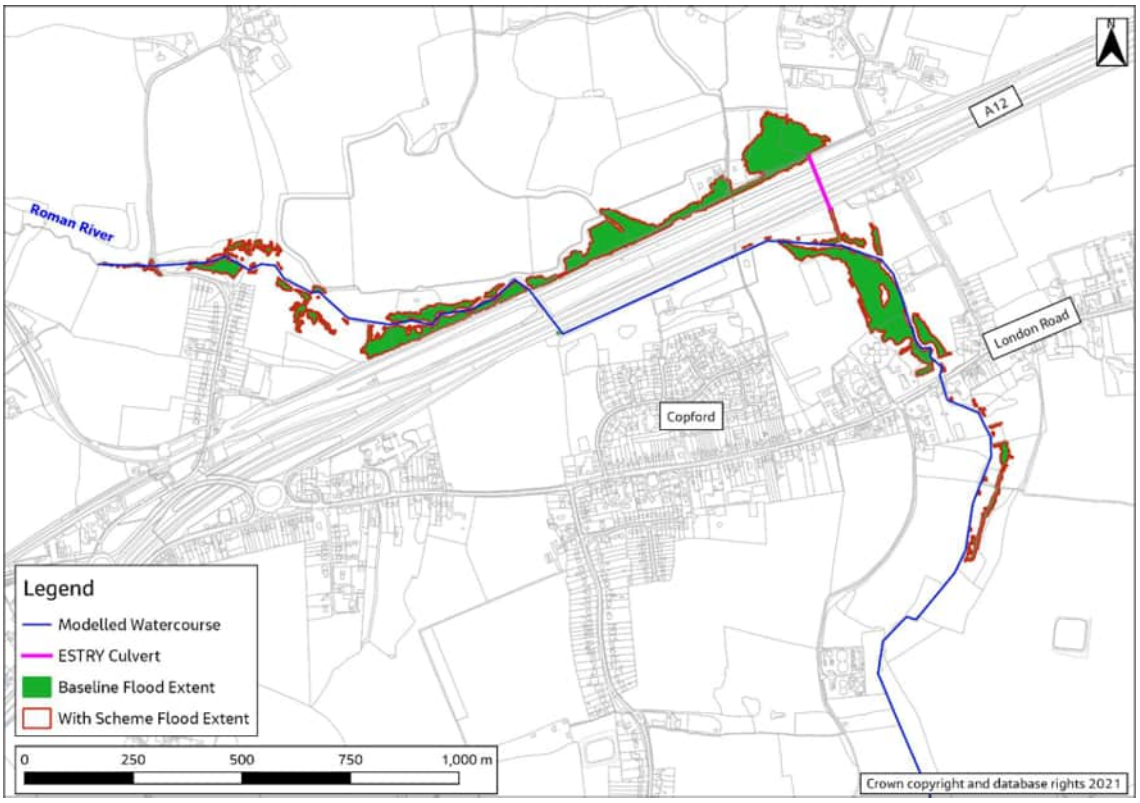
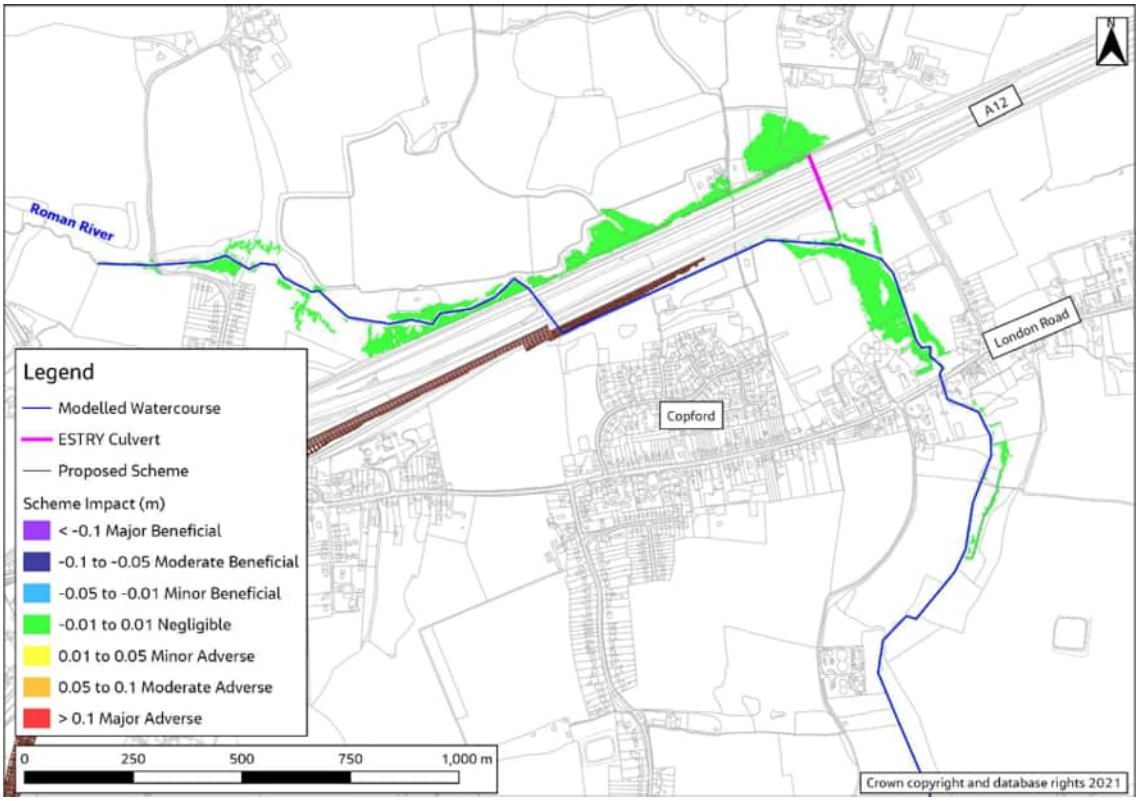


Plate 8.3 Water level difference (with scheme - baseline) for the 1% AEP (+65%CC) event



9 Model assumptions and limitations

9.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological and topographic data included in the model. Therefore, the most appropriate available information has been used to construct the model.

9.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are considered to be generally conservative for modelled water levels at the proposed scheme and are therefore appropriate for the flood risk assessment.

9.2 Hydrology

9.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- Ideally, attempts are made to improve reliability of flow estimates using recorded data. Although the Roman River is gauged at Bounstead Bridge (37017), flow data is not available on the peak flows database as it is considered to be of poor quality. These data were therefore not utilised in the assessment of flows for this study.
- The application of the same scaling factor for a given AEP at all inflow locations is considered reasonable for this study.

9.3 Hydraulic modelling

9.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for the Roman River are as follows:

- Channel roughness has been assigned using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the Flood Modeller software. The dimensions for structures have been based on detailed survey measurements where possible.
- No calibration was carried out due to poor data quality of the available gauge record.
- It is assumed that all flows from catchment ROM3 discharge to the Roman River channel directly downstream of the A12, via the drainage culvert, and that no flows from this catchment runoff to the roman river channel upstream of the A12.
- A normal depth boundary was used at the downstream end of the model, sensitivity testing was carried out to confirm that this boundary assumption is appropriate to utilise for the assessment of the proposed A12 highway scheme.

10 Conclusion

10.1.1 This report has detailed the modelling carried out to assess the baseline and with scheme flood risk for Roman River with reference to the location of the proposed A12 highway scheme.

10.1.2 Hydraulic modelling results for the baseline scenario show that for events greater than 1% AEP, areas of flood risk are seen upstream from the railway bridge due to the channel and bridge capacity being exceeded. Flood water spills towards the east along the upstream rail embankment causing ponding along the toe of the rail embankment, which inundates a small number of properties. A second location of flood risk occurs at Copford but no properties are within the model flood outline.

10.1.3 For the with scheme scenario, flood extents remain the same as in the baseline scenario for all events and differences in water level when compared with the baseline results are negligible. There is zero change in flood risk at the flooded properties to the north of the A12 embankment.

10.1.4 The A12 remains free from flooding for events up to the 1% AEP +65% CC event for both baseline and with scheme scenarios.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

References

Chow, Ven Te (1959). Open-Channel Hydraulics. McGraw-Hill.

Environment Agency (2016), Adapting to Climate Change: Advice for Flood and Coastal Risk Management Authorities.

Environment Agency (2017a). Flood Estimation Guidelines. Technical Guidance 197_08.

Environment Agency (2017b). Spatial Data Catalogue. [online] Available at: http://environment.data.gov.uk/ds/catalogue/#/catalogue [Accessed October 2017].

Environment Agency, 2020, Flood Risk Assessments: Climate Change Allowances.

Jacobs (2022). Environmental Statement - Appendix 6.6 - Flood Risk Assessment. Ref: TR10060/APP/6.6.

National River Flow Archive (2019), https://nrfa.ceh.ac.uk/peak-flow-dataset, version 8.

Annex A **Roman River Flood Estimation Report**

HE551497-JAC-EWE-SCHW-RP-LE-0082.docx

Annex B **Structure Schedule**

Table B.1 Model Structures Schedule

Structure Name	Node Label(s)	XY Coordinates	Modelling Approach
Road Bridge 7.021	ROMR_2794BrU	591626, 224385	Represented as USBPR1978. Spill unit not required. Data based on detailed survey carried out by Jacobs in 2020. FM default coefficients applied for all parameters.
Foot Bridge 7.019	Not modelled	591789, 224405	Not modelled – Small structure will not affect flow pattern.
Rail Bridge 7.012	ROMOR_1791	592489, 224320	Represented as Arch Bridge. Spill unit not required. Data based on detailed survey. FM default coefficients applied for all other parameters.
A12 Road Bridge 7.012	ROMOR_1713	592538, 224256	Represented as Symmetrical conduit. Spill unit not required. Data based on detailed survey. FM default coefficients applied for all parameters.
Road Bridge 7.004	ROMOR_0508BrU	593431, 224121	Represented as Arch Bridge. Spill unit not required. Data based on detailed survey. FM default coefficients applied for all parameters.
Culvert under A12	Estry culvert	593163, 224547	Represented as Estry circular culvert. Data based on HADDMS records. Default coefficients applied for all parameters.

Annex C **Maximum Water Level Tables and Long Sections**

Table C.1 Maximum Water Level for the 5% AEP Event

Model Node	Description	Max. water level [mAOD]		Difference (m)
		Baseline	With Scheme	
ROMR_2925	Upstream extent of modelled Roman River. 1.2km upstream of the scheme	29.38	29.38	0.00
ROMR_1713	Immediately upstream of the scheme	25.96	25.96	0.00
ROMR_1673	Immediately downstream of the scheme	25.83	25.83	0.00
ROMR_0000	Downstream end of Roman River. 1.6km downstream of the scheme	21.10	21.10	0.00

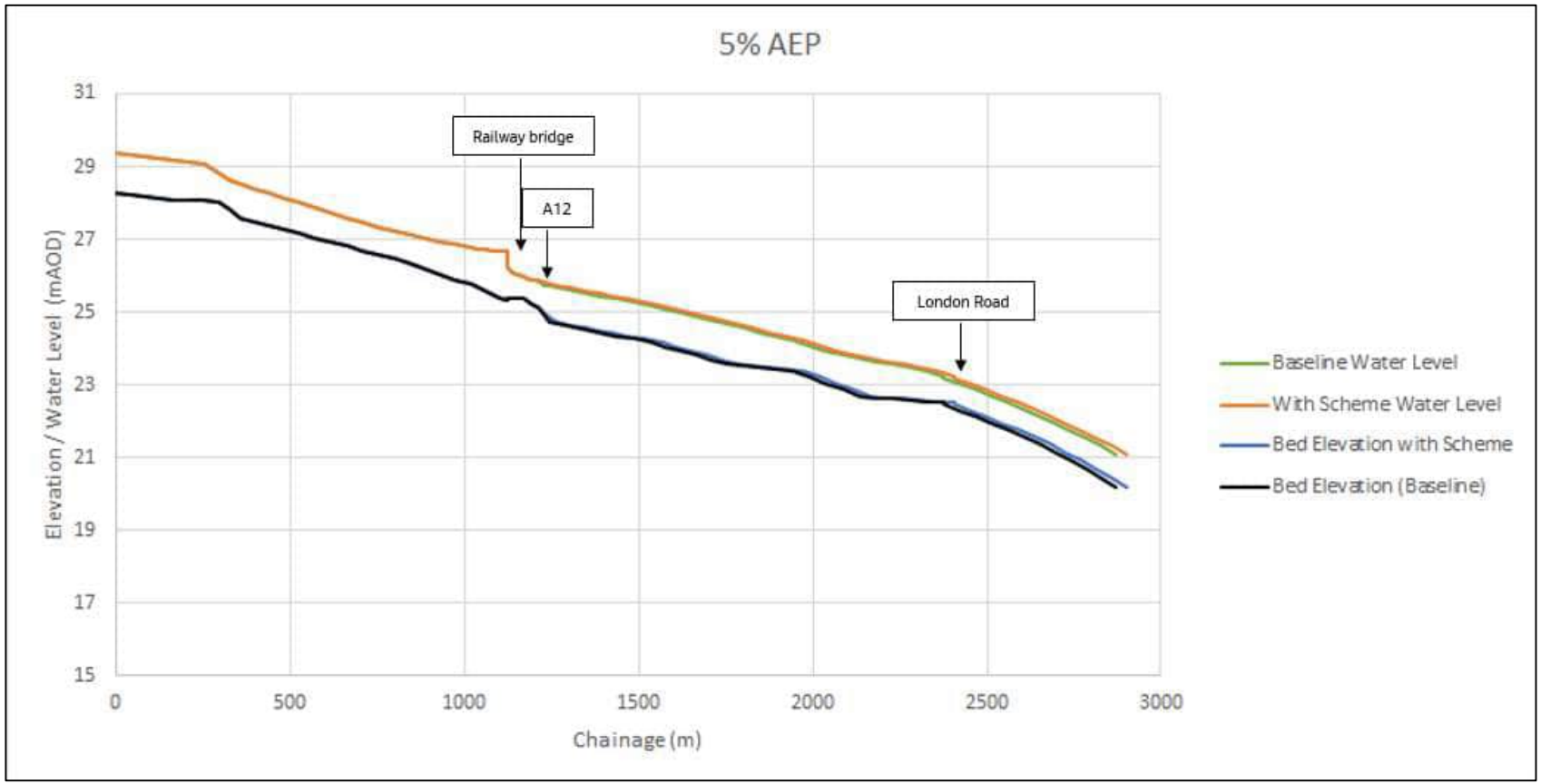
Table C.2 Maximum Water Level for the 1% AEP Event

Model Node	Description	Max. water level [mAOD]		Difference (m)
		Baseline	With Scheme	
ROMR_2925	Upstream extent of modelled Roman River. 1.2km upstream of the scheme	29.54	29.54	0.00
ROMR_1713	Immediately upstream of the scheme	26.12	26.13	0.01
ROMR_1673	Immediately downstream of the scheme	26.01	26.00	-0.01
ROMR_0000	Downstream end of Roman River. 1.6km downstream of the scheme	21.27	21.27	0.00

Table C.3 Maximum Water Level for the 1% AEP (+65%CC) Event

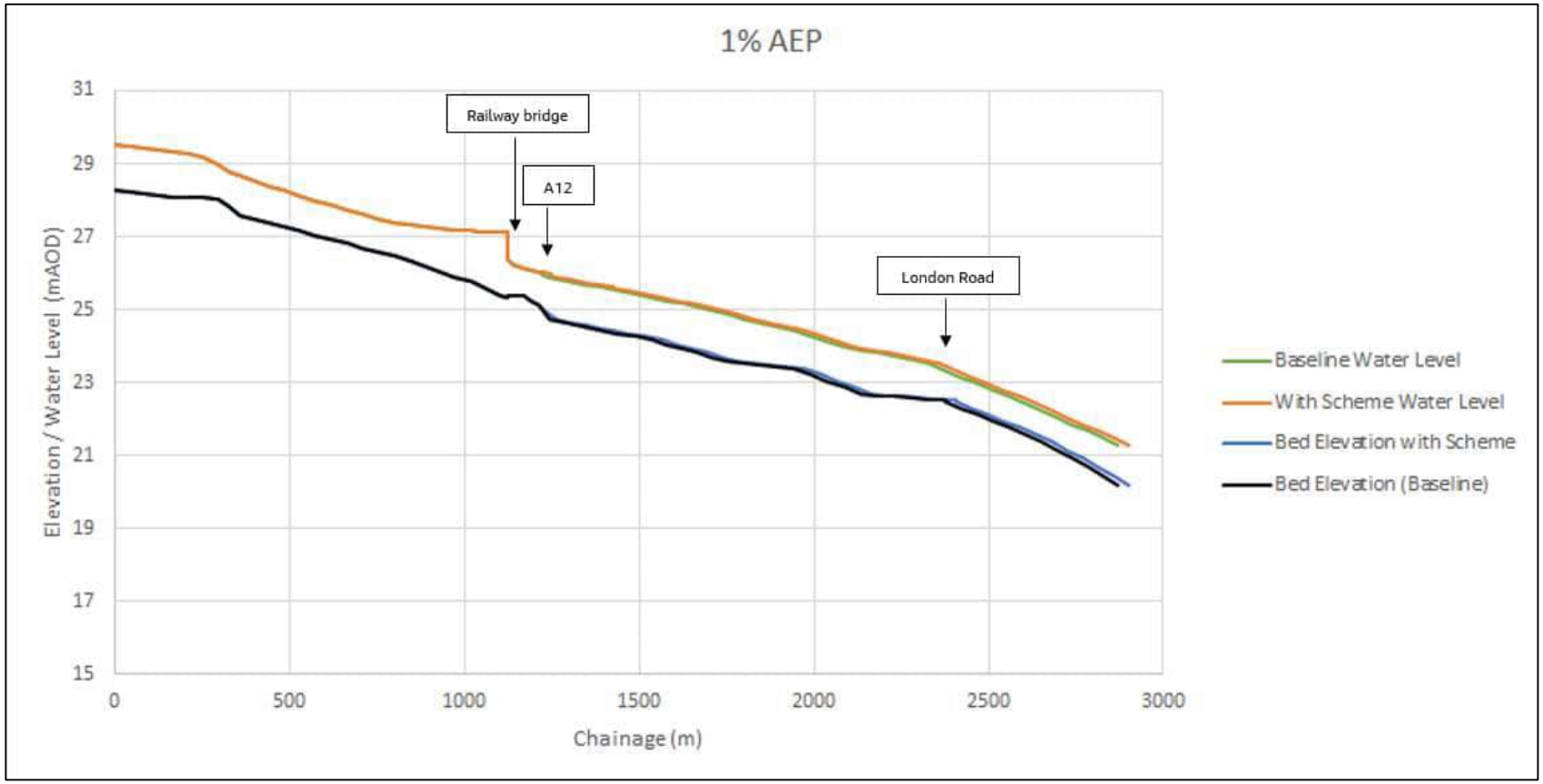
Model Node	Description	Max. water level [mAOD]		Difference (m)
		Baseline	With Scheme	
ROMR_2925	Upstream extent of modelled Roman River. 1.2km upstream of the scheme	29.84	29.84	0.00
ROMR_1713	Immediately upstream of the scheme	26.35	26.35	0.00
ROMR_1673	Immediately downstream of the scheme	26.23	26.22	-0.01
ROMR_0000	Downstream end of Roman River. 1.6km downstream of the scheme	21.60	21.60	0.00

Plate C.1 In-channel Peak Water Level Comparison between Baseline and With-Scheme for the 5% AEP event



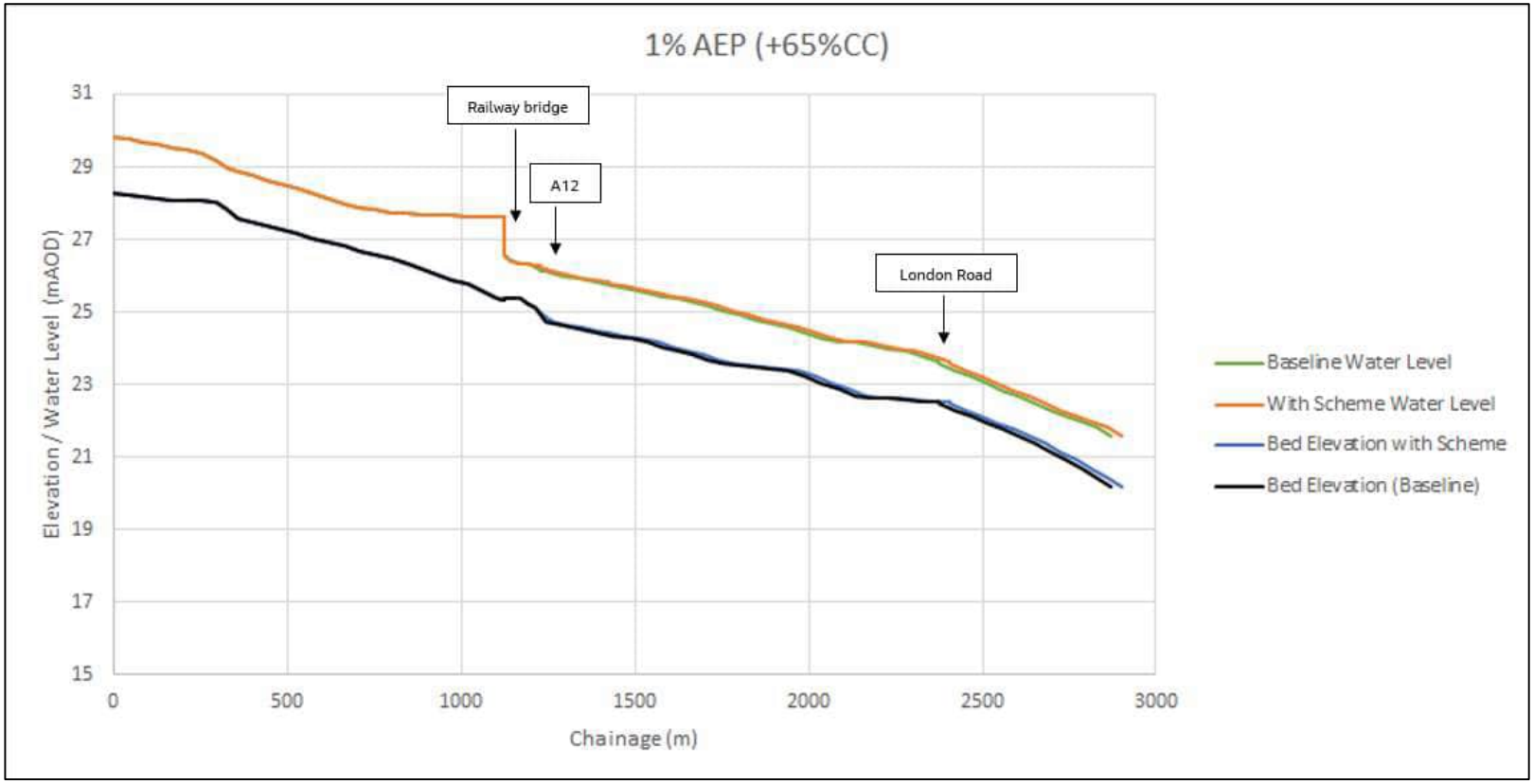
Note: The implementation of the scheme includes the re-alignment of the main channel downstream of the existing A12 highway, this adds chainage to the channel in this location, causing mis-alignment of concurrent locations at all points downstream, when both baseline and with scheme results are plotted together in long section. It should be noted that there is no difference in modelled maximum water levels within the model.

Plate C.2 In-channel Peak Water Level Comparison between Baseline and With-Scheme for the 1% AEP event



Note: The implementation of the scheme includes the re-alignment of the main channel downstream of the existing A12 highway, this adds chainage to the channel in this location, causing mis-alignment of concurrent locations at all points downstream, when both baseline and with scheme results are plotted together in long section. It should be noted that there is no difference in modelled maximum water levels within the model.

Plate C.3 In-channel Peak Water Level Comparison between Baseline and With-Scheme for the 1% AEP (+65%CC) event



Note: The implementation of the scheme includes the re-alignment of the main channel downstream of the existing A12 highway, this adds chainage to the channel in this location, causing mis-alignment of concurrent locations at all points downstream, when both baseline and with scheme results are plotted together in long section. It should be noted that there is no difference in modelled maximum water levels within the model.

Annex D **Flood Mapping**

Plate D.1 Water Level Difference (With Scheme minus Baseline) for the 5% AEP Event

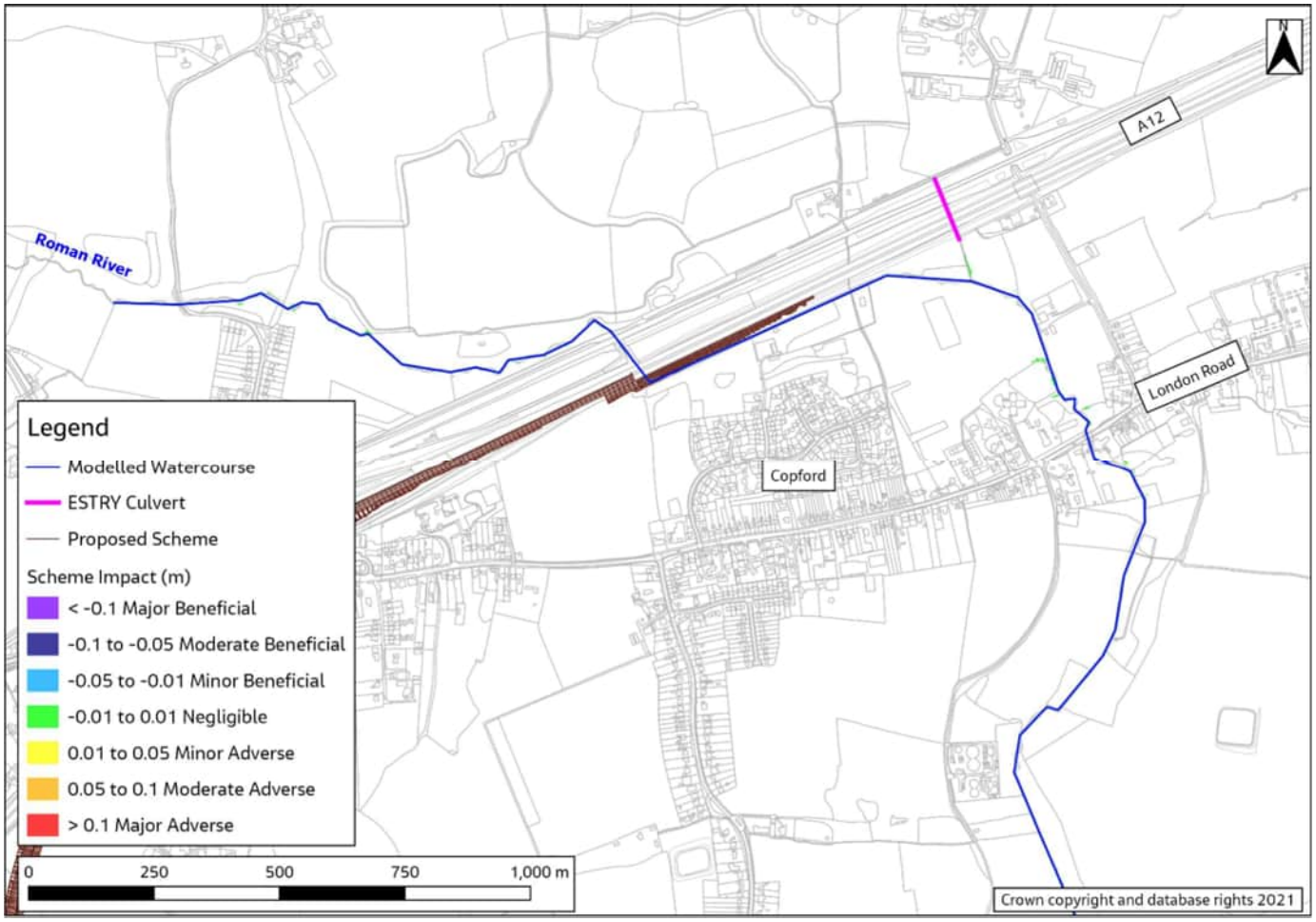


Plate D.2 Water Level Difference (With Scheme minus Baseline) for the 1% AEP Event

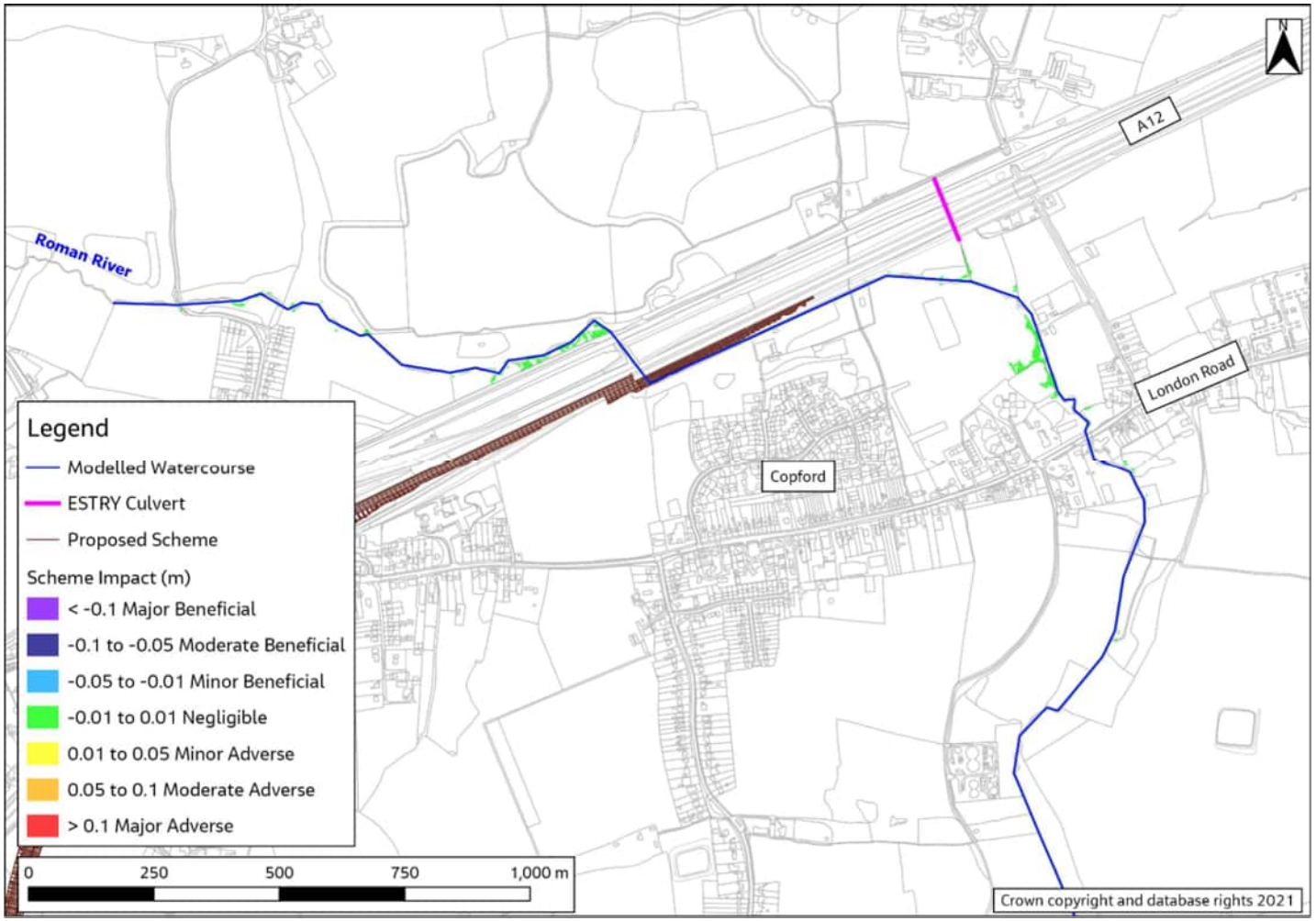


Plate D.3 Water Level Difference (With Scheme minus Baseline) for the 1% AEP + 65%CC Event

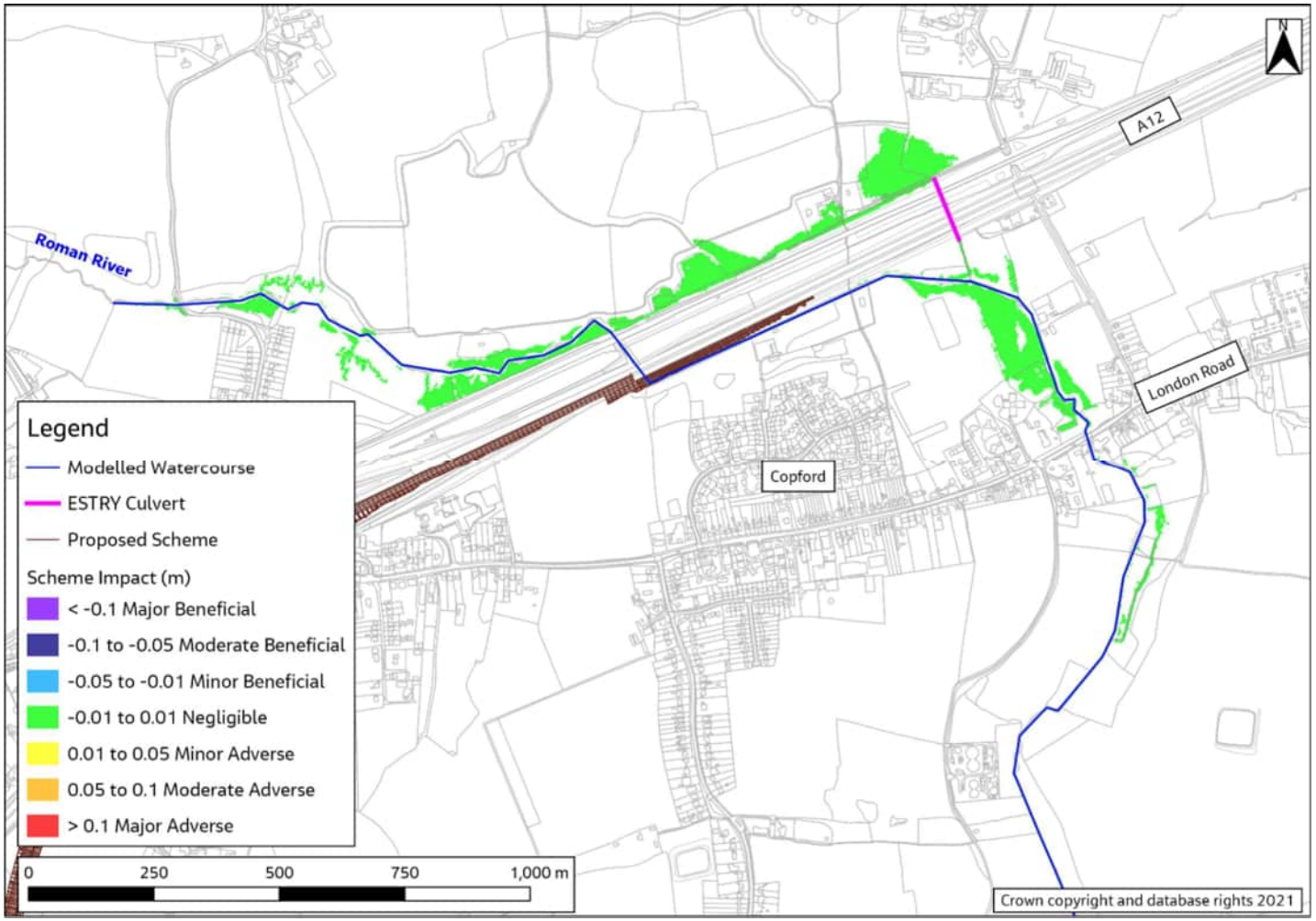
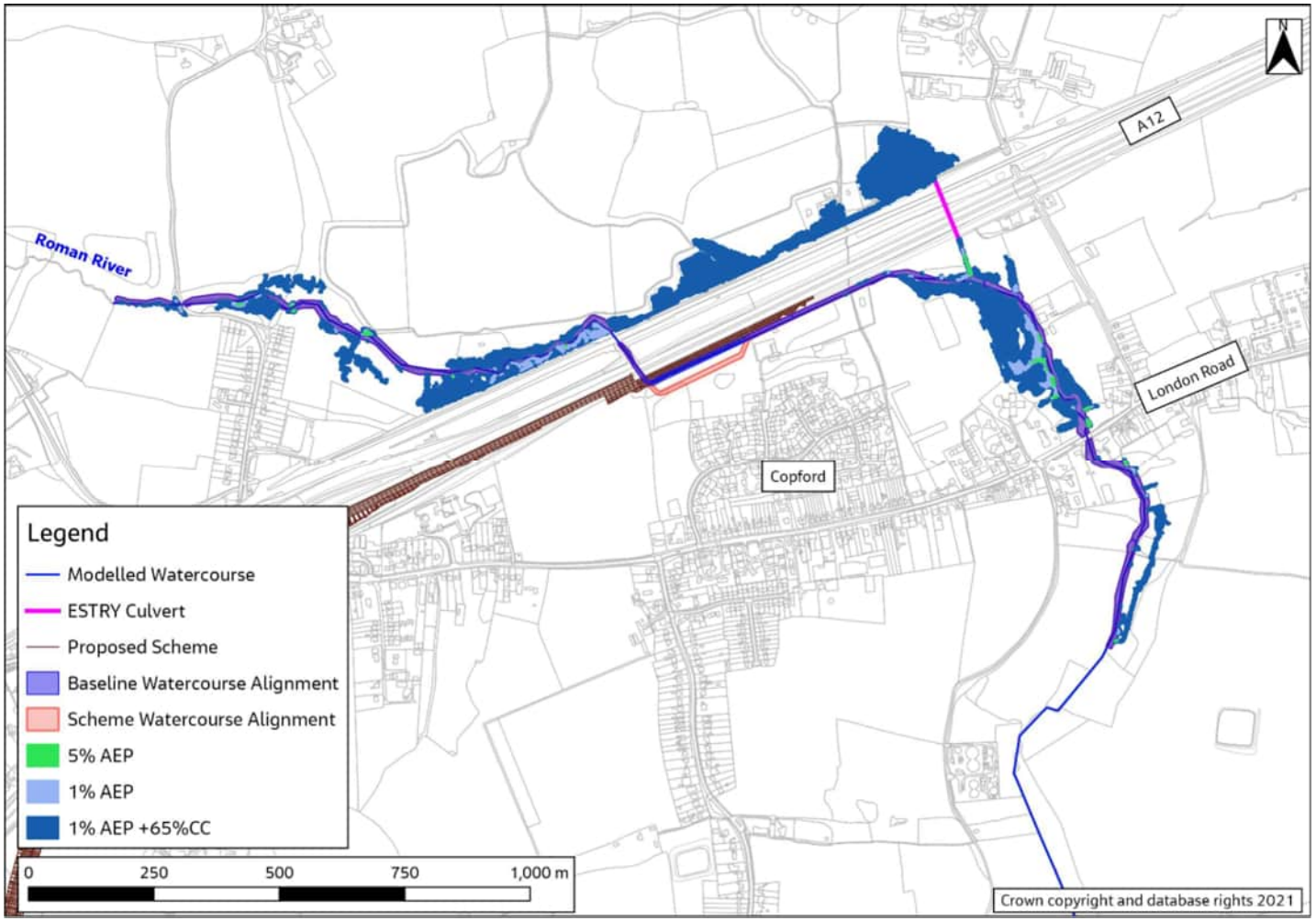


Plate D.4 Flood Extents for all Modelled Events for the With Scheme Scenario



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT ROMAN RIVER HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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ANNEXES

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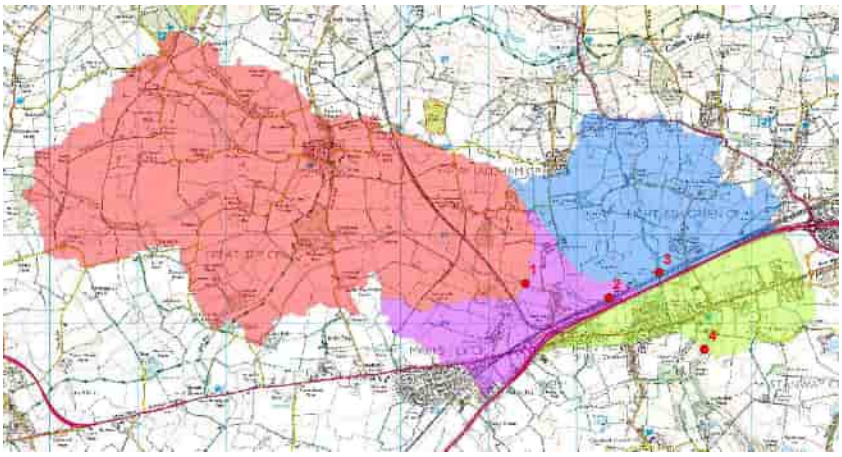
A.1 Supporting Information 19

1 Introduction

1.1.1 This document is to provide a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This record sheet is to be used where flood estimates are needed at a single location.

2 Method Statement

2.1 Overview of requirements for flood estimate

Item	Comments
Give an overview which includes: • Purpose of study • Approx. no. of flood estimates required • Peak flows or hydrographs? • Range of return periods and locations • Approx. time available	Highways England are planning Highway Improvements on the A12 from junction 19 to junction 25. A number of potential routes have been proposed. The current route of the road crosses the Roman River to the east of Marks Tey. The objective of the proposed study (hydrological assessment component) are: (i) to produce baseline and design hydrology for a river hydraulic model of Roman River. 4 locations along the watercourse have been requested by the hydraulic modellers for inputs into the hydraulic model. These reflect where there are river confluences or locations of crossings. A full hydrograph is required for each of the events 50%, 20%, 10%, 5%, 3.33%, 2%, 1.33%, 1%, 0.5% and 0.1% (equivalent to the 2, 5, 10, 20, 30, 50, 75, 100, 200 and 1000-year design return periods). Climate change allowances – The climate change allowance for the River Blackwater Operational Catchment Area has been taken from the unpublished UKCP18 peak river flow climate change allowances. The upper climate change allowance for the River Blackwater Operational River Basin is 65% to the 2080s. 

2.2 Overview of the catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The catchment of the Roman River covers an area of 19.7km^2 to the downstream model extent to the west of Colchester. It lies on a bedrock of London Clay Formation - Clay, Silt and Sand and has soils that are slightly acid loamy and clayey soils with impeded drainage. The baseflow index is 0.545 with a standard percentage runoff of 42.1. The catchment is essentially rural and receives a low average annual rainfall of 563mm.

2.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – Version 8, September 2019

2.4 Gauging stations (flow or level)

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km^2)	Type (rated / ultrasonic / level…)	Start and end of flow record
Roman River	Bounstead Bridge		37021	TL984205	52.6	flat v crump weir	01/1965 - 08/1991
Blackwater	Stisted		37017	TL792242	139.2	flat v crump weir	01/1969 - present
Blackwater	Appleford Bridge		37010	TL844158	247.3	trapezoidal flume	01/1962 - present
River Ter	Crabbs Bridge		37003	TL786107	77.8	trapezoidal flume	01/1932 - present

2.5 Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	Other comments on station and flow data quality – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Bounstead Bridge	N/A					
Stisted	1969 - 2017	N	Y	N		AMAX data corrected for non-modular flows
Appleford Bridge	1962 - 2017	N	Y	Y		Irregular record. Exceptionally high flow in 2001 considered suspect
Crabbs Bridge	1963 - 2017	Y	Y	Y	N	AMAX data corrected for non-modular flows

2.6 Rating equations

Station name	**Type of rating** e.g. theoretical, empirical; degree of extrapolation	**Rating review needed?**	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Bounstead Bridge	N/A	N	
Stisted	Velocity area at high flows	N	
Appleford Bridge	High flow rated spillway	N	Rating accounts for mon-modular flow
Crabbs Bridge	Theoretically rated	N	
Give link/reference to any rating reviews carried out		N/A	

2.7 Other data available and how it has been obtained

Type of data	**Data relevant to this study?**	**Data available ?**	**Source of data and licence reference if from EA**	**Date obtained**	**Details**
Check flow gaugings (if planned to review ratings)					
Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies					
Other data or information (e.g. groundwater, tides)					

2.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes – there is nothing unusual about the catchment that would mean that the methods were not acceptable
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…)	Locations have been selected at tributary inflows and road crossings

• Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	None
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	Both the statistical method and ReFH2 have been selected and the results compared, The catchment has been broken down into lateral catchments and an inflow catchment for a left bank tributary. An additional flow estimate has been calculated for the downstream end to allow flow consolidation.
Software to be used (with version numbers)	Peak flow data set v8 September 2019 WINFAP-FEH v4[1] / ReFH 2.3

[1] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

3 Locations where flood estimates required

3.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
ROM1	Roman River	Upstream model extent	591450	224400	11.78	11.83
ROM2	Roman River	Lateral catchment upstream of left bank tributaries	592450	224350	14.18	1.96*
ROM3	Roman River	Left bank tributaries (3)	592969	224568	1.91	3.71
ROM4	Roman River	Lateral catchment downstream of A12 to downstream model extent	593450	223700	20.3	2.21*
TOT	Roman River	Total river catchment to the downstream model extent	593450	223700	20.3	19.7
Reasons for choosing above locations						

**Residual catchments*

3.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
ROM1	0.989	0.23	0.453	3.93	18.00	564	44.29	0.006	0.102
ROM2	1.000	0.23	0.673	2.07	17.22	565	50.70	0.031	-
ROM3	1.000	0.23	0.710	1.40	14.2	561	35.80	0.003	-
ROM4	1.000	0.23	0.471	1.54	12.86	545	42.07	0.165	-
TOT	0.994	0.23	0.545	5.18	16.9	563	42.10	0.025	0.113

3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Additional catchment area has been measured from Ordnance Survey 1:25,000 maps. Lateral catchments have been calculated based on FEH catchments to the study location and the next one upstream.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	Reference was made to the Geology of Britain Viewer [redacted]
Source of URBEXT	FEH web viewer
Method for updating of URBEXT	Calculation of UAF is based on the new UAF model implemented in WINFAPv4 and described in WINFAP 4 Urban Adjustment Procedures Issue 1.01

4 Statistical method

4.1 Search for donor sites for QMED (if applicable)

Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	The Roman River is gauged at Bounstead Bridge (37017). Flow data is not available on the peak flows database due to poor quality. The River Blackwater is located to the west of the Roman River and is gauged both upstream (Stisted 37017) and downstream (Appleford Bridge 37010) of the confluence. Both of these catchments are larger than the study catchment at 139 and 247km^2 respectively. The upper Blackwater catchment is chalk. The River Ter is further away from the study catchment and is gauged at Crabbs Bridge (37003). The catchment area is more similar to the study catchment at 77.8km^2. The geology is the same as the study catchment being underlined by London Clay with an SPRHOST 41.76.

4.2 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)
37017	Rejected due to artificial transfer from Ely Ouse scheme, predominantly chalk in the upper catchment.	Not applicable				
37010	Rejected - groundwater-influenced catchment with major artificial influences.					
37017	No flow data					
37003	Accepted – nearby rural catchment with data suitable for QMED calculation.	AM	N	5.430	6.509	0.83
Which version of the urban adjustment was used for QMED at donor sites, and why? Note: The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8).						

4.3 Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average adjustment factor	
ROM1	DT	1.26	37003	16.9	0.328	0.942			1.19
ROM3	DT	0.19	37003	20.8	0.304	0.946			0.18
TOT Total river catchment to DS model boundary	DT	1.53	37003	18.3	0.319	0.944			1.44

Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?	
Which version of the urban adjustment was used for QMED, and why?	Calculation of UAF is based on the new UAF model implemented in WINFAPv4 and described in WINFAP 4 Urban Adjustment Procedures Issue 1.01

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added.

When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050[Error! Bookmark not defined.] should be used. If the original FEH equation has been used, say so and give the reason why.

The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.

The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

4.4 Derivation of pooling groups

4.4.1 The composition of the pooling groups is given in the Annex. Several subject sites may use the same pooling group.

Name of group	**Site code from whose descriptors group was derived**	**Subject site treated as gauged?** (enhanced single site analysis)	**Changes made to default pooling group, with reasons** Note also any sites that were investigated but retained in the group.	**Weighted average L-moments**, L-CV and L-skew, (before urban adjustment)
Roman River	TOT	N	7011, 26013 – Removed as short record 33054 – Removed as FARL < 0.95 26802, 26014 33032, removed as BFIHOST>0.65 (SPRHOST < 20%); 49005, 53017, 28058, 27010, 37016 added to increase stations years to 500.	L-CV = 0.275 L-SKEW = 0.164
Notes Pooling groups were derived using the revised procedures from Science Report SC050050 (2008). The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.				

4.5 Derivation of flood growth curves at subject sites

Site code	**Method** (SS, P, ESS, J)	**If P, ESS or J, name of pooling group (**Error! Reference source not found.**)**	**Distribution used and reason for choice**	**Note any urban adjustment or permeable adjustment**	**Parameters of distribution** (location, scale and shape) after adjustments	**Growth factor for 100-year return period**
TOT	P	Roman River at DS extent	GL – best fit to countrywide data	Urban adjustment made to growth curve	Location = 1.000 Scale = 0.284 Shape = -0.164	2.950
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters. Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010). Growth curves were derived using the revised procedures from Science Report SC050050 (2008).						

4.6 Flood estimates from the statistical method

Site code	**Flood peak (m^3/s) for the following return periods (in years)**									
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**200**	**1000***
ROM1	1.19	1.71	2.08	2.46	2.70	3.02	3.30	3.50	4.04	6.90
ROM3	0.18	0.26	0.32	0.37	0.41	0.46	0.50	0.53	0.61	1.05
TOT	1.44	2.07	2.51	2.98	3.27	3.66	3.99	4.24	4.89	8.35
1,000 year flows derived by applying the ratio of the 1000:100 from the ReFH method. Ratio = 1.97										

5 Revitalised flood hydrograph (ReFH) method

5.1 Parameters for ReFH model

Site code	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
ROM1	CD	8.933	376.591	49.489	2.016
ROM2	CD	6.278	601.079	53.041	2.072
ROM3	CD	5.336	686.224	51.107	2.234
ROM4	CD	5.815	589.525	41.067	2.297
TOT	CD	10.673	464.780	58.270	2.684
Brief description of any flood event analysis carried out (further details should be given below or in a project report)		N/A			

5.2 Design events for ReFH method

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
ROM1	Rural	Winter	13.0	
ROM2	Rural	Winter	9.0	
ROM3	Rural	Winter	9.0	
ROM4	Rural	Winter	9.0	
TOT	Rural	Winter	18.0	
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?				

5.3 Flood estimates from the ReFH method

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	30	50	75	100	100 +65% CC	200	1000
ROM1	1.74	2.29	2.67	3.04	3.28	3.62	3.95	4.22	6.96	5.14	8.31
ROM2	0.17	0.24	0.28	0.32	0.35	0.39	0.43	0.46	0.76	0.57	0.98
ROM3	0.28	0.38	0.45	0.52	0.56	0.63	0.69	0.75	1.24	0.94	1.65
ROM4	0.45	0.60	0.71	0.81	0.88	0.97	1.06	1.13	1.86	1.38	2.27
TOT	1.99	2.61	3.03	3.46	3.73	4.10	4.47	4.78	7.89	5.88	9.59

6 Discussion and summary of results

6.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	ReFH	Statistical	Ratio	ReFH	Statistical	Ratio
ROM1	1.74	1.19	1.46	4.22	3.50	1.21
ROM3	0.28	0.18	1.56	0.75	0.53	1.42
TOT	1.99	1.44	1.38	4.78	4.24	1.13

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The statistical method is not considered appropriate for use on lateral catchment inflows. The ReFH2 has therefore been used to provide a consistent approach to flow estimation across the Roman River catchment. Comparison of flow estimates by the 2 approaches show greater flows using the ReFH2 methods across all catchments.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The catchment areas have been more accurately determined using the OS mapping.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	Larger return periods become less reliable due to the level of extrapolation required. Recorded river flow records used in the development of the estimation methods are generally much shorter than the return period of interest.
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH **3** 12.5 or the factorial standard error from Science Report SC050050 (2008).	Confidence limits for statistical estimates of QMED at ROM4 for the total catchment area are 68% lower 1.03, upper 2.12 95% lower 0.72, upper 3.03
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	
Give any other comments on the study, for example suggestions for additional work.	

6.4 Checks

Are the results consistent, for example at confluences?	The comparison of 100 year flows at the downstream end of the model TOT ReFH2 Direct calculation – 4.78 m^3/s Sum of inflows – 6.42 m^3/s
What do the results imply regarding the return periods of floods during the period of record?	No recorded flows to compare
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	100 year growth factor for the whole catchment to the downstream model extent is 2.95 from the statistical method 2.40 from ReFH2 This is within the guidance
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	Ratio of 1,000 to 100 year flow at the downstream of the model at the Blackwater confluence is 9.59/4.78 = 2.01
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	QMED specific runoffs are 1.01 from ReFH2 0.73 from statistical
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No earlier studies have been provided
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	

6.5 Final results

Target Flows

6.5.1 Below are the target flows for the downstream end of the model using the critical catchment duration of 18 hours.

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+CC (100yr +cc)	0.5% (200-yr)	0.1% (1,000-yr)
TOT	1.99	2.61	3.03	3.46	3.73	4.10	4.47	4.78	7.89	5.88	9.59

Model inflows

6.5.2 The ReFH2 model was run for each catchment with a critical catchment duration of 18 hours. The results below show the resultant peak flows for each sub-catchment

Site code	Flood peak (m^3/s) for the following events										
	50% (2-yr)	20% (5-yr)	10% (10-yr)	5% (20-yr)	3.3% (30-yr)	2% (50-yr)	1.33% (75-yr)	1% (100-yr)	1%+CC (100yr+ cc)	0.5% (200-yr)	0.1% (1,000-yr)
ROM1	1.75	2.30	2.68	3.05	3.28	3.62	3.94	4.22	6.96	5.14	8.25
ROM2	0.17	0.23	0.27	0.31	0.34	0.38	0.42	0.45	0.74	0.56	0.95
ROM3	0.27	0.36	0.43	0.49	0.54	0.60	0.66	0.71	1.17	0.90	1.56
ROM4	0.44	0.58	0.67	0.77	0.83	0.91	1.00	1.07	1.77	1.31	2.12

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	Yes, the inflow hydrographs were needed in the four sub-catchments. There were derived using the ReFH2 method. While deriving the model inflow, same storm duration is applied to all sub-catchments. For example, for the target flow at d/s modelling extent, a storm duration of 18hrs was used for all sub-catchments.

Acronyms

Abbreviation	Term
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1989
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

Annex A

A.1 Supporting Information

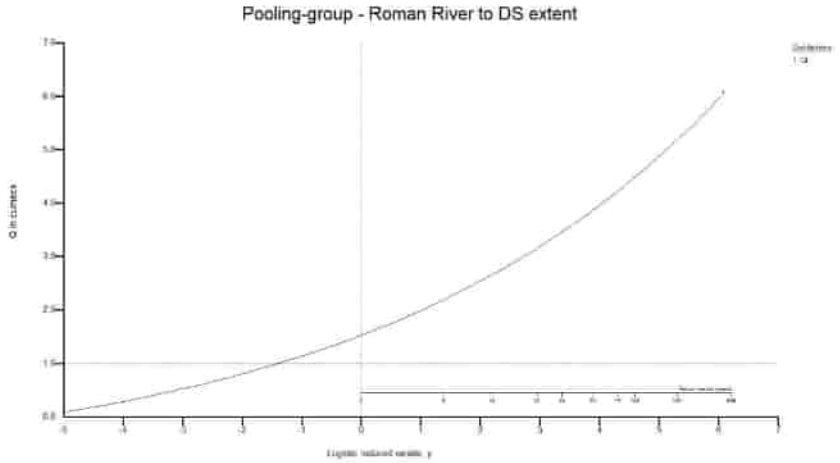
A.1.1 Pooling group composition

Table A.1 Pooling Group Composition

FINAL POOLING GROUP COMPOSITION – ROMAN RIVER TO DOWNSTREAM MODEL EXTENT

Station	Distance	Years of data	QMED AMAX	L-CV	L-SKEW	Discordancy
36010 (Bumpstead Brook @ Broad Green)	0.991	51	7.5	0.372	0.184	0.955
41020 (Bevern Stream @ Clappers Bridge)	1.303	49	13.66	0.203	0.181	0.744
25019 (Leven @ Easby)	1.418	40	5.384	0.343	0.378	1.475
36004 (Chad Brook @ Long Melford)	1.462	51	5.186	0.294	0.182	0.334
36003 (Box @ Polstead)	1.513	57	3.91	0.305	0.089	0.586
72014 (Conder @ Galgate)	1.588	50	16.465	0.233	0.162	0.175
36007 (Belchamp Brook @ Bardfield Bridge)	1.589	53	4.63	0.371	0.119	1.051
73015 (Keer @ High Keer Weir)	1.596	27	12.33	0.205	0.281	1.025
53017 (Boyd @ Bitton)	1.625	45	13.87	0.241	0.088	0.158
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	1.646	8	6.511	0.262	0.049	3.345
28058 (Henmore Brook @ Ashbourne)	1.652	13	8.838	0.188	-0.109	2.032
27010 (Hodge Beck @ Bransdale Weir)	1.658	41	9.42	0.224	0.293	0.856
37016 (Pant @ Copford Hall)	1.748	54	7.24	0.293	0.096	0.263
Total		539				
Weighted means		539		0.275	0.164	

Plate A.1 Roman River Growth Curve



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 7 MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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0 Executive Summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at Minor Watercourse 7 (MWC7), NGR grid reference TL798121. The modelling and hydrology analyses have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 2-dimensional (2D) type using TUFLOW software (BMT 2022) and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 40% climate change (CC). No flow reconciliation was required to be carried out for the design hydrology. No data was available for calibration and Environment Agency (EA) flood maps did not allow for the verification of the model. The analysis is therefore reliant on the use of industry standard modelling. The model was used to simulate baseline and with-scheme scenarios for the three design events mentioned. The with-scheme scenario involves a widening of the highway carriageway, and a new junction to the east of Minor Watercourse 7, which includes a new slip road running parallel to the north of the highway and crossing the watercourse. The extension of the existing highway culvert, CL7, was required and the introduction of a new culvert to convey flow under the slip road. Sensitivity analysis of roughness and inflows was carried out to further validate the results.

0.1.3 The analysis shows that the impact from the proposed-scheme on the downstream channel is negligible but present for the 5% (1 in 20) AEP event, whilst a small reduction in pass forward flow was observed in the other events. The flood risk consequences of the proposed-scheme were deemed inconsequential; therefore, mitigation works were not required.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the proposed-scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios, both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) are:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Minor Watercourses 7 (this report), 21, 21A, 23, 26

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out to assess the baseline and with-scheme scenarios for an unnamed tributary of the River Blackwater. The watercourse is designated as Minor Watercourse 7 based on the A12 drainage design numbering system for the main culvert crossings of the highway. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 The existing A12 crossing of the Minor Watercourse 7 (MWC7) will be retained by the proposed-scheme and extended upstream by approximately 30 metres to accommodate the widened carriageway. A

proposed culvert will be added upstream of the existing CL7, where the proposed-scheme junction slip road crosses the MWC7 north of the A12.

1.1.6 The carriageway will be upgraded to three lanes in each direction as well as the construction of a side road, north of the A12 carriageway. This will require a culvert north of the existing crossing, upstream of the watercourse.

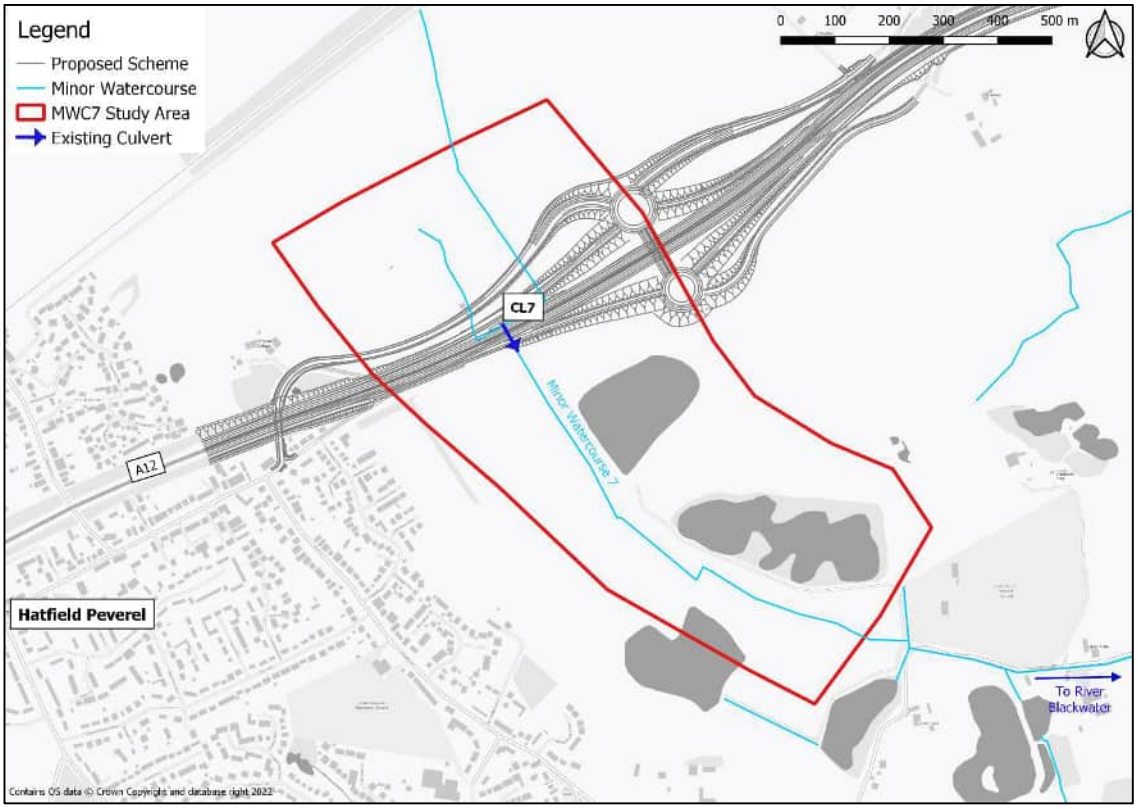
1.1.7 This report should be read in conjunction with the FRA [R10060/APP/6.6] (Jacobs 2022).

1.2 Study area

1.2.1 MWC7 is situated in the immediate east of Hatfield Peverel, Essex. The watercourse has two drainage paths originating approximately 300m upstream (north) of the A12, flowing southeast (see Plate 1.1). The two tributaries converge immediately upstream of the A12, where a 375mm culvert, CL7, conveys the flow under the highway. MWC7 continues its course southeast for approximately 3km before joining the River Blackwater.

1.2.2 The model extent covers an area of 0.60km^2 and includes a total reach length of approximately 1600m.

Plate 1.1 MWC7 Study Area



2 Input Data

2.1.1 The data used to construct the hydraulic model for MWC7 is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR	1m resolution composite Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) 2020 data. Used to inform the hydraulic model with ground level information.	Department for Environment, Food & Rural Affairs (DEFRA), 2020
Ordnance Survey (OS) Mastermap	Land use data used to specify roughness values across the 2D floodplain.	National Highways
Hydrological Analysis	Calculation of inflow hydrographs into the model.	National Highways, 2020
As-built survey	Survey of key existing structures in the model	MKSurveys December 2019
Outline Design Drawings	Drawings of the proposed-scheme design.	National Highways, 2021
Flood Zone Mapping	EA Risk of Flooding Surface Water (RoFSW) flood maps. Used for model verification.	Environment Agency

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of MWC7 for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event is also required including an allowance for climate change (plus CC). For full details of the flow derivation methods and background hydrology refer to Appendix A Minor Watercourse 7 Hydrology Report (HE551497-JAC-EWE-SCHW-RP-LE-0084).

3.2 Hydrological Considerations

3.2.1 MWC7 is located in South East England just outside Hatfield Peverel and crosses the A12 at approximately TL 79865 12176. MWC7 has a small catchment area of approximately 0.59km^2 to the downstream model extent. This minor watercourse discharges to the River Blackwater approximately 3km downstream of the A12 road crossing. The catchment is classified as essentially rural with land use being composed of predominantly agricultural land. There is one waterbody located downstream of the A12 road crossing within the model extent catchment area.

3.2.2 The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 411d (Hanslope) within the upper and eastern reaches of the catchment and 571z (Hamble 2) within the middle and lower reaches of the catchment. The geology of the catchment was assessed using the Geology of Britain Viewer and found to be composed of London Clay formations consisting of clay, silt and sand with superficial deposits being composed of clay, silt and Lowestoft Formation (Diamicton).

3.3 Peak Flow Derivation

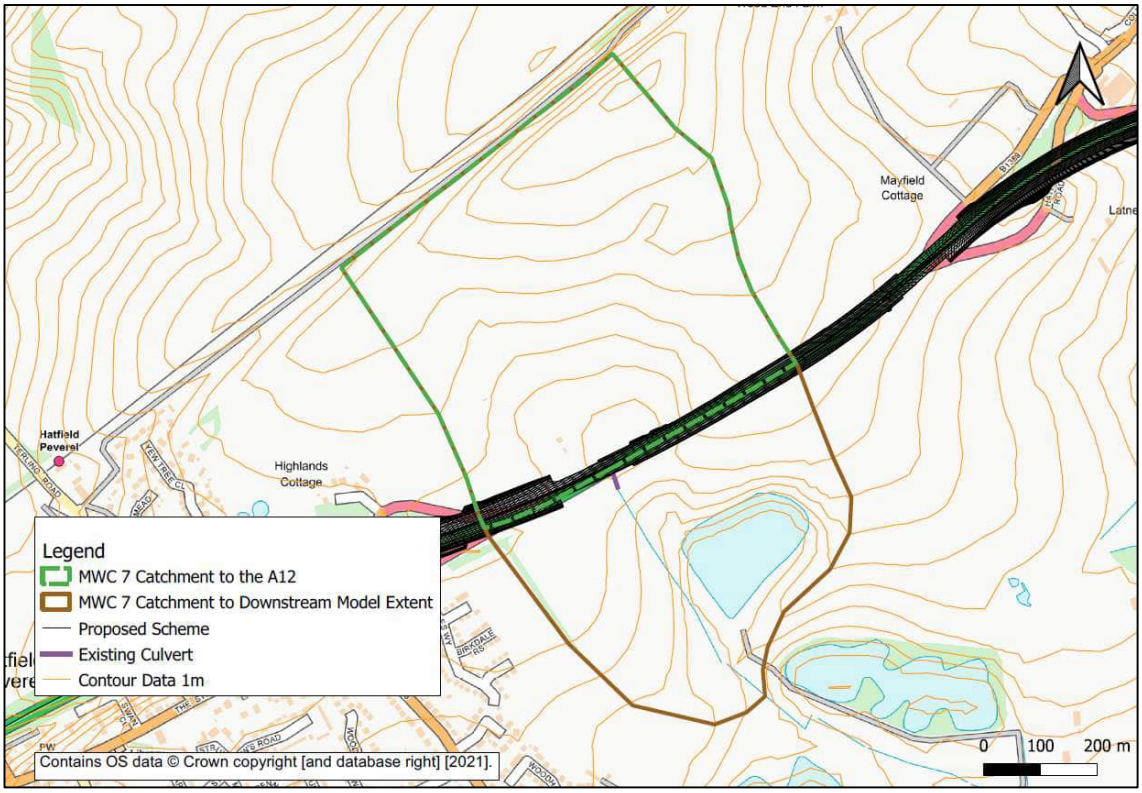
3.3.1 MWC7 is a small ungauged catchment and therefore flow estimation is open to great uncertainty. Both ReFH2.3 and FEH Statistical methods were therefore applied to estimate design peak flows for this watercourse. ReFH2.3 resulted in higher design peak flow estimates for MWC7 than the FEH Statistical method. Given ReFH2.3 uses the latest rainfall data (FEH13) and soil data (i.e. BFIHOST19), and as it allows climate change uplift to be applied directly in the software, the ReFH2.3 method was selected for final peak flows estimates.

3.4 Hydraulic Model Inflows

3.4.1 For hydraulic modelling purposes one flow estimation point has been adopted at the downstream modelling extent due to the small catchment size of MWC7. Within the hydraulic model, flows are split by applying 61% of the flow as point inflow upstream of the A12 road crossing (at the start of the hydraulic model) and the remaining 39% of the inflow from the residual catchment as a lateral inflow downstream of the A12 crossing.

3.4.2 A schematisation of the MWC7 catchment is presented in Plate 3.1.

Plate 3.1 Schematisation of the MWC7 catchment



3.4.3 The adopted final peak flow estimates for the MWC7 catchment are presented in Table 3.1. The flows have been assessed as generally appropriate for this small, essentially rural catchment located in South East England with relatively low rainfall (SAAR = 557mm), BFIHOST19 = 0.443 / SPRHOST = 42.93.

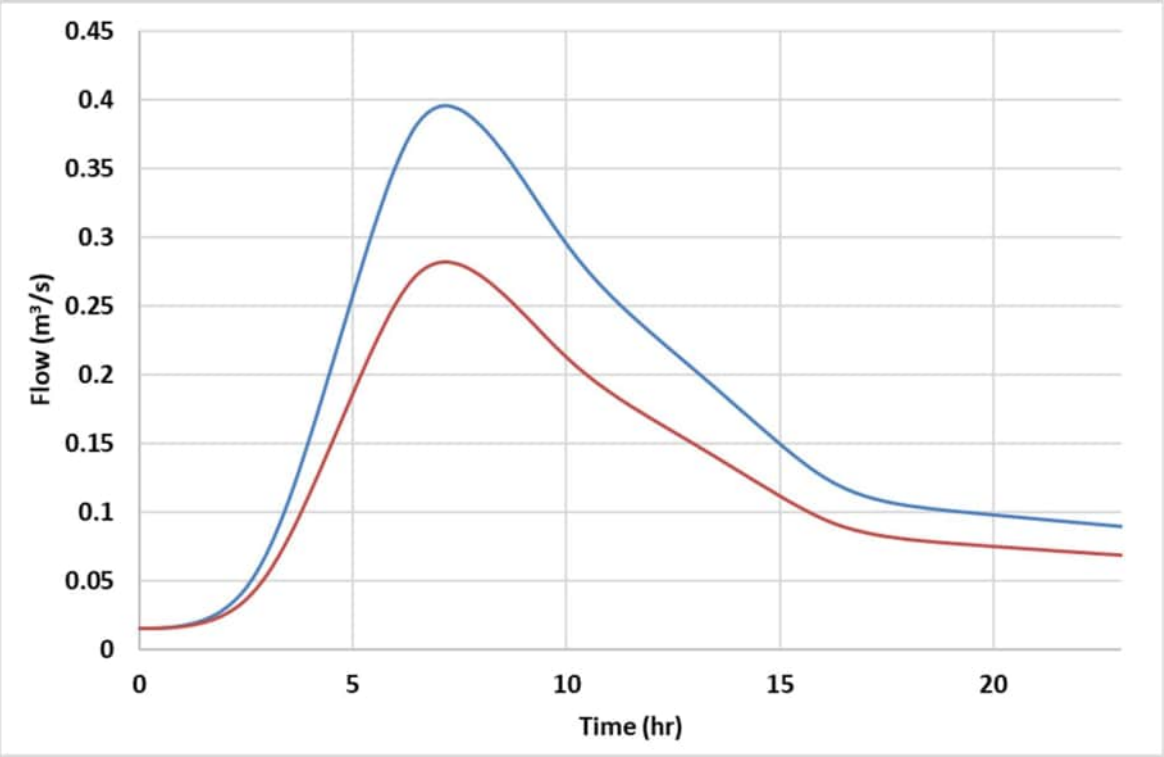
Table 3.1 MWC7 peak flow estimates

Inflow	Peak Flow (m^3/s)		
	AEP 5% (1 in 20)	AEP 1% (1 in 100)	AEP 1% + CC 40% uplift (1 in 100)
Full Catchment	0.281	0.396	0.575

3.5 Inflow Hydrograph Shape

3.5.1 As the MWC7 catchment is ungauged it was assessed as appropriate to derive hydrograph shapes for input into the hydraulic model using ReFH2.3. The ReFH2.3 hydrograph shape was derived at the downstream model extent. The 5% (1 in 20) and 1% (1 in 100) AEP event hydrographs are presented in Plate 3.2.

Plate 3.2 MWC7 5% (1 in 20) and 1% (1 in 100) AEP Inflow Hydrographs



3.6 Climate Change

3.6.1 A climate change allowance for peak rainfall intensity was obtained from the latest government guidance at the time of assessment (Environment Agency, 2021). The peak rainfall intensity allowance is appropriate for small catchments (less than 5km^2). An uplift of 40% (the upper end, 2080s scenario) was applied to the 1% (1 in 100) AEP event. The peak flow for the 1% (1 in 100) AEP event plus 40% climate change uplift at the downstream model extent is 0.575 m^3/s.

3.7 Flow Reconciliation

3.7.1 Given the model inflows were estimated just 0.5km downstream of the MWC7 A12 crossing (target location) using the ReFH2.3 method, flow reconciliation was not considered necessary for this minor watercourse.

3.8 Summary

3.8.1 Design peak flows and inflow hydrographs for MWC7 were derived using ReFH2.3, using the FEH catchment descriptors available on the FEH Web Service (available to select approximately 350m downstream of the end of the modelled reach) and adopting climate change allowances as per latest EA guidance. Design inflow hydrographs to be used in the numerical hydraulic model were derived for MWC7 using the critical storm duration to the downstream model extent due to the small and straight forward nature of the target catchment.

4 Baseline Modelling

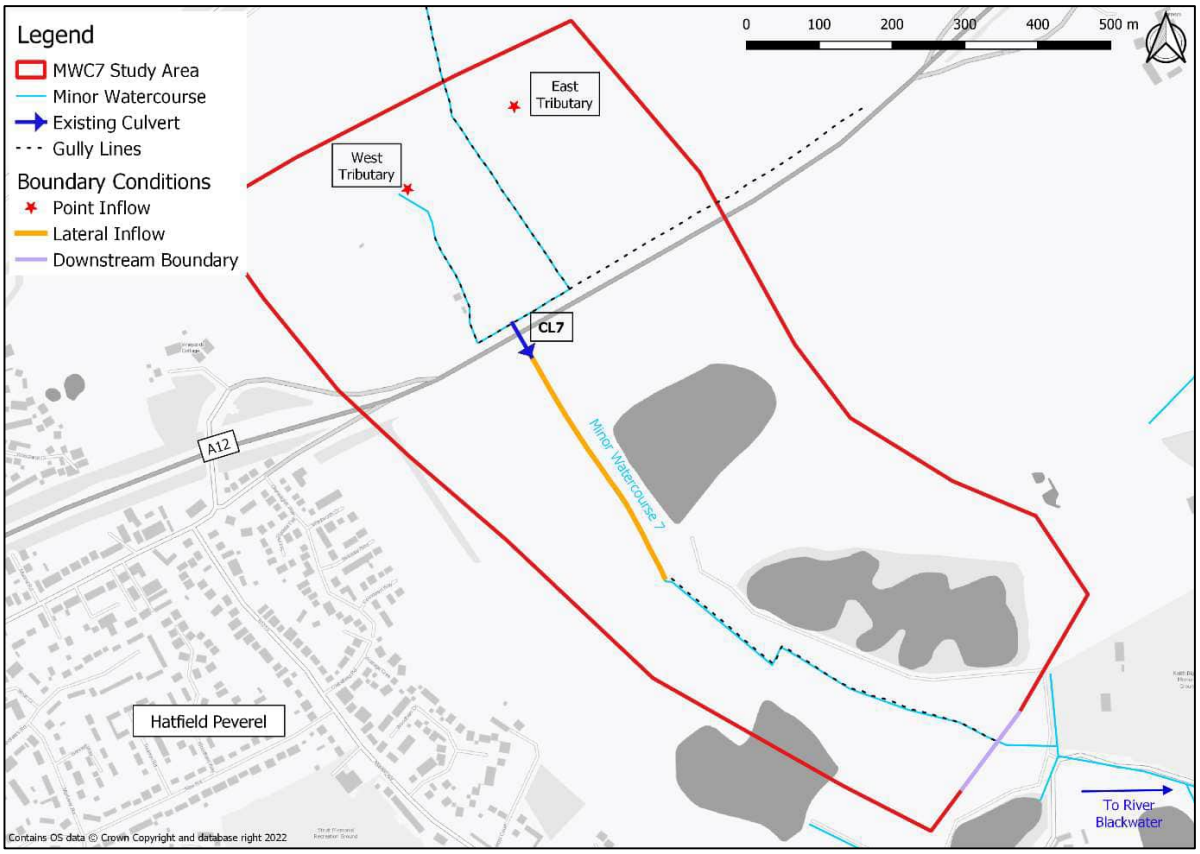
4.1.1 The MWC7 hydraulic model uses a two-dimensional (2D) schematisation, built in TUFLOW version 2020-10-AA (BMT, 2020). 1D 'ESTRY' elements were used to represent the culverts within the TUFLOW model and linked to the 2D domain via SX links. The 2D representation of the channel was considered appropriate because the highway culvert is the main control on pass forward flows and upstream water levels and therefore a detailed representation of channel capacity is not required.

4.1.2 The model was built from scratch and used to simulate baseline and with-scheme scenarios. Three events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) AEP + 40% CC.

4.2 Model Extent

4.2.1 The model extent covers an area of 0.6 km^2. The model extent has been drawn in a way to capture the left and right floodplains of the upper catchment and the drainage area downstream of the hydrological catchment. Representation of the model extent is shown in Plate 4.1.

Plate 4.1 Model Setup of the MWC7 catchment



4.3 Model Resolution and Topography

4.3.1 1m horizontal resolution LiDAR DTM data from 2020 was available for the model extent and was downloaded from the DEFRA website to inform the hydraulic model with ground level information.

4.3.2 A 1m grid cell resolution was deemed appropriate for the model to capture the necessary detail of the watercourse in the 2D Domain as LiDAR inspection shows the downstream channel has an average width of 2m between bank tops. The model grid has been orientated to align with key flow paths.

4.3.3 The representation of existing farm drains and watercourses has been reinforced in the model with gully lines applied using point data selected from the 2020 1m LiDAR. Several simple footbridge structures have been ignored downstream of A12, as investigation shows that these features would not make any difference to the model results within the area of interest.

4.3.4 The culvert (CL7) conveys flow from north to south of the A12 and is the only structure modelled within MWC7 study area. CL7 is a concrete, circular culvert with a surveyed diameter of 375mm and a length (measured in GIS) of 52.5m. CL7 was schematised in the model using an embedded 1D 'Estry' circular culvert with invert levels taken from the available survey and a Manning's 'n' roughness value of 0.013. Culvert losses are applied based on standard recommended values; 0.5 and 1 for the inlet loss and outlet loss, respectively.

4.3.5 As seen in Plate 4.1, the eastern tributary inflow does not align with the presented gully line. This is due to the natural flow path being predominantly overland through the field towards the drain. This was verified using a "Rain on grid" modelling technique.

4.4 Hydraulic Friction

4.4.1 OS MasterMap data was used to identify land use type and inform the TUFLOW model with different hydraulic roughness coverages. Hydraulic roughness coefficients were taken from standard guidance (Chow, 1959) and applied over each grid cell of the 2D domain depending on the land use taken from the MasterMap data. Depth varying roughness has been used to stabilise the model at shallow depths. Roughness values adopted are shown in Table 4.1. For depths between 50mm and 300mm, the Manning's n value is interpolated between n1 and n2. Below 50mm, n1 is applied, and above 300mm, n2 is applied. The default interpolation method uses a curved fit so that the n values transition gradually.

4.4.2 As mentioned in Section 4.2.5 above, CL7 was modelled as having a roughness value of 0.013, taken from Chow (1959) as a high value for concrete culvert that is straight and free of debris.

Table 4.1 MWC7 hydraulic roughness values

Land Use	Depth varying Manning's 'n' roughness	
	n1	n2
Buildings	1.0	
Inland Water	0.025	
General Surface – Natural	0.07	0.035
Natural Environment / Orchard	0.1	
Roads, Tracks, Paths, Roadside and Steps	0.05	0.025
Manmade Structures	0.05	0.025
Unclassified Land (Industrial yards, car parks)	0.05	0.025

4.5 Boundary Conditions

4.5.1 Two types of boundary conditions have been implemented in the model and can be seen in Plate 4.1:

- Flow-time point boundaries (ST) representing the MWC7 upstream catchment inflow and a lateral inflow, applied along a channel length of 350m downstream from the A12 crossing. Hydraulic model inflows are split by applying 61% of the flow as point inflow upstream of the A12 road crossing and the remaining 39% of the inflow from the residual catchment as a lateral inflow downstream of the A12 crossing, as further detailed in Section 3.4.

- The downstream boundary is applied as a normal depth boundary line (HQ) with surface slope taken from inspection of the LiDAR data, allowing free-flow out of the model.

5 Modelled Events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 40% uplift in accordance with the latest EA guidance as detailed in Section 3.

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, and hydrological inflows (see Section 6.4).

Table 5.1 Modelled events for MWC7

Scenario	5%AEP (1 in 20)	1%AEP (1 in 100)	1%AEP + 40% CC (1 in 100)
Baseline	✓	✓	✓
Design	✓	✓	✓
Roughness Sensitivity		✓	
Hydrological Inflow Sensitivity		✓	

6 Model Proving

6.1 Introduction

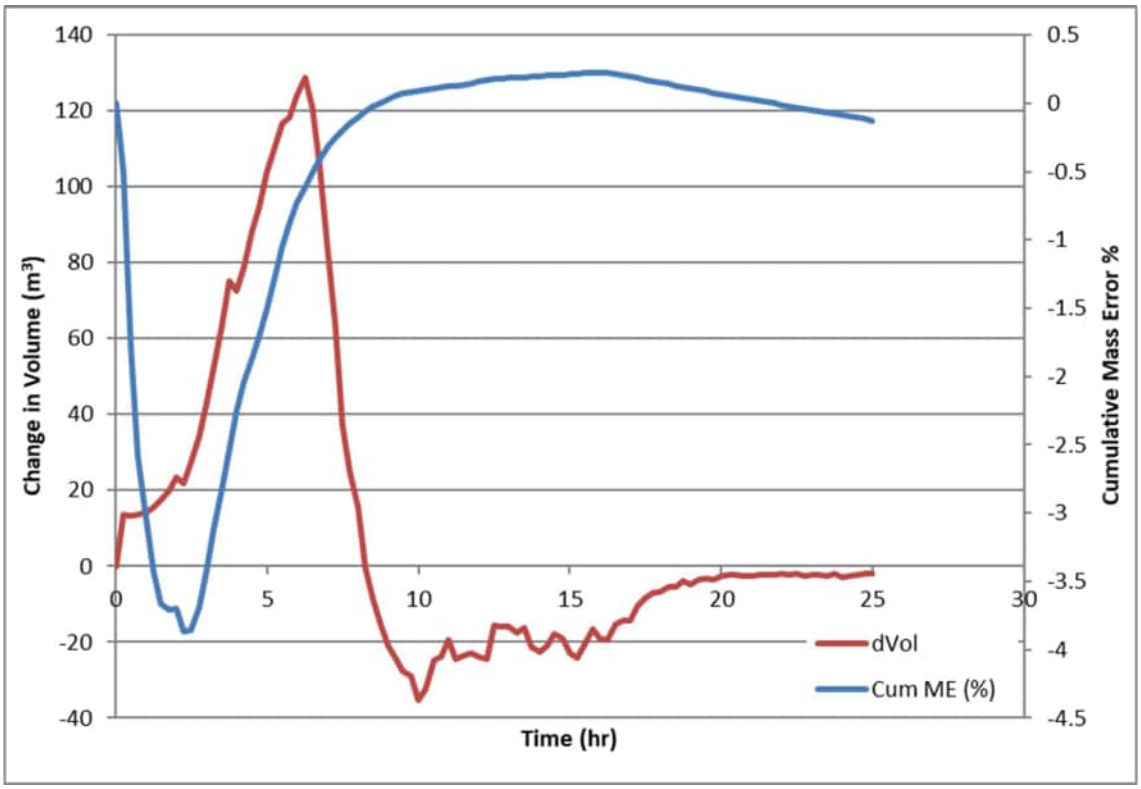
6.1.1 The following sections discuss the model performance and the verification process.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model convergence was achieved. The cumulative mass error diagnostics output from the model have been checked for each model simulation. Plate 6.1 shows the cumulative mass error (Cum ME) and the change in volume (dVol) for the baseline 1% (1 in 100) AEP event simulation. This plot is typical for all the events and scenarios simulated. The tolerance range recommended by the software manual is +/-1% mass balance error.

6.2.2 Although the cumulative mass error exceeds tolerance limits during the onset of the floodplain wetting, it stabilises to less than 1% error before the inflow hydrographs peak. This is considered acceptable. Plate 3.2 demonstrates the cumulative mass error and the corresponding dVol.

Plate 6.1 Model proving Cum ME and dVol plot for baseline 1% AEP event



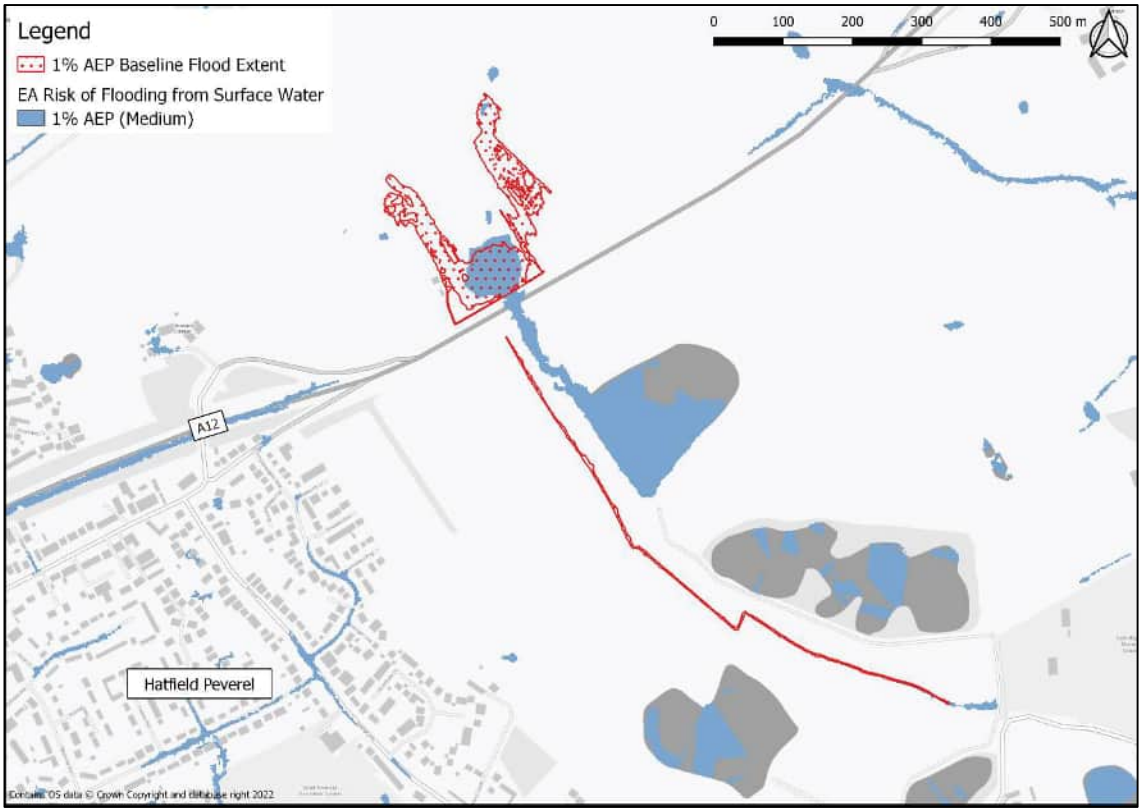
6.3 Comparison with EA Flood Maps

6.3.1 MWC7 has not been included in the Environment Agency (EA) published Flood Zone mapping of fluvial flood risk. Therefore, a direct comparison between model results and EA flood maps is not possible. However, the EA Risk of Flooding from Surface Water (RoFSW) mapping can be used to identify areas of pooling.

6.3.2 Plate 6.2 shows the modelled 1% (1 in 100) AEP event flood extent and the EA RoFSW mapping extents. There is limited data available for verification of the MWC7 model, as the EA flood map shows some greater areas of ponding within the study area, connecting the watercourse to floodplain storage features. As the inflows are directly applied into the channel, flood mechanisms from the RoFSW could not be replicated within the MWC7 model.

6.3.3 Furthermore, overtopping of the A12 carriage is evident in the RoFSW mapping because the existing culverts across the A12 have not been represented. Considering this, verification of the model cannot be based on the comparison of results with EA flood maps. The analysis is therefore reliant on the use of industry standard modelling and hydrology tools, as is common for small watercourse studies.

Plate 6.2 EA Risk of Flooding from Surface Water and 1% AEP Extent



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness and hydrological inflows. All sensitivity tests were performed using the 1% (1 in 100) AEP event. 2D model outputs for the sensitivity testing are presented in Appendix B.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Appendices B shows the impact of changing the model roughness on predicted peak water levels. The results show that only the road drain upstream of CL7, along the toe of the A12 embankment is sensitive to adjustment of model roughness, with water level response of +220mm and -200mm, when the roughness is increased and decreased respectively. Downstream of the A12, local spots in-channel are also showing moderate sensitivity to changes in roughness. The change in roughness has no impact on the 1% (1 in 100) AEP flood extent.

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Appendix B shows the impact of changing the model inflows on predicted peak water levels. The results show that the downstream in-channel water levels are moderately sensitive to the reduction in flow. When inflows are increased by 20%, CL7 is the controlling structure and limit the downstream impacts. The change in flow has no impact on the 1% (1 in 100) AEP flood extent.

7 Design Modelling

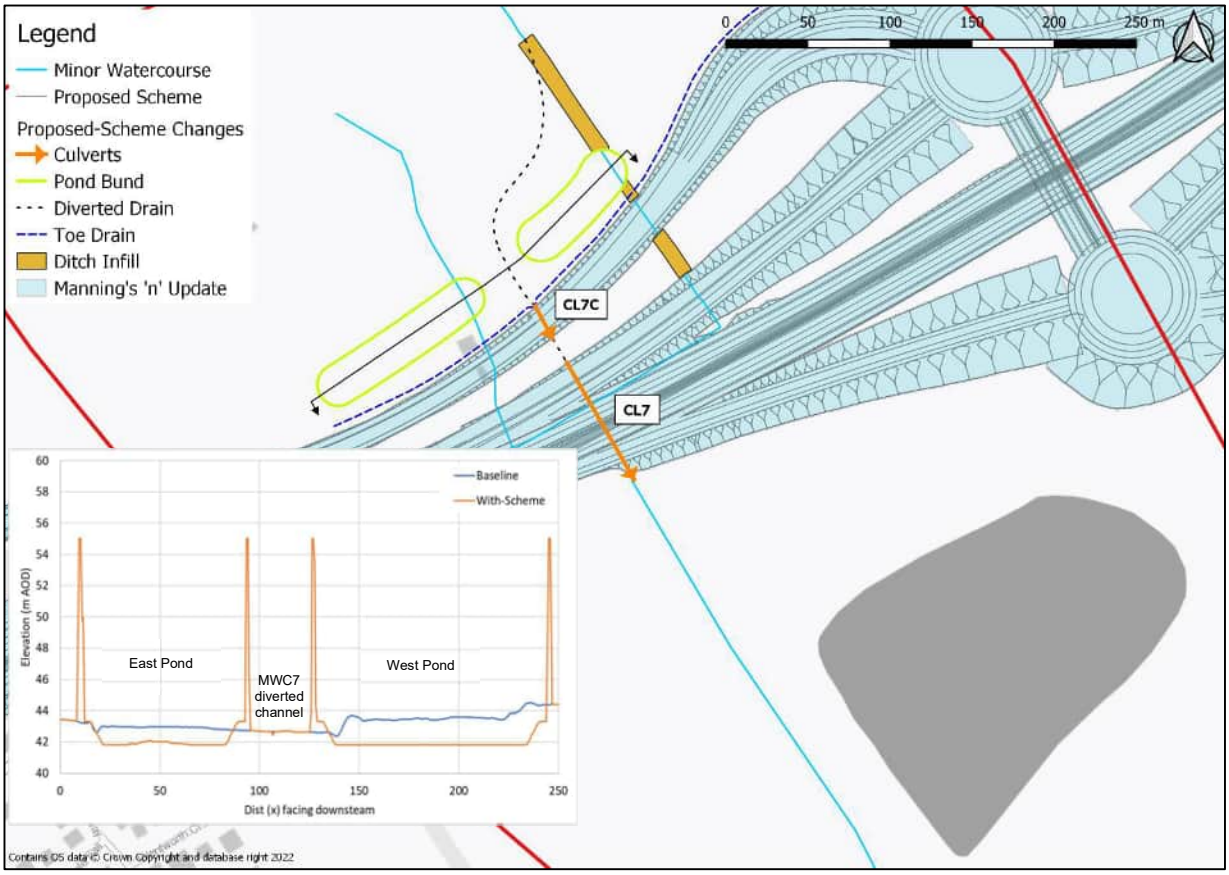
7.1 Design Model Build

7.1.1 The baseline model was updated to include the proposed A12 Stage 3 Scheme. Changes to the model were as follows and illustrated in Plate 7.1:

- The proposed A12 side road design was included in the model as modification to the baseline model topography using an Ascii DTM surface overlain on the LiDAR baseline.
- Culvert 7 (CL7) was extended 30m to the north, on the upstream side, to accommodate highway embankment widening. The new culvert section matches the 375mm diameter and concrete pipe material of the existing structure. The proposed-scheme works do not include any extension of the southern face of the highway embankment, so no downstream extension is required.
- A proposed culvert (CL7C) was added to upstream of the existing CL7. CL7C is a 450mm concrete circular culvert and 22m in length, located where the proposed-scheme junction slip road crosses the MWC7, north of the A12. CL7C was modelled using an Estry element with a manning's value of 0.013 for a concrete surface, and standard inlet and outlet losses as described in Section 4.2. Invert levels were matched to existing channel levels.
- The eastern farm ditch north of the A12 is redirected south-west towards the CL7C inlet. To represent the new channel; a gully line was used to realign the channel while a Z-shape was used to backfill the existing drain to surrounding ground levels. Additionally, a new toe-drain has also been included as a gully line on the upstream (northern) side of the proposed-scheme slip road.
- The Manning's roughness layer was updated to reflect the additional road and in-channel surfaces resulting from the proposed-scheme. Manning's 'n' values were assigned as per Table 4.1.

7.1.2 As part of the proposed-scheme, highway drainage attenuation ponds will occupy the MWC7 floodplain immediately to the north of the A12. these features have been represented in the MWC7 design model using TUFLOW Z-shapes, comprising the ponds and the perimeter bund structures. Both pond base levels are 41.8 m AOD and the perimeter bund crest level is set to 55m AOD, which is a nominal, conservative, glass wall value. The final design level will be determined in future design stages. An indicative pond cross-section is shown in Plate 7.1

Plate 7.1 MWC7 Mitigation model updates



8 Model Results

8.1 Baseline Results

8.1.1 Plate 8.1 shows the maximum flood extent for all modelled events for the baseline scenario. The flood extents for individual flood events can be found in Appendix C.

8.1.2 The area immediately upstream of A12 is flooded for all events simulated, with almost identical flood extents in the 5% (1 in 20) and 1% (1 in 100) AEP events. This is due to the upstream field being extremely flat, with steep sides acting to contain the flood depths. This natural floodplain storage then flows into the field drain and onwards to CL7, without any impounding on A12 in the 5% (1 in 20) and 1% (1 in 100) AEP events. For the 1% (1 in 100) AEP + 40% CC, the modelled flood extent reaches the toe of the A12 embankment.

8.1.3 There is no flooding to private properties and the A12 carriageway for any of the modelled events and downstream of the A12 crossing, the modelled flows remain in channel for all events.

8.1.4 Table 8.1 shows the peak flow through CL7 with upstream and downstream peak water levels for all events. For the 1% (1 in 100) AEP + 40% CC event, CL7 reaches the pipe full capacity of 0.26m^3/s. The culvert runs surcharged at the upstream end for all events simulated.

Table 8.1 Baseline structure peak flow and water levels for all events

AEP	Culvert 7		
	Peak Flow (m^3/s)	U/S Peak Water Level (m AOD)	D/S Peak Water Level (m AOD)
5% (1 in 20)	0.16	41.78	40.84
1% (1 in 100)	0.23	42.17	40.90
1% + 40% CC (1 in 100)	0.26	42.52	40.91
Culvert Soffit	-	41.70	40.91

Plate 8.1 Baseline Flood Extents



8.2 With-Scheme Model Results

8.2.1 With-Scheme modelling results show there are no properties at risk of flooding and the A12 carriageway and northern slip road remain unaffected by flooding for all the events simulated. The baseline flooding has been displaced north around the proposed-scheme side road and the main highway widening works, as seen in Plate 8.2.

8.2.2 Whilst the overall flood extent, north of the A12, remains largely unchanged in the with-scheme scenario for all events, upstream flood depths, local to the proposed-scheme, have increased due to the restriction caused by with-scheme works, the proposed A12 drainage design and the natural topography.

8.2.3 The extended CL7 culvert is at capacity for the 1% (1 in 100) AEP and 1% (1 in 100) AEP + 40% CC events, these peak flows are seen in Table 8.2. Between the northern slip road and the A12, a small storage area and additional culvert provide minor attenuation to flows. In the two larger modelled events, pass forward flow has been reduced at the downstream end of the model by up to -0.06m^3/s. For the 5% (1 in 20) AEP event, model outflow is marginally increased by +0.01m^3/s as less upstream attenuation storage is engaged.

8.2.4 No mitigation works were deemed necessary, as the change in pass forward flow is small, especially in relation to the receiving watercourse which is the River Blackwater, and the only change in flood risk is a small area immediately upstream of the A12 located inside the project compulsory purchase order (CPO) boundary.

8.2.5 Plate 8.3 shows the changes to the water level due to the proposed A12 scheme for the 1% (1 in 100) AEP + 40% CC event. The maximum flood level difference between baseline and with-scheme are categorised into levels of impact due to the proposed-scheme. The impact of proposed-scheme for all other flood events can be found in Appendix C.

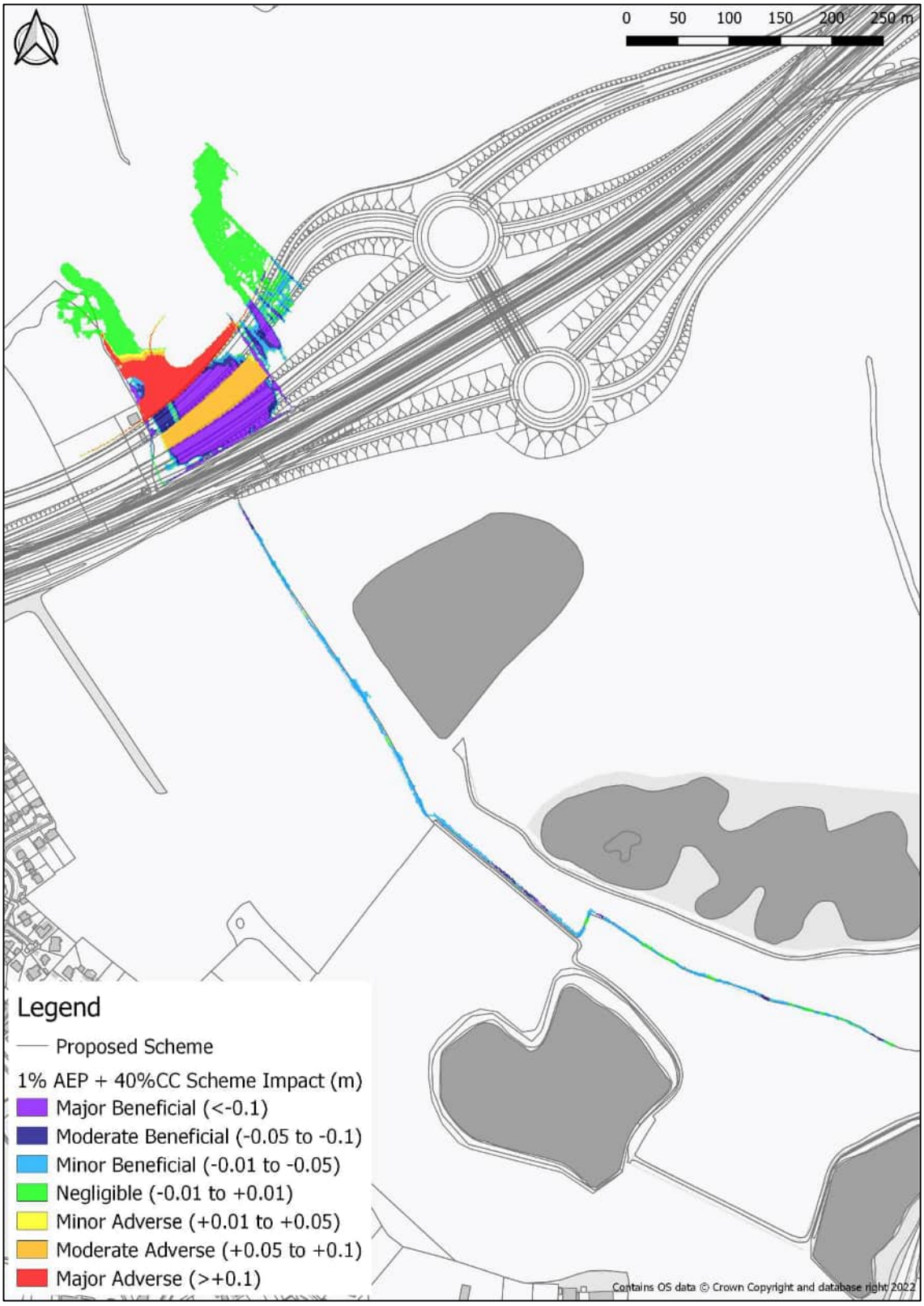
Table 8.2 With-Scheme structures peak pass-forward flows for all events

Event	Downstream boundary Peak Flow (m^3/s)		
	Baseline	With-Scheme	Difference
5% AEP (1 in 20)	0.261	0.272	+0.011
1% AEP (1 in 100)	0.379	0.360	-0.019
1% + 40% CC AEP (1 in 100)	0.481	0.437	-0.044

Plate 8.2 With-Scheme maximum modelled flood extents



Plate 8.3 1% AEP + 40% CC Water Level Difference (With-scheme – Baseline)



9 Model Assumptions and Limitations

9.1 Introduction

9.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological, surveyed, and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

9.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed-scheme and are therefore, appropriate for the flood risk assessment.

9.2 Hydrology

9.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- As MWC7 is a small ungauged catchment, there is inevitable uncertainty associated with flow estimation for small, ungauged catchments.
- The ReFH2.3 rainfall runoff model is the most appropriate for the target catchment, as it uses the most up to date BFIHOST19 and FEH13 rainfall data.

9.3 Hydraulic Modelling

9.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for MWC7 are as follows:

- Culvert and model roughness values have been assigned across the model using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the TUFLOW Manual. The dimensions for structures have been based on detailed survey measurements.
- The 1m horizontal resolution LiDAR data is assumed to appropriately represent the floodplain.
- A 1m grid has been used. This is deemed a sufficient level of detail to represent floodplain topography and flooding mechanisms predicted by the model in the vicinity of the proposed-scheme.
- No calibration was carried out as the catchment is ungauged.

10 Conclusion

10.1.1 This report has detailed the modelling carried out to assess the baseline and with-scheme flood risk for Minor Watercourse 7 with respect to the location of the proposed highway scheme.

10.1.2 The results of the baseline modelling have shown that there are no properties at risk of flooding and the A12 remains free from flooding up to the 1% (1 in 100) AEP + 40% CC event. This remains the case with the proposed-scheme in place.

10.1.3 Some of the baseline flood extent been displaced by the proposed-scheme slip road and main highway widening works, however the modelled flood extent remains unchanged. The proposed culverts and floodplain storage within the proposed-scheme causes a small betterment to the downstream pass forward flows in the 1% (1 in 100) AEP and 1% (1 in 100) AEP + 40% CC events. However, there is a minor increase (+0.01m^3/s) in pass forward flow in the 5% (1 in 20) event. This impact remains within the channel of MWC7, and the proposed-scheme poses no significant change in flood risk to the River Blackwater downstream due to the very small change in pass forward flow. Therefore, no mitigation works are deemed necessary.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

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Appendix A - Minor Watercourse 7 Hydrology Report

HE551497-JAC-EWE-SCHW-RP-LE-0084

Appendix B - Sensitivity Testing

Plate B.1 Manning's 'n' Roughness changed by -20% (Water level difference for the 1% AEP event)



Plate B.2 Manning's 'n' Roughness changed by +20% (Water level difference for the 1% AEP event)



Plate B.3 Hydrological Inflows into model changed by -20% (Water level difference for the 1% AEP event)

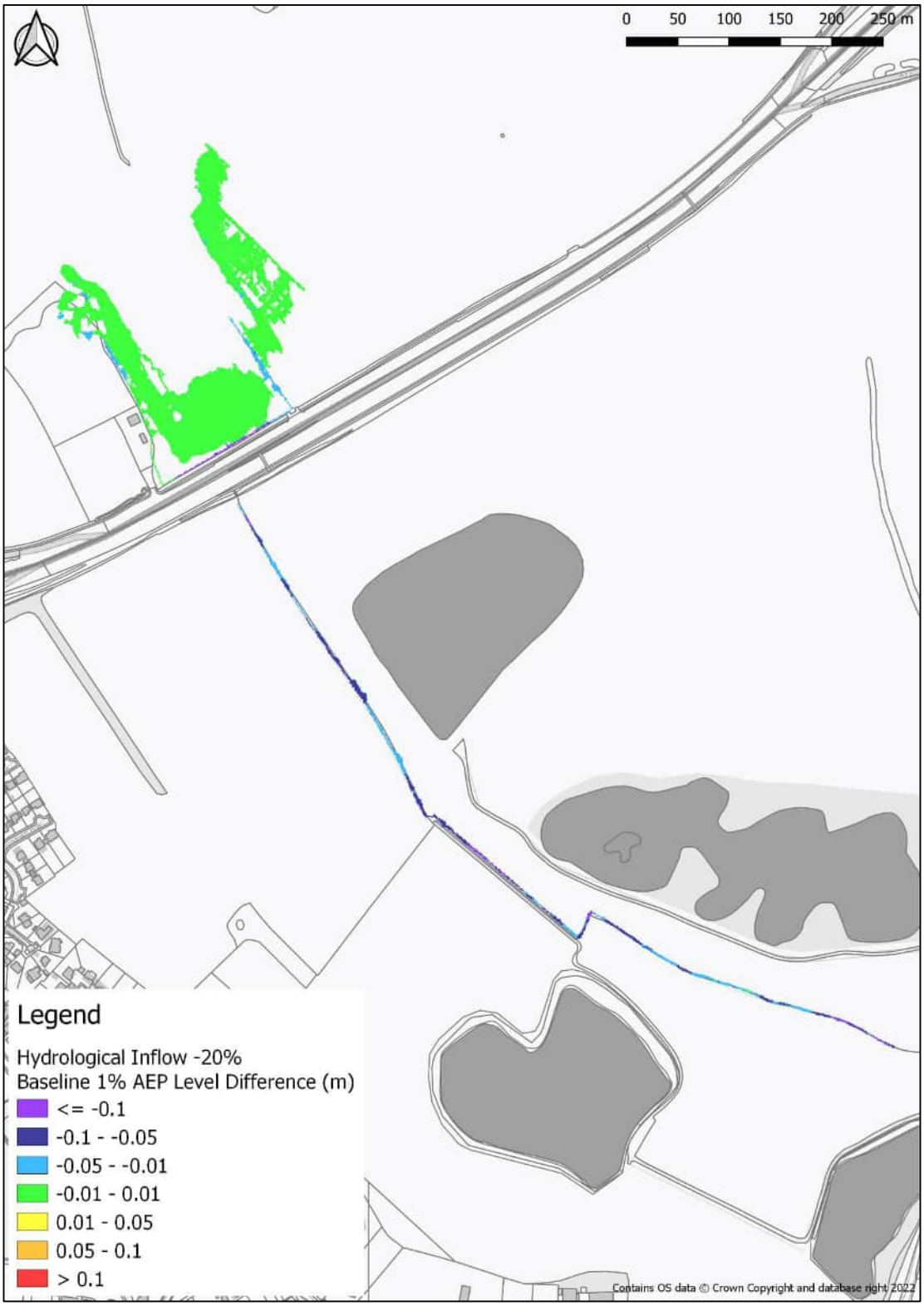


Plate B.4 Hydrological Inflows into model changed by +20% (Water level difference for the 1% AEP event)



Appendix C - **Flood Maps**

Plate C.1 5% AEP maximum flood extent for baseline and with-scheme scenarios

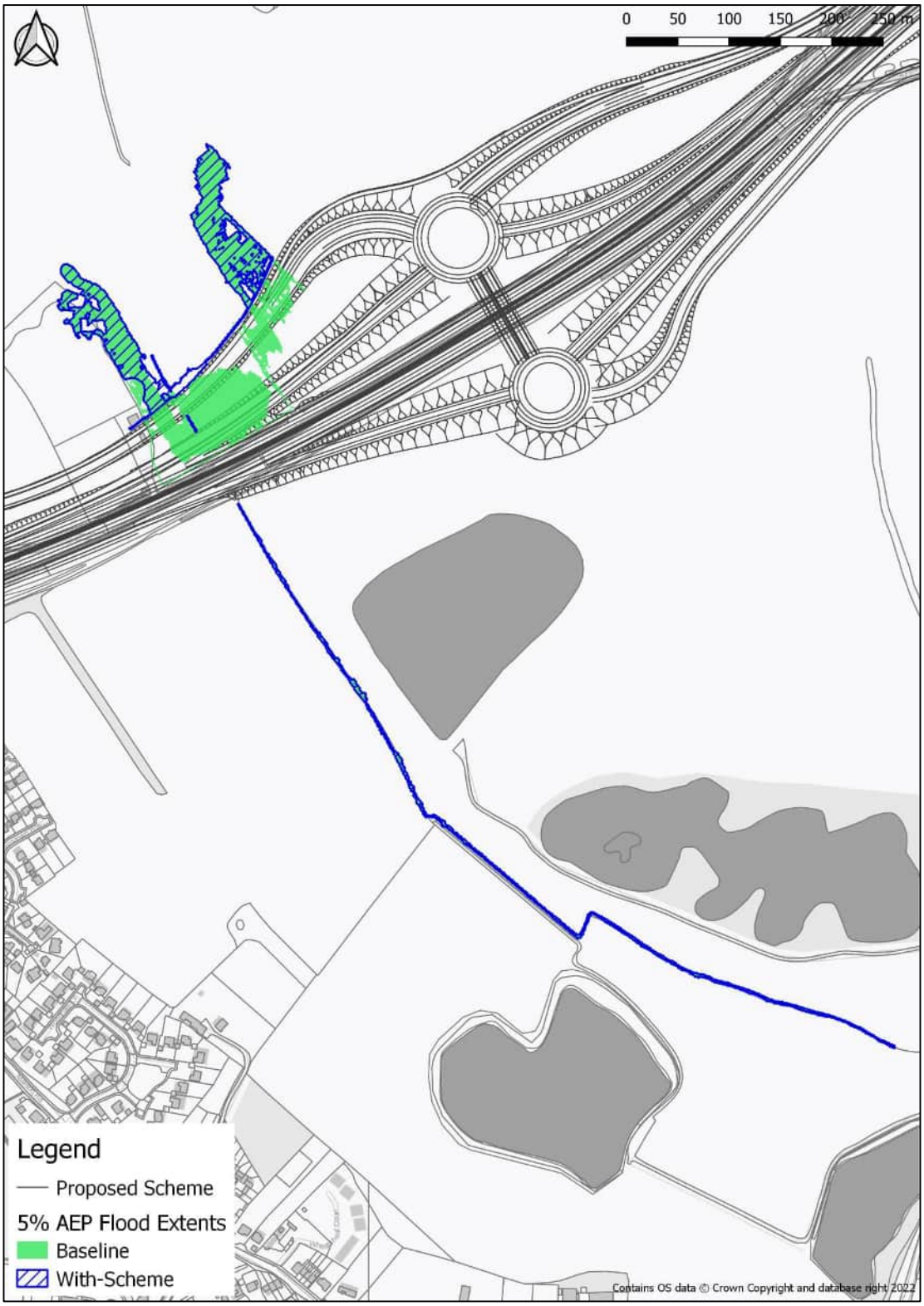


Plate C.2 1% AEP maximum flood extent for baseline and with-scheme scenarios

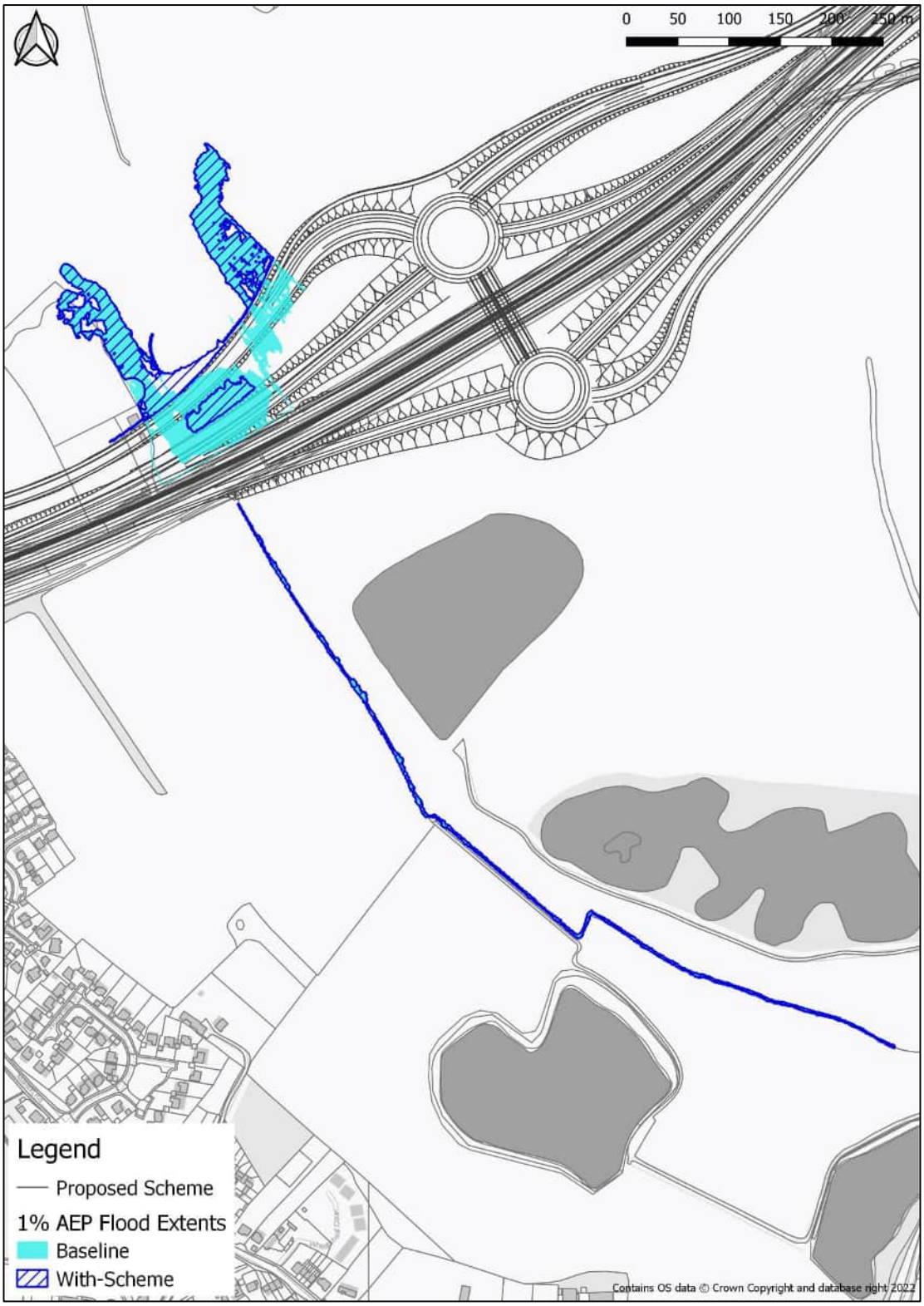


Plate C.3 1% AEP + 40%CC maximum flood extent for baseline and with-scheme scenarios

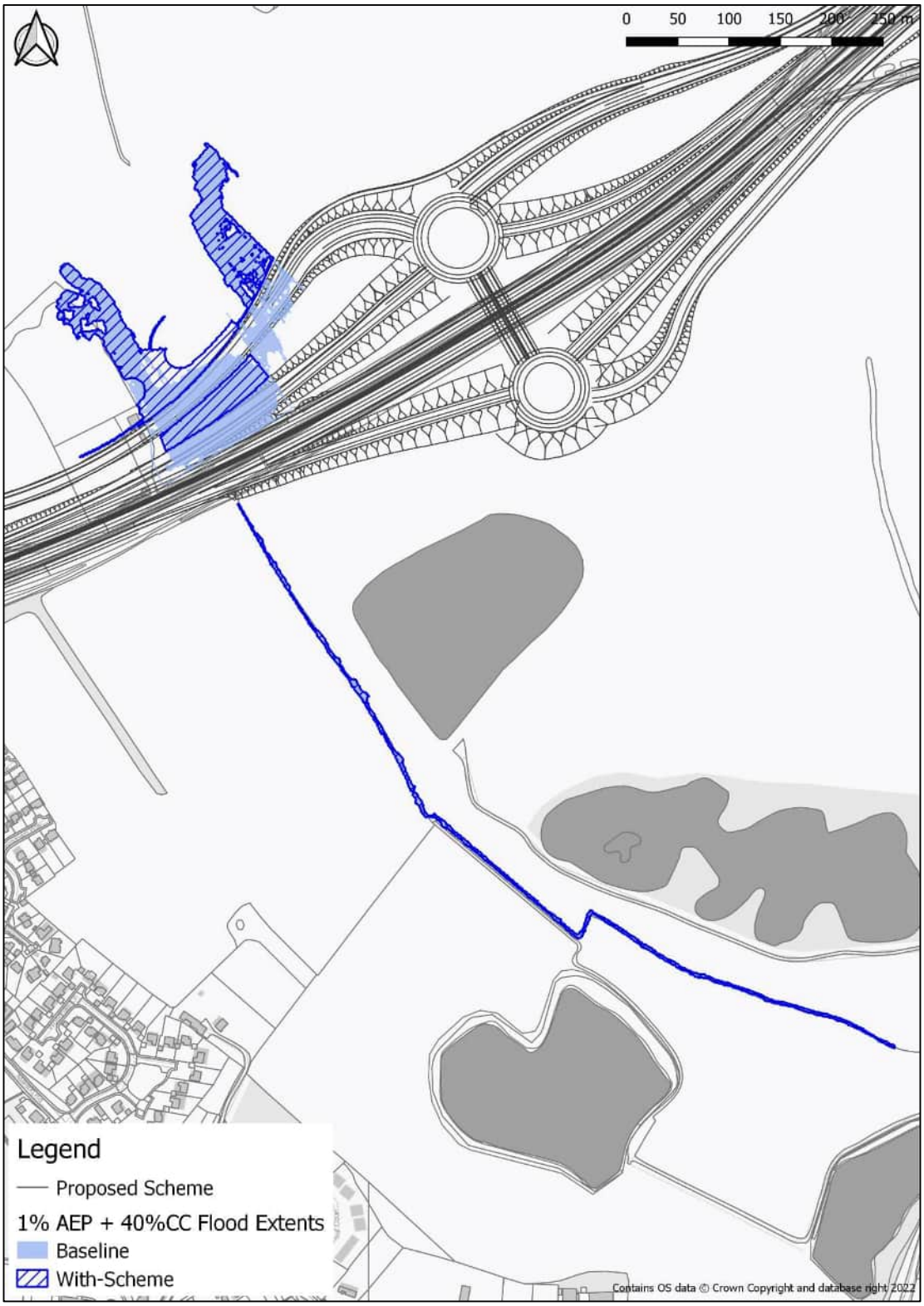


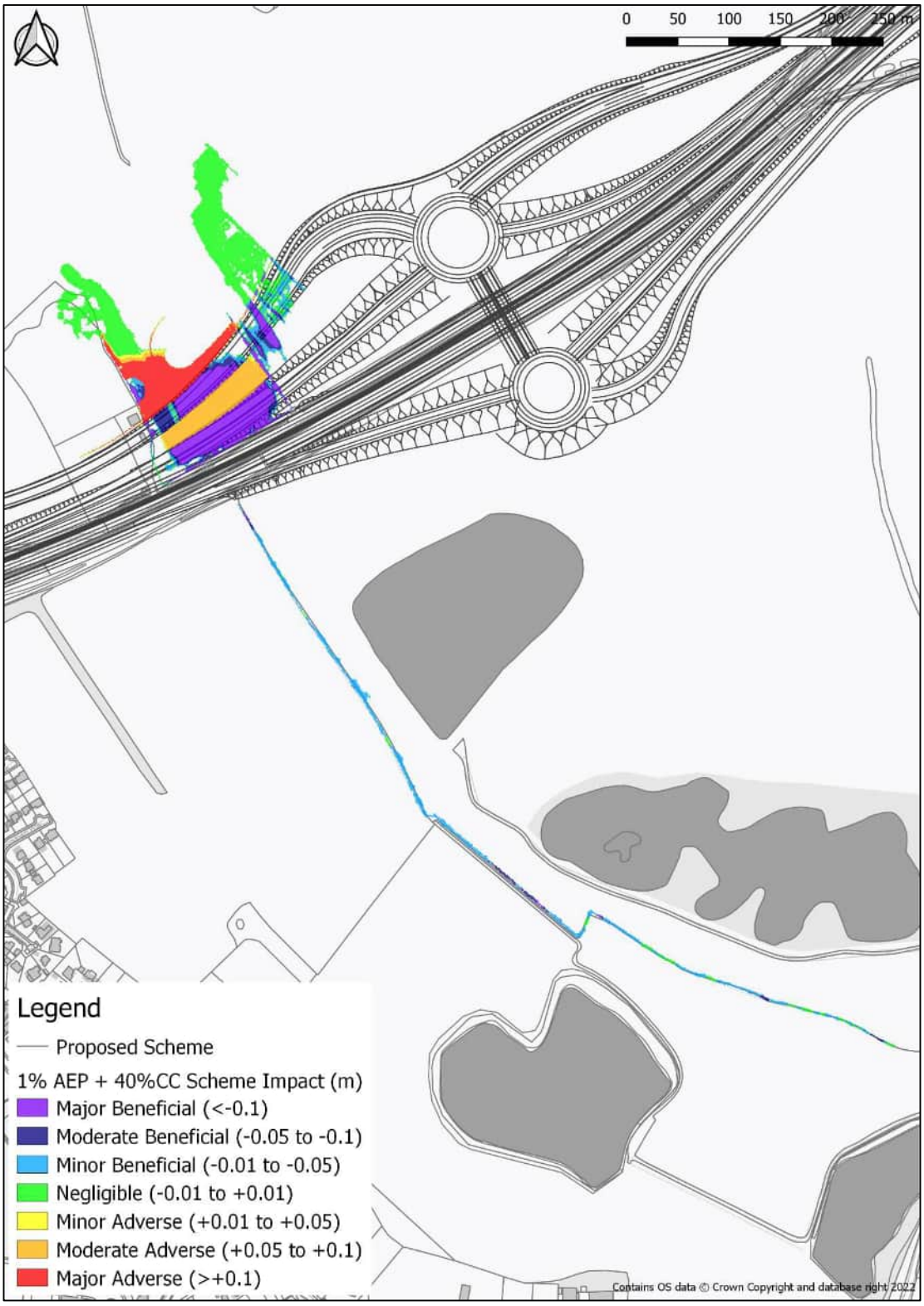
Plate C.4 Water level difference (with-scheme - baseline) for the 5% AEP event



Plate C.5 Water level difference (with-scheme - baseline) for the 1% AEP event



Plate C.6 Water level difference (with-scheme - baseline) for the 1% AEP + 40%CC event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 7 HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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1 Executive Summary

1.1.1 This document has been prepared to support the A12 Minor Watercourse 7 with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation for the Minor Watercourse 7 (MWC7) catchment which crosses the A12. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future.

3 Summary of assessment

3.1 Summary

3.1.1 A summary of the key information contained within the detailed hydrological assessment of MWC7 is provided in the table below.

Catchment location	Minor watercourse 7 (MWC7) is located in South East England, just outside Hatfield Peverel and crosses the A12 at approximately TL 79865 12176. This minor watercourse discharges to the River Blackwater approximately 3km downstream of the A12 road crossing.
Purpose of study and scope	The A12 is proposed to be upgraded and therefore peak flows and inflow hydrographs are required for the numerical hydraulic modelling of this watercourse. Flows have been derived for the following Annual Exceedance Probability (AEP) events: 5%, 1% and 1% plus climate change (plus cc), equivalent to the 20-year, 100-year and 100-year plus cc return period events respectively.
Key catchment features	The minor watercourse has a rural catchment with land use being predominantly agricultural. There is one reservoir located downstream of the A12 road crossing within the model extent catchment area.
Flooding mechanisms	Peak flows / volumes.
Gauged / ungauged	Ungauged.
Final choice of method	ReFH2.3
Key limitations / uncertainties in results	Due to the small size of this ungauged watercourse the catchment was not able to be picked up at the downstream model extent by the FEH Web Service but was picked up just downstream of this location at TL 80450 11700.

3.2 Note on flood frequencies

3.2.1 The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

3.2.2 Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval.

3.2.3 The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Table 3.1 Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

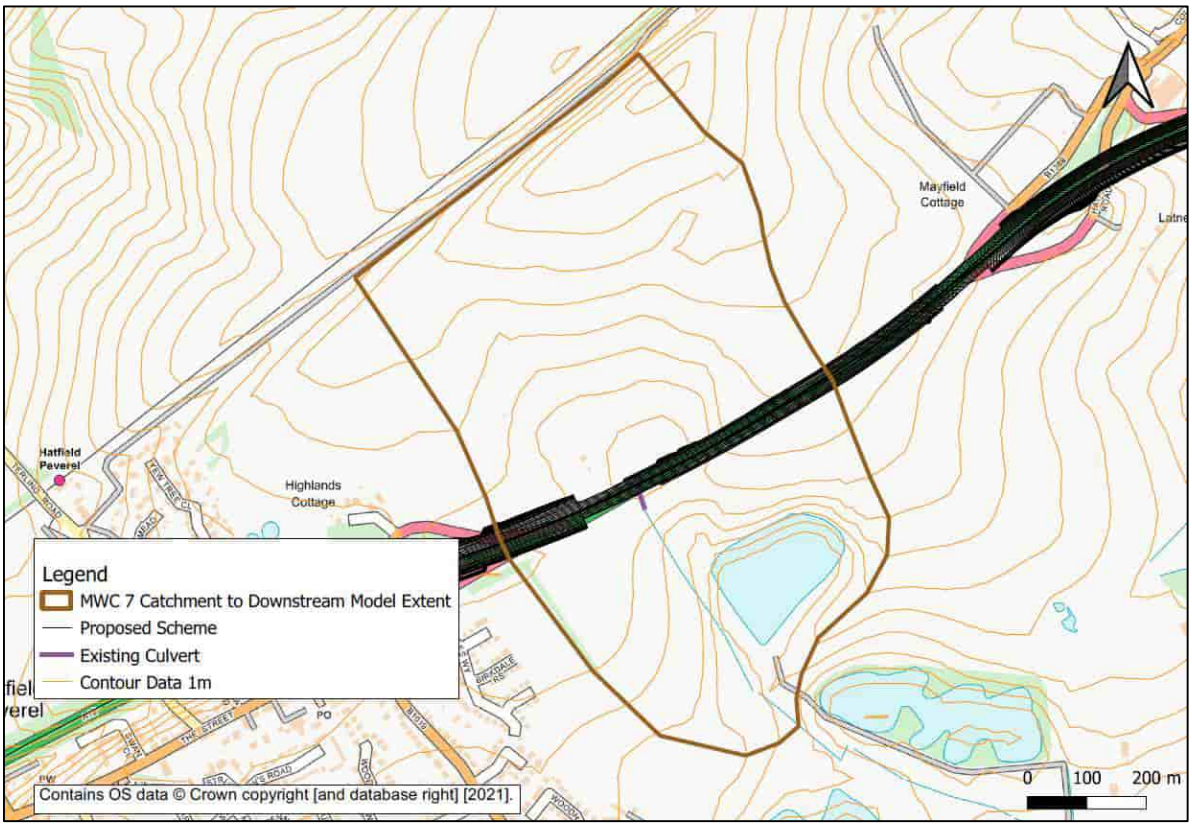
4 Method statement

4.1 Requirements for flood estimates

Overview	The proposed A12 upgrade route crosses Minor Watercourse 7 (MWC7) and therefore peak flows and hydrographs are required for the hydraulic modelling of this watercourse. Design peak flows and hydrographs are required for the following AEP events: 5%, 1% and 1% plus climate change (plus cc). This is equivalent to the 20-year, 100-year and 100-year plus cc return period events. A climate change uplift factor of plus 40% will be applied for the 1% AEP (100-year return period) event. This is based on current Environment Agency guidance for small catchments[1] (peak rainfall allowance, upper end, 2080s scenario). MWC7 is a small and ungauged watercourse. For hydraulic modelling purposes one flow estimation point has been adopted at the downstream modelling extent due to the small catchment size. The estimated total flow will be split by applying 61% of the flow as a point inflow upstream of the A12 road crossing and the remaining 39% of the flow will be applied as a lateral inflow downstream of the A12 crossing.
Project scope	This is a simple hydrological study for one minor watercourse with a catchment area up to the downstream model extent of approximately 0.6km^2. Therefore, ReFH model parameter calibration for this ungauged catchment is outwith the scope of this project.

[1] Environment Agency (2021). Flood risk assessments: climate change allowances guidance.

4.2 The catchment



Description	As shown in the above figure, Minor Watercourse 7 has a small catchment area. Approximately 61% of the total catchment area up to the downstream model extent is located upstream of the A12 crossing. MWC7 discharges to the River Blackwater approximately 3km downstream of the A12 crossing. The catchment is classified as essentially rural with land use being predominantly agricultural. There is a reservoir located in the lower catchment located downstream of the A12 crossing. The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 411d (Hanslope) within the upper and eastern reaches of the catchment and 571z (Hamble 2) within the middle and lower reaches of the catchment. The soils within the catchment were also assessed using Soilscapes[2], which indicates that there are freely draining slightly acid loamy soils in the lower and western side of the catchment with lime-rich loamy and

[2] http://www.landis.org.uk/soilscapes/

	clayey soils with impeded drainage present within the upper and eastern areas of the catchment. According to the Geology of Britain Viewer[3] the geology of the catchment is composed of London Clay Formations consisting of clay, silt and sand with superficial deposits being composed of clay, silt and Lowestoft Formation (Diamicton).

4.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released 24th September 2020. This contains provisional data up to water year 2019-20.

4.4 Gauging stations (flow or level)

Watercourse	Station name	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
River Blackwater	Blackwater @ Appleford Bridge	37010	247.3	Double throated trapezoidal flume and high flow rated spillway.	1962 to present
River Ter	Ter @ Crabbs Bridge	37003	77.8	Theoretically rated	1932 to present
River Brain	Brain @ Guithavon Valley	37009	60.7	Essex profile weir with throat tapping. Secondary instrument installed in 2003 to record non-modular flows.	1962 to present

4.5 Data available at each flow gauging station

[3] geology from the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
Blackwater @ Appleford Bridge	1962 to present	N	Yes	Yes	N	Groundwater influenced regime. Intermittently affected by the Ely / Ouse transfer scheme.
Ter @ Crabbs Bridge	1963 to present	N	Yes	No	N	Recently removed from suitable for pooling category. This was noted as due to many AMAX events being out of bank and this is not accounted for in rating.
Brain @ Guithavon Valley	1962 to present	N	Yes	Yes	N	No POT data are presented for this station as they have no non-modular flow correction applied. AMAX data have been corrected. Full period of record peak flow data reviewed and released in September 2019 (WINFAP Files v8).

4.6 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings	N/A			Not required for this assessment.
Historical flood data	N/A			Not Available
Flow or river level data for events	N/A			Not available for this assessment.
Rainfall data for events	N/A			Not required for this assessment.
Potential evaporation data	N/A			Not required for this assessment.
Results from previous studies	N/A			Not available for this assessment.
Other data or information	N/A			Not available for this assessment.

4.7 Hydrological understanding of catchment

Conceptual model	Hydraulic modelling of MWC7 is required to assess potential impacts of upgrading the A12. The target location is the A12 crossing of MWC7. Flooding is likely to be caused by peak flows/volumes at the A12 crossing location.
Unusual catchment features	The catchment is a small rural catchment located in South East England.

4.8 Initial choice of approach

Is FEH appropriate?	Yes, FEH methods are appropriate.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	ReFH2.3 and FEH Statistical methods have been applied to derive the peak flow estimates and hydrographs for this catchment. The hydrographs have been derived using ReFH2.3 method. Due to the small catchment size one flow estimate has been derived for each of the modelled AEP events. The flow estimates have been derived to the downstream model extent which is located approximately 0.5km downstream of the A12 crossing. The peak flow estimates have been area scaled within the hydraulic model. 61% of the target flow has been applied as a point inflow at the start of the hydraulic model with the remaining 39% of the flow being applied as a residual lateral inflow downstream of the A12 crossing.
Software to be used (with version numbers)	FEH Web Service[4] / WINFAP 4[5] / ReFH2.3

4.9 Locations where flood estimates required

4.9.1 The table below lists the location of the subject site. The site code listed below are used in all subsequent tables to save space.

[4] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

[5] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.10 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
DS Model Extent	L	MWC7	Catchment to downstream model extent.	580134	211794	-	0.588
US A12	S	MWC7	Catchment upstream of the A12.	579861	212192	-	0.360*
DS A12	S	MWC7	Catchment downstream of the A12.	579981	211980	-	0.227*
*For area scaling of design peak flows only							

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required. Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced. The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.							



4.11 Important catchment descriptors at each subject site (incorporating any changes made)

4.11.1 The FEH Web Service does not pick up the MWC7 catchment until approximately 350m downstream of the model extent. These FEH catchment descriptors (derived to TL 80450 11700) were therefore adopted for MWC7 after a visual check on catchment descriptors such as FARL.

Site code	FARL	PROPWET	BFIHOST	BFIHOST 19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
Donor	1	0.25	0.465	0.443	0.75	10.46	557	0	0.174
DS Model Extent	1	0.25	0.465	0.443	0.75	10.46	557	0	0.174

4.12 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	The catchment areas for this watercourse were assessed using 1m contour data and LIDAR data.
Record how other catchment descriptors were checked and describe any changes.	Donor FEH catchment descriptors were adopted from a location on MWC7 approximately 350m downstream of the hydraulic model extent. Key catchment descriptors such as FARL and soils were checked using mapping data.
Source of URBEXT	URBEXT1990 and URBEXT2000 calculated using URBAN50K method.
Method for updating of URBEXT	URBEXT calculated using URBAN50K.

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	The FEH Statistical method was used as a comparison against ReFH2.3 peak flow estimates.

5.2 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjust-ment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor			
						Weight	Weighted ave. adjustment		
DS Model Extent	0.094	CDs	-	-	-			1.000	0.094

Are the values of QMED spatially consistent?	N/A
Method used for urban adjustment for subject and donor sites	N/A - URBEXT calculated using URBAN50K.

Parameters used for WINFAP v4 urban adjustment if applicable

Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010) in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016).

5.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Donor gauges were assessed for the purposes of QMED adjustment; however, no donor was selected. The River Blackwater @ Appleford Bridge gauge (37010) was assessed, however, the River Blackwater @ Appleford Bridge gauges a much larger catchment with groundwater influence in the upper reaches of the catchment. This gauge was therefore assessed as not suitable as a donor for this small ungauged catchment. The River Ter @ Crabbs Bridge (37003) and River Brain @ Guithavon Valley (37009) gauges were also assessed as potential donors for QMED adjustment, but these gauges were both found to be unsuitable due to them having much larger catchment areas and varying catchment descriptors to the target site.

5.4 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)

5.5 Derivation of pooling groups

5.5.1 FEH Statistical Pooling was undertaken on MWC7 approximately 350m downstream of the model extent where the catchment can be picked up by the FEH Web Service at TL 80450 11700.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
Final Pooling MWC7 DS Donor	DS Donor	N	Removed 27073 / 26016 (high BFIHOST) 47022 / 106002 (low FARL) / 49005 (discordant / relatively high BFIHOST). Added 27010 / 49003.	L-CV: 0.209 L-Skew: 0.279

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

5.6 Derivation of flood growth curves at subject sites

Site code	Method	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
DS Donor	P	Final Pooling MWC7 DS Donor	GL - distribution gives an acceptable fit.	Urban adjustment	Location: 1.000 Scale: 0.202 Shape: -0.279	2.884
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis Urban adjustments are all carried out using the method of Kjeldsen (2010). Growth curves were derived using the procedures from Science Report SC050050 (2008).						

5.7 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	25	50	100	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events									
	50	20	10	5	4	2	1	0.5	0.2	0.1
DS Model Extent	0.094	0.126	0.151	0.181	0.191	0.227	0.271	0.324	0.411	0.493

6 Revitalised flood hydrograph 2 (ReFH2) method

6.1 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 has been applied to derive both peak flow estimates and hydrographs for the target watercourse at the downstream model extent for input into the hydraulic model.

6.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp}	BL (hours)	BR
DS Model Extent	CD	3.74	-	375.8	-	33.5	2.13
Brief description of any flood event analysis carried out							
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

6.3 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
DS Model Extent	Rural	Winter	5.5

6.4 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
DS Model Extent	Winter	5.5	0.588	Storm duration based on catchment area to downstream model extent.

6.5 Flood estimates from the ReFH2 method

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
DS Model Extent	0.144	0.202	0.241	0.281	0.295	0.340	0.396	0.575	0.481	0.656	0.806

7 Discussion and summary of results

7.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak			
	Return period 20-years / 5% AEP		Return period 100-years / 1% AEP	
	FEH Statistical	ReFH2.3	FEH Statistical	ReFH2.3
DS Model Extent	0.181	0.281	0.271	0.396

7.2 Final choice of method

Choice of method and reasons	The final choice of method is ReFH2.3 as it uses latest rainfall data (FEH13) and allows climate change uplift to be applied directly in the software.
How will the flows be applied to a hydraulic model?	The ReFH2.3 design hydrographs will be applied in the hydraulic model. For this assessment one flow estimation point has been adopted at the downstream modelling extent (DS Model Extent) due to the small catchment size. The ReFH2.3 flows will be split within the hydraulic model. 61% of the flow will be applied as a point inflow at the start of the hydraulic model upstream of the A12 road crossing. The remaining 39% of the flow will be applied as lateral flow downstream of the A12 road crossing between the A12 and the model extent.

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are as follows: • ReFH2.3 is most appropriate for the target catchment. • The reservoir downstream of the A12 crossing is offline.
Discuss any particular limitations	Small ungauged catchment.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	The ReFH2.3 method has been used to derive design peak flows and hydrographs. This method is appropriate but there is inherent uncertainty regarding using default model parameters. The standard approach in this situation is to use default model parameters. If the catchment was gauged the ReFH parameters could be adjusted to reduce the uncertainty.
Comment on the suitability of the results for future studies	The design peak flows have been estimated for the purposes of the A12 road improvements project only with

	the target location being the A12 crossing. Future studies should assess the appropriateness of these peak flow estimates / hydrographs for the purposes of their investigation.
Give any other comments on the study	None

7.4 Checks

Are the results consistent, for example at confluences?	N/A
What do the results imply regarding the return periods / frequency of floods during the period of record?	N/A ungauged catchment.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The 1 % AEP growth factor is 2.884 for the FEH Statistical method and 2.745 for the ReFH2.3 method. This is within the expected range.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the 0.1% AEP / 1% AEP is 1.820 for the FEH Statistical method and 2.039 for the ReFH2.3 method.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No other studies were available for this minor watercourse.
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	N/A

7.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
DS Model Extent	0.144	0.202	0.241	0.281	0.295	0.340	0.396	0.575	0.481	0.656	0.806

7.6 Uncertainty bounds

7.6.1 The Flood Estimation Guidelines 197_08 (2020) notes that it is more difficult to quantify uncertainty in design flows estimated from the ReFH rainfall-runoff model than for the FEH Statistical method which it reports approximate uncertainty bounds for. It is therefore highlighted that the final flow estimates are open to uncertainty, but uncertainty bounds have not been provided for this assessment.

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hydrographs are required for the hydraulic modelling of MWC7. See: MWC7 ReFH2.3 Hydrographs 1

Acronyms

Abbreviation	Term
AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

References

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Annex A

A.1 Pooling Group Details

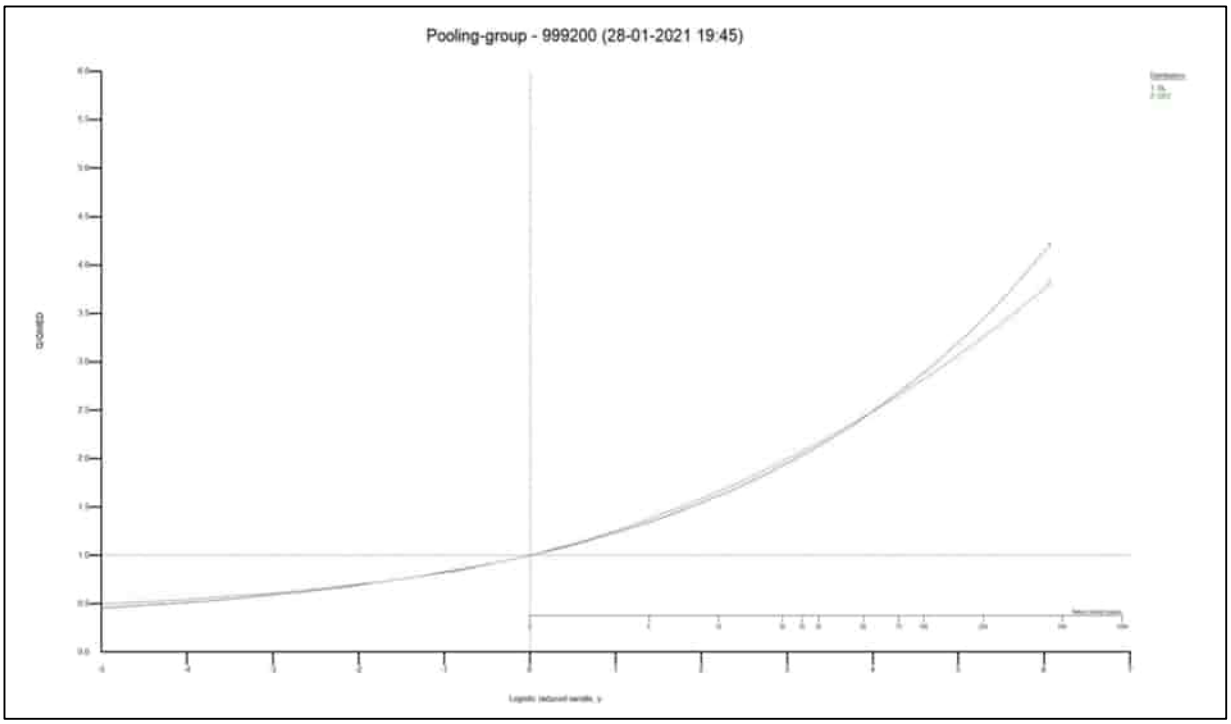
A.1.1 Initial Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
76011 (Coal Burn @ Coalburn)	2.317	42	1.84	0.163	0.301	0.621	1.63	1096	0.074	1	0
27073 (Brompton Beck @ Snainton Ings)	3.895	39	0.812	0.215	0.035	1.295	8.06	721	0.237	1	0.008
45816 (Haddeo @ Upton)	4.263	26	3.456	0.3	0.406	0.947	6.81	1210	0.011	1	0.005
27051 (Crimple @ Burn Bridge)	4.295	47	4.524	0.218	0.156	0.217	8.17	855	0.013	1	0.006
28033 (Dove @ Hollinsclough)	4.531	44	4.177	0.228	0.371	0.624	7.92	1346	0.007	1	0
91802 (Allt Leachdach @ Intake)	4.939	34	6.35	0.153	0.257	0.674	6.54	2554	0.003	0.992	0
25019 (Leven @ Easby)	5.039	41	5.09	0.342	0.386	1.588	15.09	830	0.019	1	0.004
26016 (Gypsey Race @ Kirby Grindalythe)	5.04	22	0.1	0.321	0.266	1.206	15.85	757	0.03	1	0
25003 (Trout Beck @ Moor House)	5.105	46	15.142	0.168	0.29	0.53	11.4	1905	0.041	1	0
71003 (Croasdale Beck @ Croasdale Flume)	5.109	37	10.9	0.212	0.323	0.274	10.71	1882	0.016	1	0
25011 (Langdon Beck @ Langdon)	5.138	33	15.647	0.232	0.328	0.703	12.79	1463	0.012	1	0.001
47022 (Tory Brook @ Newnham Park)	5.141	25	6.176	0.257	0.191	0.56	13.43	1403	0.023	0.942	0.014
106002 (Laxdale @ Laxdale)	5.174	12	17.449	0.103	0.132	1.16	10.64	1993	0.024	0.888	0
54022 (Severn @ Plynlimon Flume)	5.178	38	14.988	0.156	0.171	0.716	8.75	2481	0.01	1	0
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	5.193	9	5.777	0.271	0.151	2.877	16.08	1044	0.023	0.991	0.006
206006 (Annalong @ Recorder)	5.356	48	15.33	0.189	0.052	2.008	14.44	1704	0.023	0.981	0
Total		543									
Weighted means				0.219	0.243						

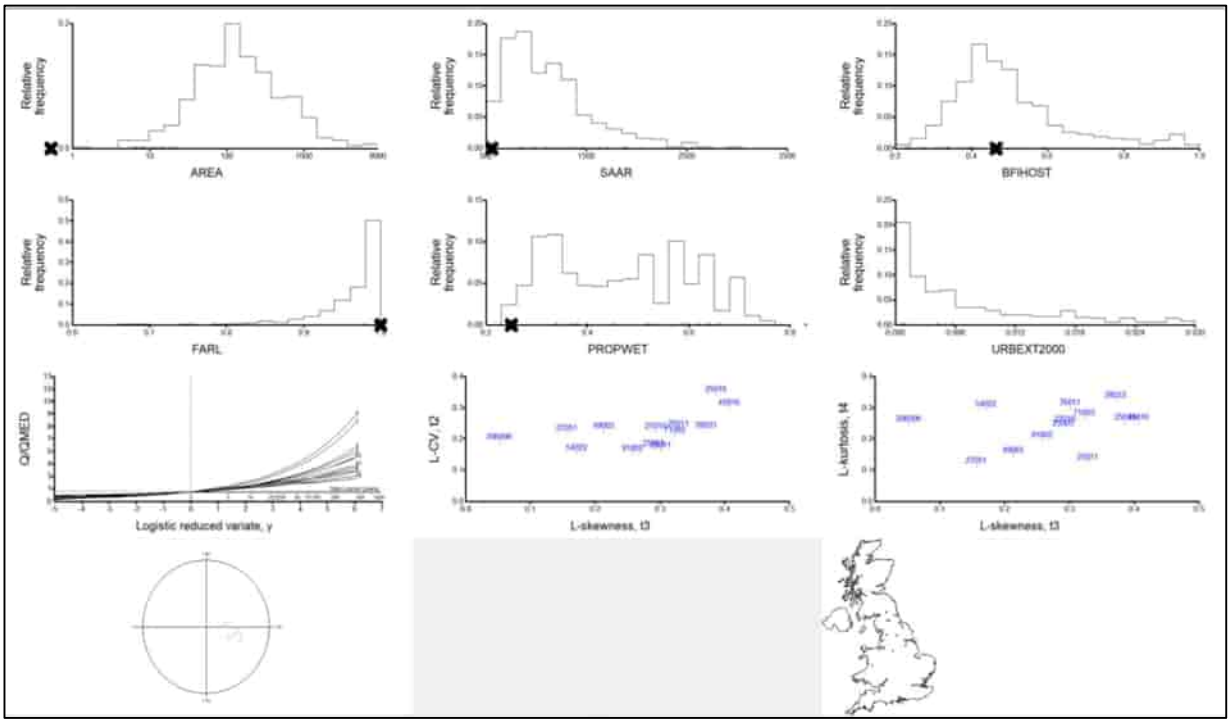
A.1.2 Final Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
76011 (Coal Burn @ Coalburn)	2.317	42	1.84	0.163	0.301	0.833	1.63	1096	0.074	1	0
45816 (Haddeo @ Upton)	4.263	26	3.456	0.3	0.406	0.968	6.81	1210	0.011	1	0.005
27051 (Crimple @ Burn Bridge)	4.295	47	4.524	0.218	0.156	1.359	8.17	855	0.013	1	0.006
28033 (Dove @ Hollinsclough)	4.531	44	4.177	0.228	0.371	0.833	7.92	1346	0.007	1	0
91802 (Allt Leachdach @ Intake)	4.939	34	6.35	0.153	0.257	0.845	6.54	2554	0.003	0.992	0
25019 (Leven @ Easby)	5.039	41	5.09	0.342	0.386	1.996	15.09	830	0.019	1	0.004
25003 (Trout Beck @ Moor House)	5.105	46	15.142	0.168	0.29	0.529	11.4	1905	0.041	1	0
71003 (Croasdale Beck @ Croasdale Flume)	5.109	37	10.9	0.212	0.323	0.189	10.71	1882	0.016	1	0
25011 (Langdon Beck @ Langdon)	5.138	33	15.647	0.232	0.328	1.364	12.79	1463	0.012	1	0.001
54022 (Severn @ Plynlimon Flume)	5.178	38	14.988	0.156	0.171	0.984	8.75	2481	0.01	1	0
206006 (Annalong @ Recorder)	5.356	48	15.33	0.189	0.052	2.44	14.44	1704	0.023	0.981	0
27010 (Hodge Beck @ Bransdale Weir)	5.422	41	9.42	0.224	0.293	0.029	18.82	987	0.009	1	0.001
49003 (de Lank @ de Lank)	5.701	53	13.646	0.225	0.213	0.63	21.61	1628	0.064	0.998	0
Total		530									
Weighted means				0.215	0.27						

A.2 Final Growth Curve



A.3 Diagnostic Plots



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 21A MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

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0 Executive Summary

0.1.1 This report describes the hydraulic modelling and the results obtained for the existing and proposed A12 highway crossing at Minor Watercourse 21A (MWC21A), NGR grid reference TL871184. The modelling and hydrology analyses have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 2-dimensional (2D) type using TUFLOW software (BMT 2022) and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 40% climate change (CC). No flow reconciliation was required to be carried out for the design hydrology. No data was available for calibration, but the model was verified against the Environment Agency (EA) flood maps. The model was used to simulate baseline and with-scheme scenarios for the three design events mentioned.

0.1.3 The with-scheme scenario involved widening of the highway carriageway at MWC21A along with a new junction arrangement. This required a sequence of new culverts to convey water under the junction slip roads and across the main highway. Sensitivity analysis of roughness and inflows was carried out to further validate the results.

0.1.4 The baseline (existing) modelling scenario showed that the A12 carriageway is not at risk of flooding. The initial scheme arrangement caused flood risk to the scheme elements; therefore, mitigation works were required. The mitigation measures comprised of realignment of the east tributary towards a new culvert inlet, with the addition of an online bund and side spill. The headwall was also extended along the right bank of the west tributary to redirect flows towards the inlet. In addition, a toe drain and bund were added on the left bank of the west tributary to direct floodplain flows back into channel. These mitigation works were effective in preventing flooding of the proposed-scheme. The mitigation design was also demonstrated to produce negligible flood risk impact to the watercourse downstream from the scheme.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA, which took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios, both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) were:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Minor Watercourses 7, 21, 21A (this report), 23, 26

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out to assess the baseline and with-scheme scenarios for an unnamed tributary of the River Blackwater. The watercourse is designated as Minor Watercourse 21A based on the A12 drainage design numbering system for the main culvert crossings of the highway. This is a technical report, focused on the hydraulic modelling and catchment hydrology, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 At the A12 crossing of Minor Watercourse 21A (MWC21A), the highway will be upgraded to include slip roads leading towards a new junction. This will require minor widening of the highway embankment on both sides and

replacement of the existing crossing. Full details of the proposed-scheme arrangements are given in Section 7.

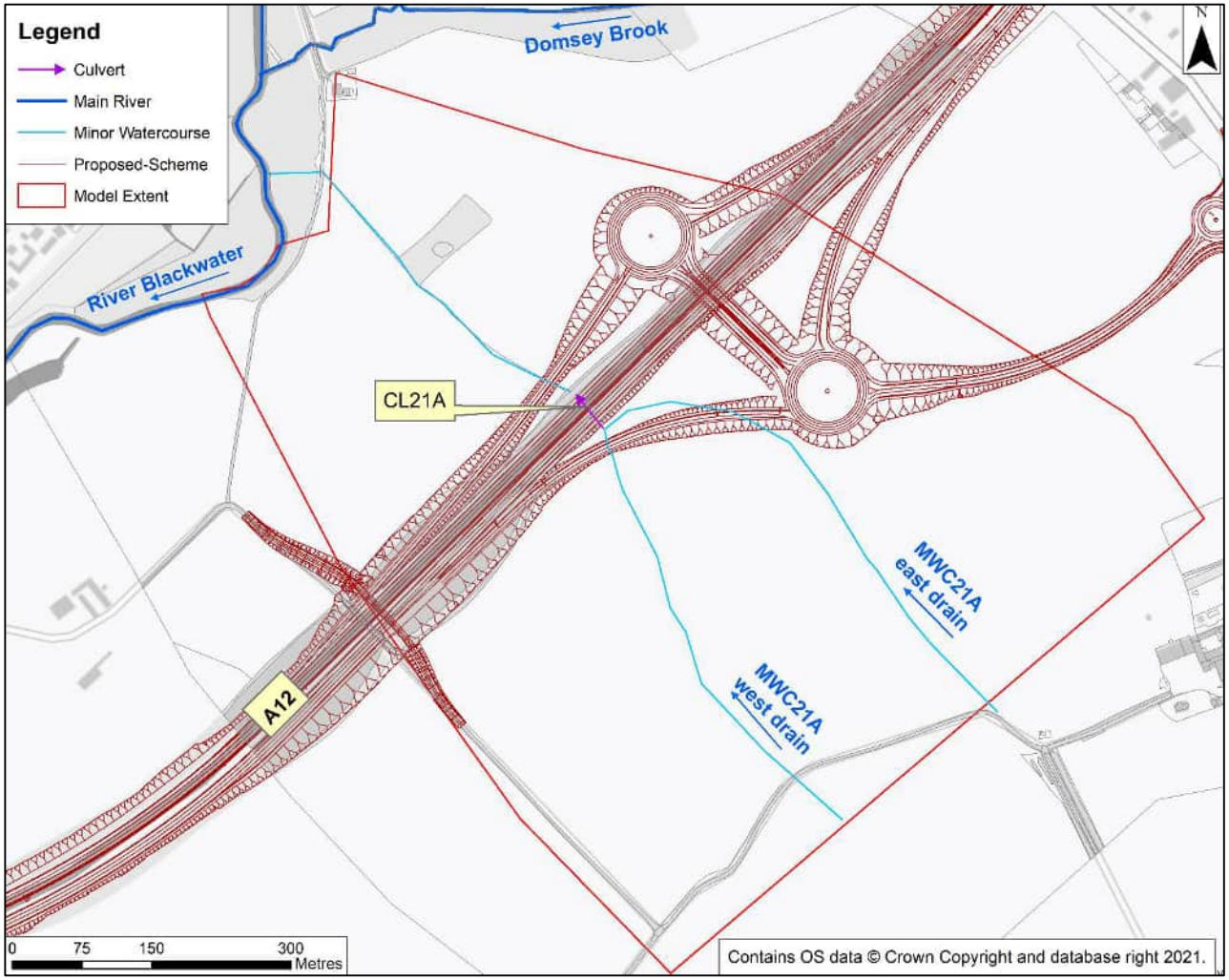
1.1.6 This report should be read in conjunction with the FRA [R10060/APP/6.6] (National Highways 2022).

1.2 Study area

1.2.1 MWC21A is situated in the southeast of Kelvedon, Essex. It crosses the A12 at approximately NGR: TL 87121 18444. The watercourse has two distinct drainage paths originating approximately 500m upstream (south) of the A12, flowing northwest (see Plate 1.1). The east tributary runs for a length of 360m before the channel and bank tops flatten and the channel is no longer defined in the topography. From this point, the natural flow path curves to the west, following the natural contours of the land. The two tributaries converge immediately upstream of the A12, where a 900mm culvert (CL21A in Plate 1.1) conveys the flow under the highway. The channel continues its course northwest for approximately 400m before discharging into the River Blackwater, just south of its confluence with Domsey Brook.

1.2.2 The model extent covers an area of 0.55km^2 and includes a total reach length of approximately 1425m.

Plate 1.1 MWC21A study area



2 Input data

2.1.1 The data used to construct the hydraulic model for MWC21A is summarised in Table 2.1.

Table 2.1 MWC21A hydraulic model inputs

Data	Description	Source
LiDAR	1m resolution composite Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) 2019 data. Used to inform the hydraulic model with ground level information.	Department for Environment, Food & Rural Affairs (DEFRA), 2019
Ordnance Survey (OS) Mastermap	Land use data used to specify roughness values across the 2D floodplain.	National Highways
Hydrological Analysis	Calculation of inflow hydrographs into the model.	National Highways, 2020
As-built survey	Survey of key hydraulic structures in model	MKSurveys December 2019
Outline Design Drawings	Drawings of the proposed-scheme design.	National Highways, 2021
Flood Zone Mapping	EA Risk of Flooding Surface Water (RoFSW) flood maps. Used for model verification.	Environment Agency

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of MWC21A for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event is also required including an allowance for climate change. For full details of the flow derivation methods and background hydrology refer to Appendix A Flood Risk Assessment Minor Watercourse 21A Hydrology Annex (HE551497-JAC-EWE-SCHW-RP-LE-0086).

3.1.2 Hydrological considerations

3.1.3 MWC21A has a small catchment area of approximately 0.75km^2 to the River Blackwater confluence. The catchment is classified as essentially rural with a few dwellings present throughout the catchment. Land use is predominantly agricultural. There is also a solar farm within the upper reaches of the catchment (accounting for approximately 8% of the total catchment area).

3.1.4 The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 813b (Fladbury 1) near the River Blackwater, 571s (Efford 1) and 572q (Ashley) in the middle reaches, with 712c (Windsor) being present in the upper reaches of the catchment. The HOST class of the soil groups were assessed and soil type 712c was found to contain HOST class 23 and 25. There are noted issues within the Flood Estimation Guidelines with BFIHOST values for soil which contains HOST 23 and 25 which has been corrected for BFIHOST19. The geology of the catchment was assessed using the Geology of Britain Viewer and found to be composed of London Clay formations consisting of clay, silt and sand.

3.2 Peak flow derivation

3.2.1 MWC21A is a small ungauged catchment and therefore flow estimation is open to great uncertainty. Both ReFH2.3 and FEH Statistical methods were therefore applied to estimate design peak flows for this watercourse. ReFH2.3 resulted in higher design peak flow estimates for MWC21A than the FEH Statistical method. Given ReFH2.3 is more conservative and as the FEH Statistical method uses BFIHOST which has known issues where the soils contain HOST Class 23 and 25, the ReFH2.3 method was selected for final peak flows estimates.

3.3 Hydraulic model inflows

3.3.1 For the purposes of hydraulic modelling, the MWC21A catchment was split. Flow estimates were derived for the upper catchment to the A12

crossing (US A12) which is the target location. The flow estimates were then area scaled between the eastern and western tributaries within the hydraulic model. 56% of the flow estimated at the A12 crossing was applied to the east (right-hand side) tributary and the remaining 44% to the west (left-hand side) tributary. A residual hydrograph was also derived for the catchment area downstream of the road (between the A12 road crossing and the River Blackwater confluence) and applied as a point inflow within the hydraulic model.

3.3.2 A schematisation of the MWC21A catchment is presented in Plate 3.1.

3.3.3 The adopted final design peak flow estimates for the upper catchment to the A12 crossing (US A12), for the residual catchment between the A12 road and the Blackwater confluence (Residual) and for the total catchment area to the River Blackwater Confluence (DS A12) are presented in Table 3.1.

Plate 3.1 Schematisation of the MWC21A catchment

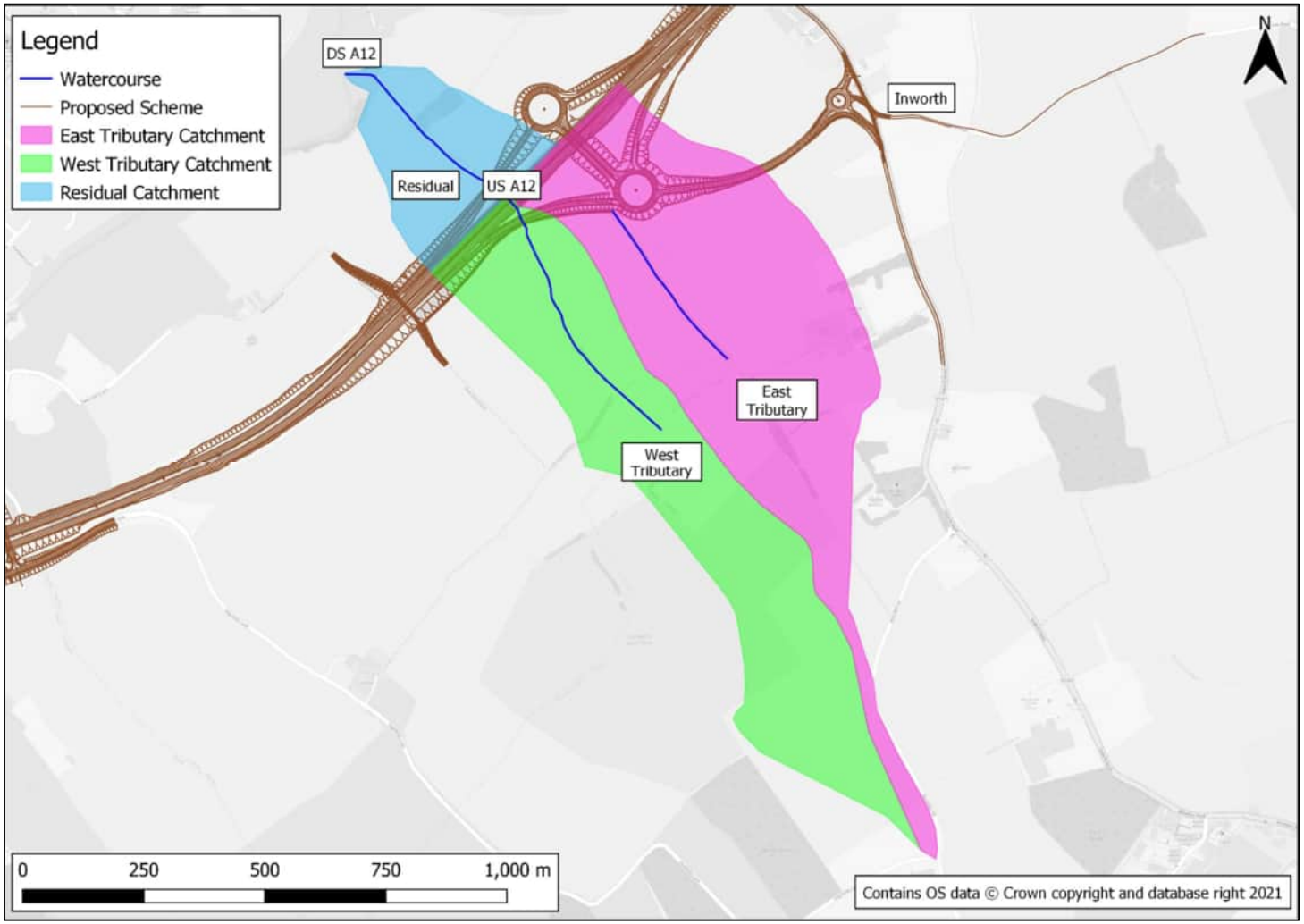


Table 3.1 MWC21A peak flow estimates (m^3/s) at the target location (US A12), for the residual catchment (Residual) and downstream model extent (DS A12)

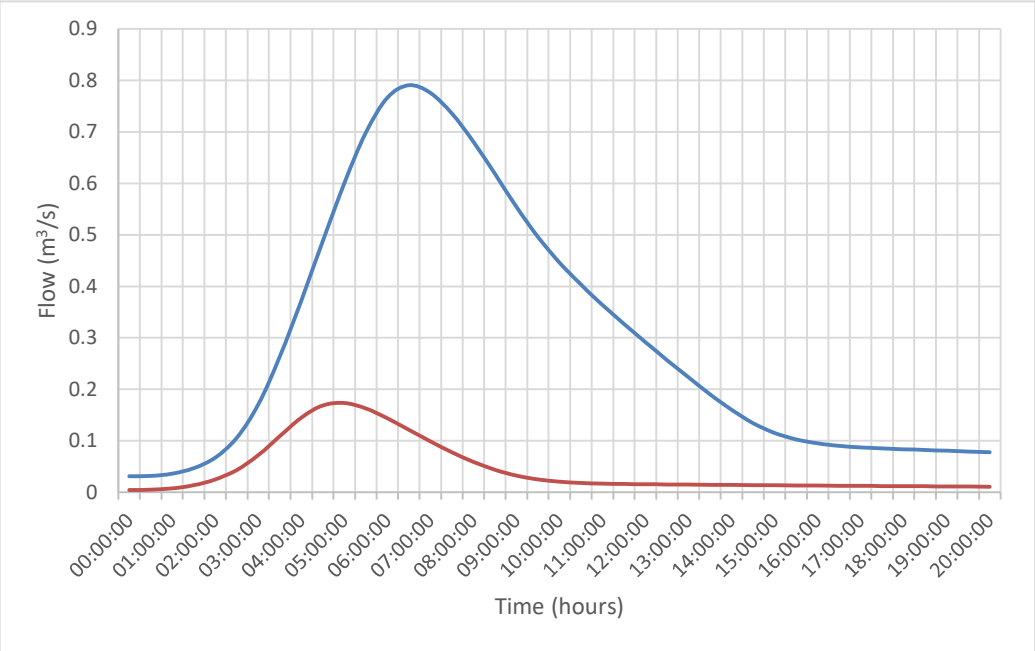
Inflow	Peak Flow (m^3/s)										
	AEP 50% (1 in 2)	AEP 20% (1 in 5)	AEP 10% (1 in 10)	AEP 5% (1 in 20)	AEP 3.33% (1 in 30)	AEP 2% (1 in 50)	AEP 1.33% (1 in 75)	AEP 1% (1 in 100)	AEP 1% +CC 40% uplift (1 in 100)	AEP 0.5% (1 in 200)	AEP 0.1% (1 in 1000)
US A12	0.30	0.41	0.49	0.57	0.62	0.68	0.74	0.79	1.14	0.95	1.58
Residual	0.06	0.09	0.11	0.12	0.14	0.15	0.16	0.17	0.25	0.21	0.35
DS A12	0.33	0.46	0.54	0.63	0.68	0.75	0.82	0.87	1.25	1.04	1.74

3.4 Inflow hydrograph shape

3.4.1 As the MWC21A catchment is ungauged it was assessed as appropriate to derive hydrograph shapes for input into the hydraulic model using ReFH2.3. The ReFH2.3 hydrograph shape was derived at the A12 crossing (the target location) and for the residual catchment downstream of the A12 and are based on a 5.5h event storm duration, which is equivalent to the critical storm duration for the catchment upstream of the target location (the A12 crossing).

3.4.2 The 1% (1 in 100) AEP event hydrographs for the upstream catchment to the A12 crossing (US A12) and residual catchment are presented in Plate 3.2.

Plate 3.2 1% (1 in 100) AEP event hydrographs



3.5 Climate change

3.5.1 A climate change allowance for peak rainfall intensity was obtained from the latest government guidance at the time of assessment (Environment Agency 2021). The peak rainfall intensity allowance is appropriate for small catchments (less than 5km^2). An uplift of 40% (the upper end, 2080s scenario) was therefore applied. The peak for the 40% uplift at the A12 crossing (US A12) is 1.142 m^3/s.

3.6 Flow reconciliation

3.6.1 Given the model inflows were estimated directly at the MWC21A A12 crossing (target location) using the ReFH2.3 method, flow reconciliation is not considered necessary for this minor watercourse.

3.7 Summary

3.7.1 Design peak flows and inflow hydrographs for MWC21A were derived using ReFH2.3 model, using the FEH catchment descriptors borrowed from a hydrologically similar neighbouring donor catchment and refining the key catchment descriptors for this watercourse. A donor catchment was selected due to the target catchment not being picked up by the FEH Web Service. Appropriate climate change allowance was applied as per the latest Environment Agency guidance. The design inflow hydrographs to be used in the numerical hydraulic model were derived for MWC21A using the critical storm duration to the target location, that is, at the A12 crossing of this watercourse.

4 Baseline modelling

4.1 Methodology

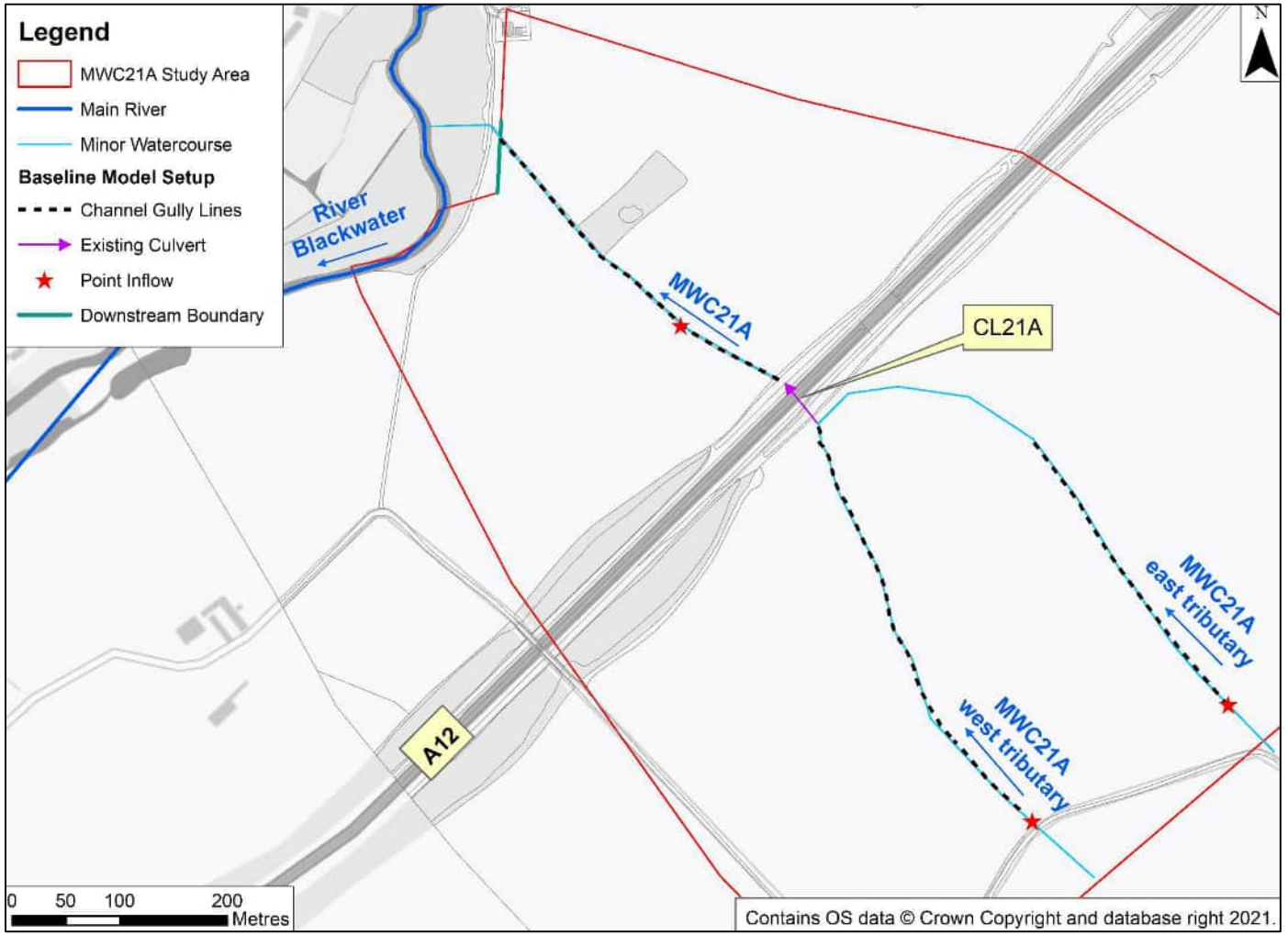
4.1.1 The MWC21 hydraulic model uses a two-dimensional (2D) schematisation, built in TUFLOW version 2020-10-AA (BMT, 2020). 1D 'ESTRY' elements were used to represent the culverts within the TUFLOW model and linked to the 2D domain via SX links. The 2D representation of the channel was considered appropriate as the highway culvert controls pass-forward flows and upstream water levels, therefore the detailed representation of the channel capacity is not required.

4.1.2 The model was built from scratch and used to simulate baseline, with-scheme and mitigation scenarios. Three design events were simulated for each scenario: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) AEP +40%CC.

4.2 Model extent

4.2.1 The model extent covers an area of 0.6 km^2. The model extent has been drawn in a way to capture the floodplains of each tributary and the downstream end of the watercourse. Representation of the model extent is shown in Plate 4.1, and model configuration is explained in detail in Section 4.3.

Plate 4.1 MWC21A TUFLOW model setup



4.3 Model resolution and topography

4.3.1 1m horizontal resolution LiDAR DTM data from 2019 was available for the model extent and was downloaded from the DEFRA website to inform the hydraulic model with ground level information.

4.3.2 A 1m grid cell resolution was deemed appropriate for the model, as this is sufficient to represent the open channel, the floodplain topography and the active floodplain details while avoiding a long simulation run time. The model grid has been oriented to aligned with key flow paths.

4.3.3 The open channel sections were represented in the model using 2m thick gully lines to define channel width, whereby the riverbed levels were reinforced using selected LiDAR data, based on inspection of the 1m LiDAR DTM and aerial photography. Bridge structures downstream of the A12 have been ignored, as investigation shows that these features would not affect the model results within the area of interest.

4.3.4 The culvert (CL21A) conveys flow from south to north of the A12 crossing and is the only structure modelled within the MWC21A study area. CL21A is a circular culvert with a surveyed diameter of 900mm and a length of 65m. CL21A was represented in the model using an embedded 1D 'ESTRY' element with a Manning's 'n' roughness value of 0.013, which is a mid-value for either metal or concrete culvert types (Chow 1959). Culvert losses are applied based on standard recommended values; 0.5 and 1 for the inlet and outlet loss, respectively.

4.4 Hydraulic friction

4.4.1 OS MasterMap data was used to identify land use type and inform the TUFLOW model with different hydraulic roughness (Manning's 'n' coefficient) values. Hydraulic roughness coefficients are applied over each grid cell of the 2D domain depending on the land use taken from the MasterMap data. Roughness values adopted were taken from standard guidance (Chow, 1959).

4.4.2 Depth-varying roughness has been used to stabilise the model solution at shallow depths and adopted values are shown in Table 4.1. For depths between 50mm and 150mm, Manning's 'n' value is interpolated between n1 and n2. Below 50mm, n1 is applied, and above 150mm, n2 is applied. The default interpolate method uses a curved fit so that the n values transition gradually.

4.4.3 As mentioned in Section 4.2.3 above, CL21A was modelled as having a roughness value of 0.013, taken from Chow (1959) as a mid-value for either concrete or metal culverts that are straight and free of debris.

Table 4.1 MWC21A roughness

Land Use	Depth varying Manning's 'n' roughness	
	n1	n2
Natural Surface	0.1	0.05
Manmade Surface	0.05	0.025
Property Gardens	0.1	0.05
Roads/Tracks/Paths	0.05	0.025
Manmade Structures	0.05	0.025
Slopes	0.1	0.05
Water	0.025	
Natural Land	0.1	
Buildings	1.0	

4.5 Boundary conditions

4.5.1 Two types of boundary conditions have been implemented in the model and can be seen in Plate 4.1:

- QT boundaries (TUFLOW ST type) were applied to the 2D domain at the upstream ends of the two tributaries. A lateral inflow was applied to the channel 110m downstream of the existing A12 crossing, at the location immediately downstream of the proposed new culvert outlet. This placement was selected in order for the baseline model to be consistent with the with-scheme scenario. The catchment hydrology analysis is explained in Section 3.

- A water level vs flow (TUFLOW HQ type) downstream boundary was implemented to the north of the model extent to allow channel flows out of the model and prevent 'glass walling' of water. The boundary applies a normal depth assumption with slope determined by inspection of the LiDAR data.

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 40% uplift in accordance with the latest Environment Agency guidance as detailed in Section 3.

5.1.2 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients and hydrological inflows (see Section 6.4).

Table 5.1 Modelled events for MWC21A

Scenario	5% AEP	1% AEP	1% AEP +40%CC
Baseline	✓	✓	✓
Design	✓	✓	✓
Mitigation	✓	✓	✓
Roughness Sensitivity		✓	
Hydrological Inflow Sensitivity		✓	

6 Model proving

6.1 Introduction

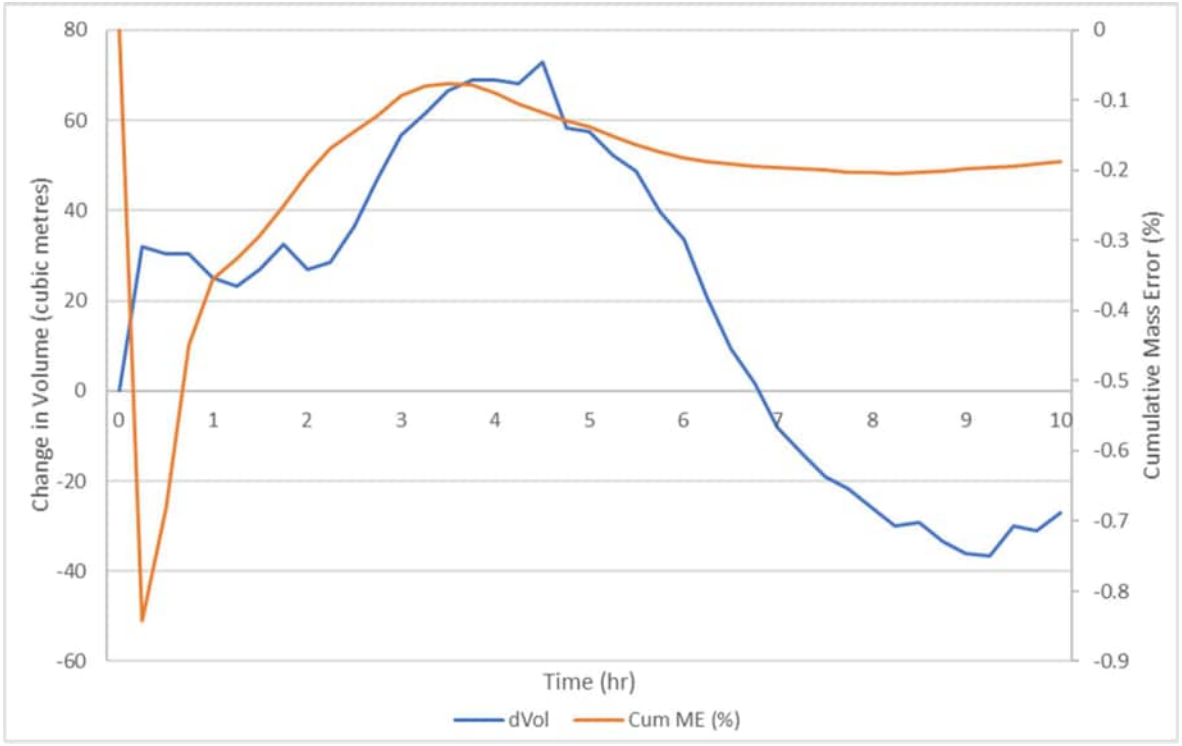
6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model computational performance was achieved. The cumulative mass error diagnostics outputs from the model have been checked for each model simulation. Plate 6.1 shows the cumulative mass error (Cum ME) and the change in volume (dVol) for the baseline 1% (1 in 100) AEP event simulation. This plot is typical for all the events and scenarios simulated. The tolerance range recommended by the software manual is +/-1% mass balance error.

6.2.2 The cumulative mass error tolerance is less than 1% error throughout the simulation; therefore the mass error is considered acceptable.

Plate 6.1 Model proving cumulative ME and dVol plot for 1% (1 in 100) AEP event



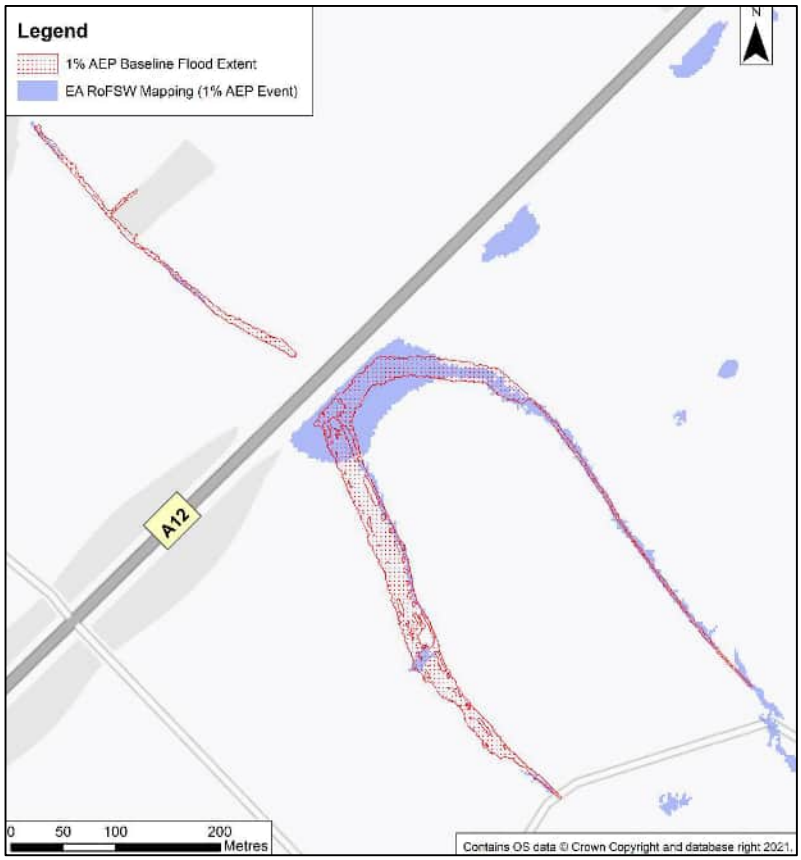
6.3 Comparison with Environment Agency flood maps

6.3.1 MWC21A has not been included in the Environment Agency (EA) published Flood Zone mapping of fluvial flood risk. Therefore, a direct comparison between model results and EA flood maps is not possible. However, the EA Risk of Flooding from Surface Water (RoFSW) mapping can be used to identify areas of pooling.

6.3.2 Plate 6.2 shows the modelled 1% (1 in 100) AEP event flood extent and the EA RoFSW mapping extents. The EA RoFSW flood map shows greater ponding against the southern side of the A12.

6.3.3 There is limited data available for verification of the MWC21A model, as the EA flood map shows a larger flood extent than the model where ponding occurs against the A12 embankment, because the existing culverts across the A12 have not been represented in the EA RoFSW mapping. Conversely, a smaller flood extent is observed with the EA flood map downstream of the A12 against the model. Considering this, verification of the model cannot be based on the comparison of results with EA flood maps. The analysis is therefore reliant on the use of industry standard modelling and hydrology tools, as is common for small watercourse studies.

Plate 6.2 Environment Agency risk of flooding from surface water and 1% (1 in 100) AEP extent



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness and hydrological inflows. All sensitivity tests were performed using the 1% (1 in 100) AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Appendix B show the impact of changing the model roughness on predicted peak water levels. The results show that the floodplain is relatively insensitive to changes to Manning's 'n', with a change in modelled water level of no more than +15mm or -20mm when the Manning's value is increased or decreased, respectively. The low sensitivity of the model to changes in roughness is due to the relatively steep terrain. The most sensitive area is upstream of the A12 embankment, where ground level flattens. Upstream and downstream of the A12, local spots in-channel, generally show moderate sensitivity to changes in roughness, with a maximum water level difference of +350mm and -450mm when roughness is increased and decreased, respectively. The change in roughness has no impact on the 1% (1 in 100) AEP flood extent.

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Appendix B show the impact of changing the model inflows on predicted peak water levels. The results show that the area surrounding the culvert inlet is sensitive to changes in flow, by +110mm and -100mm when flow is increased and decreased, respectively. Downstream of the A12, local spots in-channel, show major sensitivity to changes in flow by up to +120mm and -120mm when flow is increased and decreased, respectively. Decreasing the flow has almost no impact on the 1% (1 in 100) AEP flood extent but increasing the flow causes additional flooding local to the west tributary upstream of the A12.

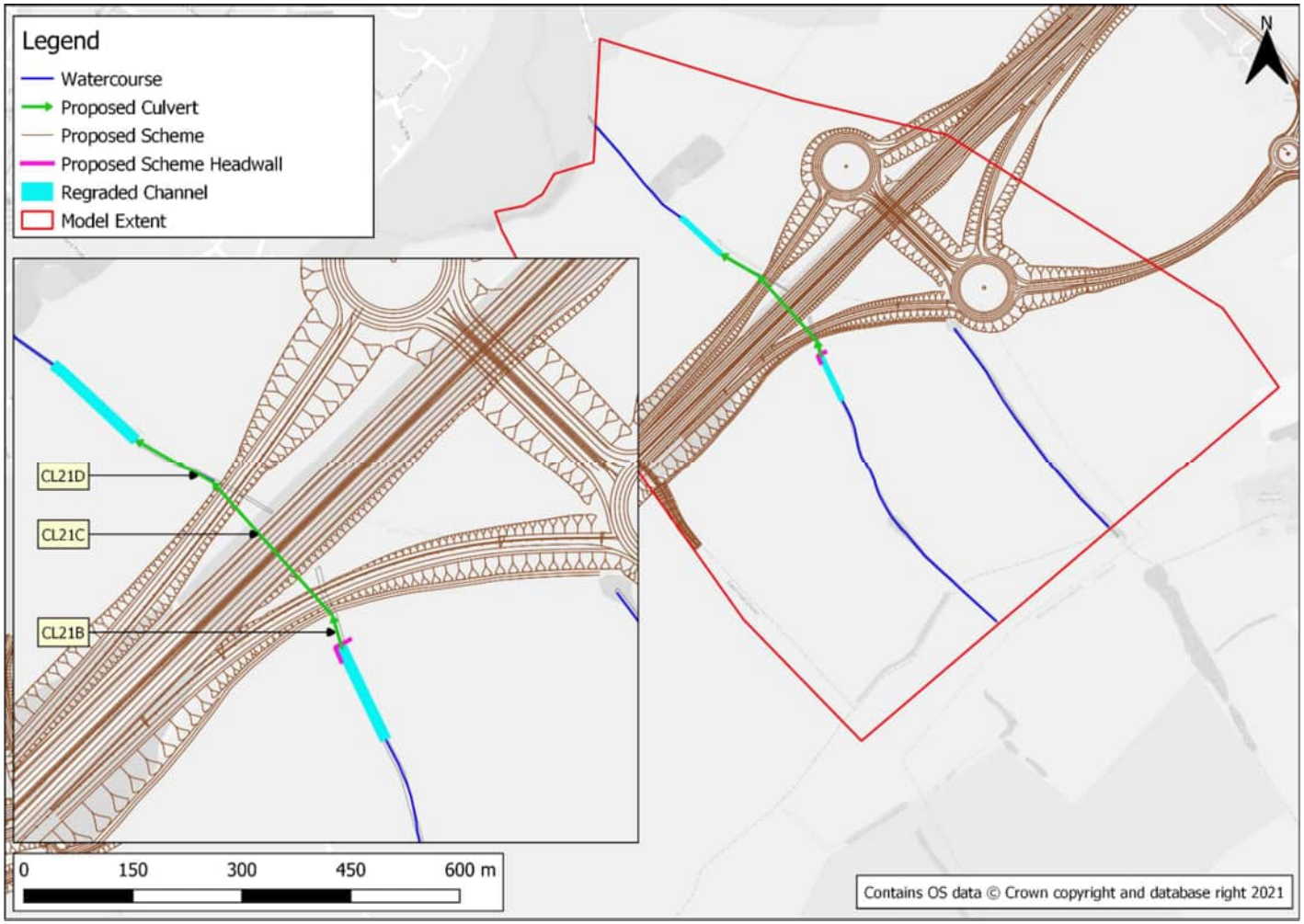
7 Design modelling

7.1 Design model build

7.1.1 The baseline model was updated to include the proposed-scheme. Changes to the model were as follows and illustrated in Plate 7.1:

- The proposed A12 side road design was included in the model as modification to the baseline model topography using an Ascii DTM surface overlain.
- CL21A culvert was abandoned and replaced with a realigned proposed crossing, which consists of a series of three culverts:
  - The upstream part of the crossing (culvert CL21B) lies approximately 30m upstream of the toe of the embankment of the proposed-scheme and consist of a circular concrete pipe of 1.2m diameter and a length of 31.5m. The pipe invert level of 30.36m AOD at the inlet was taken from the Lidar DTM. A L-shaped headwall was included at the inlet with an elevation of 32.5m AOD to avoid overtopping of the structure and to prevent spilling of the west tributary onto its left floodplain.
  - The main crossing (culvert CL21C) is a circular concrete culvert of 1.2m diameter and 210m length, it connects to the CL21B outlet via a 1.2m diameter cylindrical manhole chamber and conveys water underneath the proposed A12 carriageway from upstream to downstream of the proposed-scheme.
  - The downstream part of the crossing (culvert CL21D) connects to the CL21C outlet via a 1.2m diameter cylindrical manhole chamber and discharges water into MWC21A open channel approximately 70m downstream of the proposed-scheme. It is a circular, concrete culvert of 1.2m diameter and 60m length.
  - All three culverts were modelled using Manning's 'n' value of 0.013 for concrete, with recommended standard inlet and outlet losses of respectively 0.5 and 1.
- Manning's 'n' roughness layer was updated to reflect the additional road surface resulting from the proposed-scheme. Manning's 'n' values were kept the same as per Table 4.1.
- For 68m upstream of the proposed CL21B inlet and 74m downstream of the proposed CL21D outlet, the channels required regrading to be made steeper from a slope of 0.031 to a slope of 0.044. The model topography was modified to represent the newly excavated channels.

Plate 7.1 MWC21A proposed-scheme



8 Model results

8.1 Baseline results

8.1.1 Plate 8.1 shows the maximum flood extent for all modelled events for the baseline scenario. The flood extents for each of the modelled events can be found in Appendix C.

8.1.2 Where the east tributary is clearly defined in the topography of the landscape, water remains in-channel for all storm events.

8.1.3 The left floodplain of the west tributary is flooded along its entire length in the 1% (1 in 100) AEP and 1% (1 in 100) AEP +40%CC events.

8.1.4 Water ponds upstream of the A12 crossing where the east and west tributaries converge, near the CL21A inlet to depths of nearly 375mm during a 1% (1 in 100) AEP +40%CC event.

8.1.5 There is no flooding to the A12 highway for any AEP events. The culvert inlet becomes surcharged during a 1% (1 in 100) AEP +40%CC event. Downstream of the A12, flow remains in-channel for all storm events.

8.1.6 For the 1% (1 in 100) AEP event, CL21A culvert is not surcharged and runs at 76% capacity.

Plate 8.1 Baseline flood extents



8.2 With-scheme model results

8.2.1 As shown in Plate 8.2, the proposed-scheme is flooded in all events by the east tributary flowing directly into the proposed slip road, which lies 3.5m lower than the bed of the east tributary at the location where it enters the scheme. This impact to the proposed-scheme required the investigation of mitigation works to ensure no flood risk to the scheme.

8.2.2 Plate 8.3 shows the changes to the water level due to the proposed A12 scheme for the 1% (1 in 100) AEP +40%CC event. The maximum flood level difference between baseline and with-scheme are categorised into levels of impact due to the proposed-scheme. The impact of the proposed-scheme for all other flood events can be found in Appendix C.

8.2.3 For the 1% (1 in 100) AEP +40%CC event, CL21B is at 36% capacity. This is predominantly because it is not receiving any flows from the east tributary, and also due to the increase in pipe diameter which is 300mm wider than the existing culvert.

Plate 8.2 With-scheme flood extents

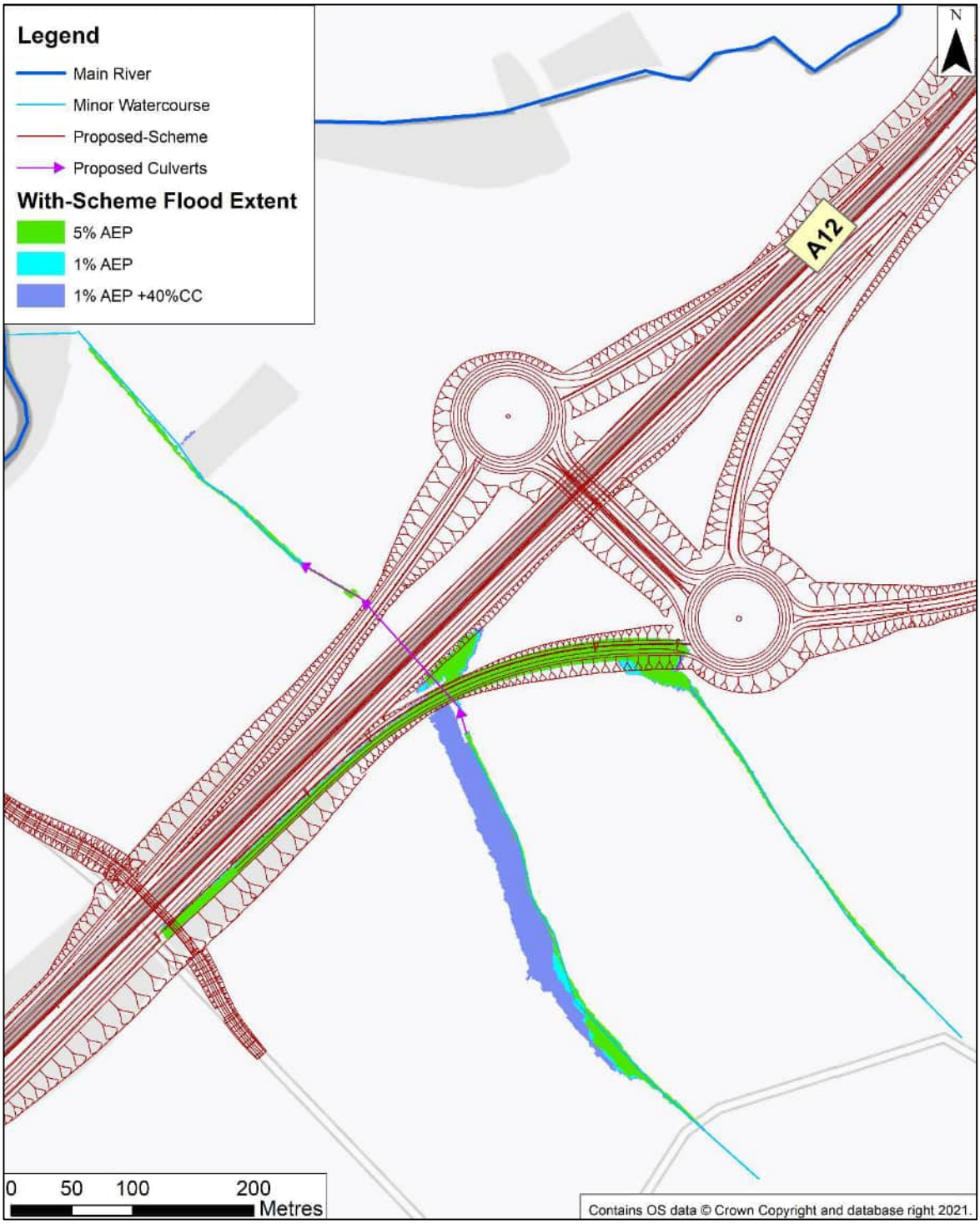
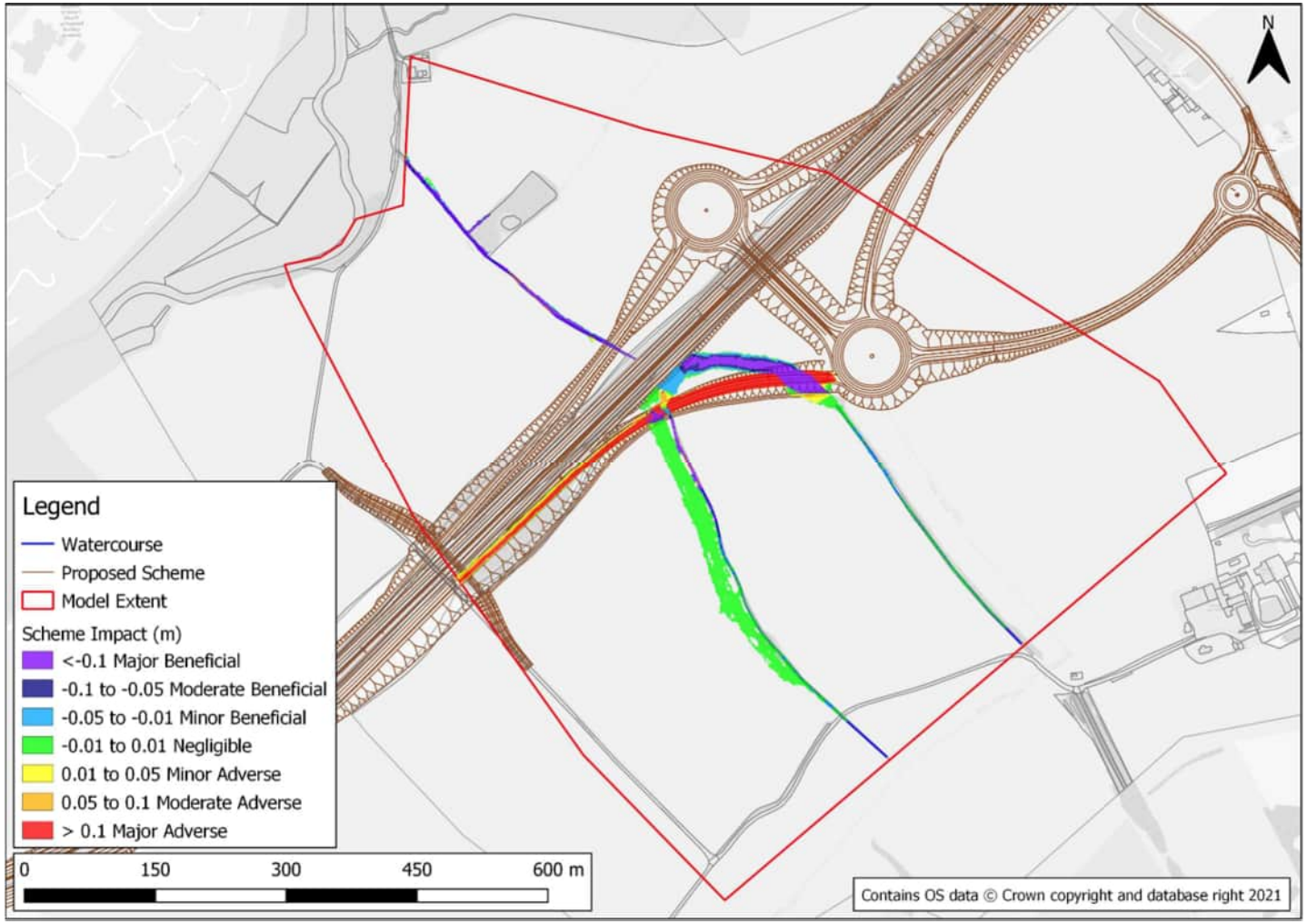


Plate 8.3 Water level difference (With-scheme minus baseline) for 1% (1 in 100) AEP +40%CC



9 Mitigation modelling

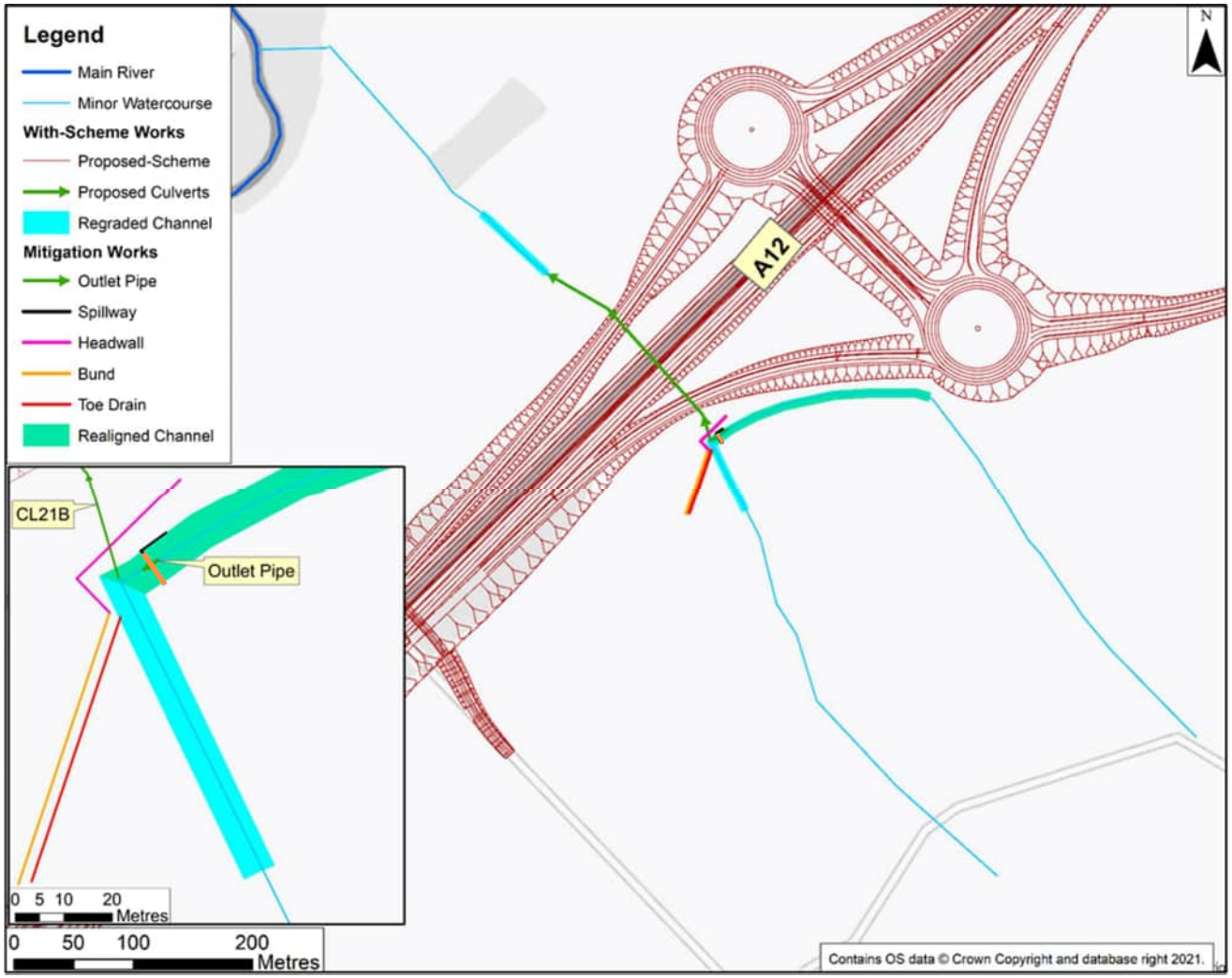
9.1 Mitigation model build

9.1.1 The with-scheme model was updated to include the mitigation measures to address the impacts resulting from the proposed-scheme.

9.1.2 Changes to the model were as follows and illustrated in Plate 9.1:

- A newly excavated channel was added to divert flows from the east tributary towards the inlet of CL21B, using an online bund at its downstream end for flow attenuation purposes, alongside with a 500mm diameter outflow pipe to allow the diverted channel to empty. The combination of the bund and pipe was to prevent any increase in pass-forward flow compared to the baseline. The channel is 1m wide at its base, 8m wide at its bank tops and 2m deep.
- An offline spillway was placed at the downstream end of the diverted channel on its right (north) bank, with its crest level set 120mm below the top of bank, to allow flows in excess of the pipe capacity to overtop into the west tributary.
- The L-shaped headwall was re-positioned and placed approximately 4m downstream of the culvert inlet. It was also extended approximately 16m northeast along the right bank of the diverted channel to catch outflanking flows coming from the spillway and redirect those towards CL21B inlet.
- On the left floodplain of the west tributary, a 500mm high bund of 60m length which ties into the L-shaped headwall was also added as an extra measure to catch overland runoff and redirect it towards the culvert inlet via a toe drain, which was represented as a gully line.

Plate 9.1 MWC21A mitigation setup



9.2 Mitigation model results

9.2.1 The proposed-scheme remains dry for all AEP events as demonstrated by Plate 9.2.

9.2.2 The excavated channel diverting the east tributary westward and flow control measures at its downstream end are effective in keeping the proposed-scheme dry in all events. Flows from the east tributary are attenuated before discharging into the west tributary. During a 1% (1 in 100) AEP +40%CC event, the side spill is overtopped, but the extended headwall redirects flows towards the CL21B inlet.

9.2.3 The new drain and bund on the left floodplain of the west tributary capture all floodwater and redirect it back into the main channel, ensuring that there is no ponding against the proposed A12 embankment for all AEP events.

9.2.4 Plate 9.3 shows the changes to the water level due to the mitigation works for the 1% (1 in 100) AEP +40%CC event. The maximum flood level differences between baseline and “with-scheme plus mitigation” are categorised into levels of impact. The impact of the proposed-scheme for all other flood events can be found in Appendix D.

Plate 9.2 Mitigation flood extents

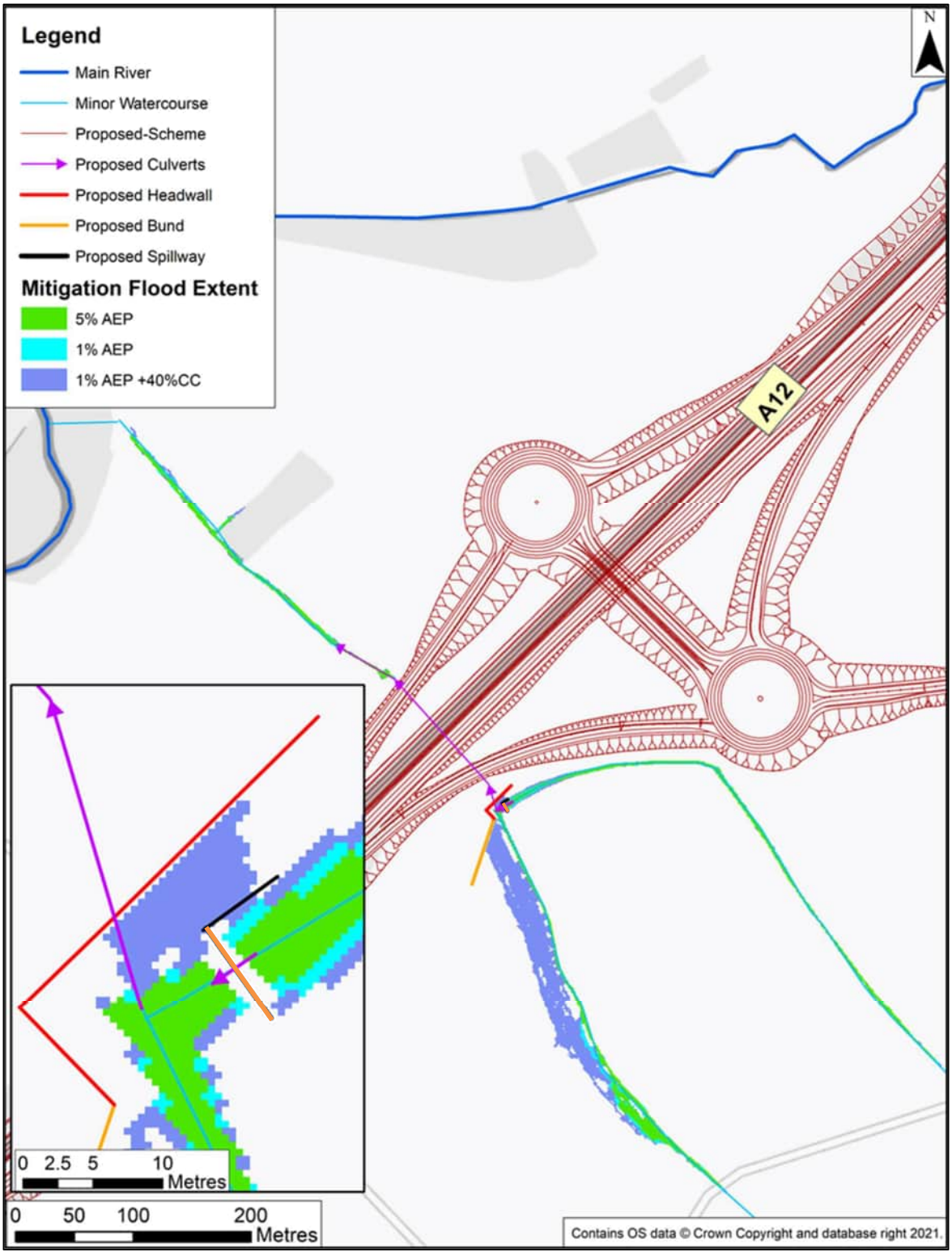
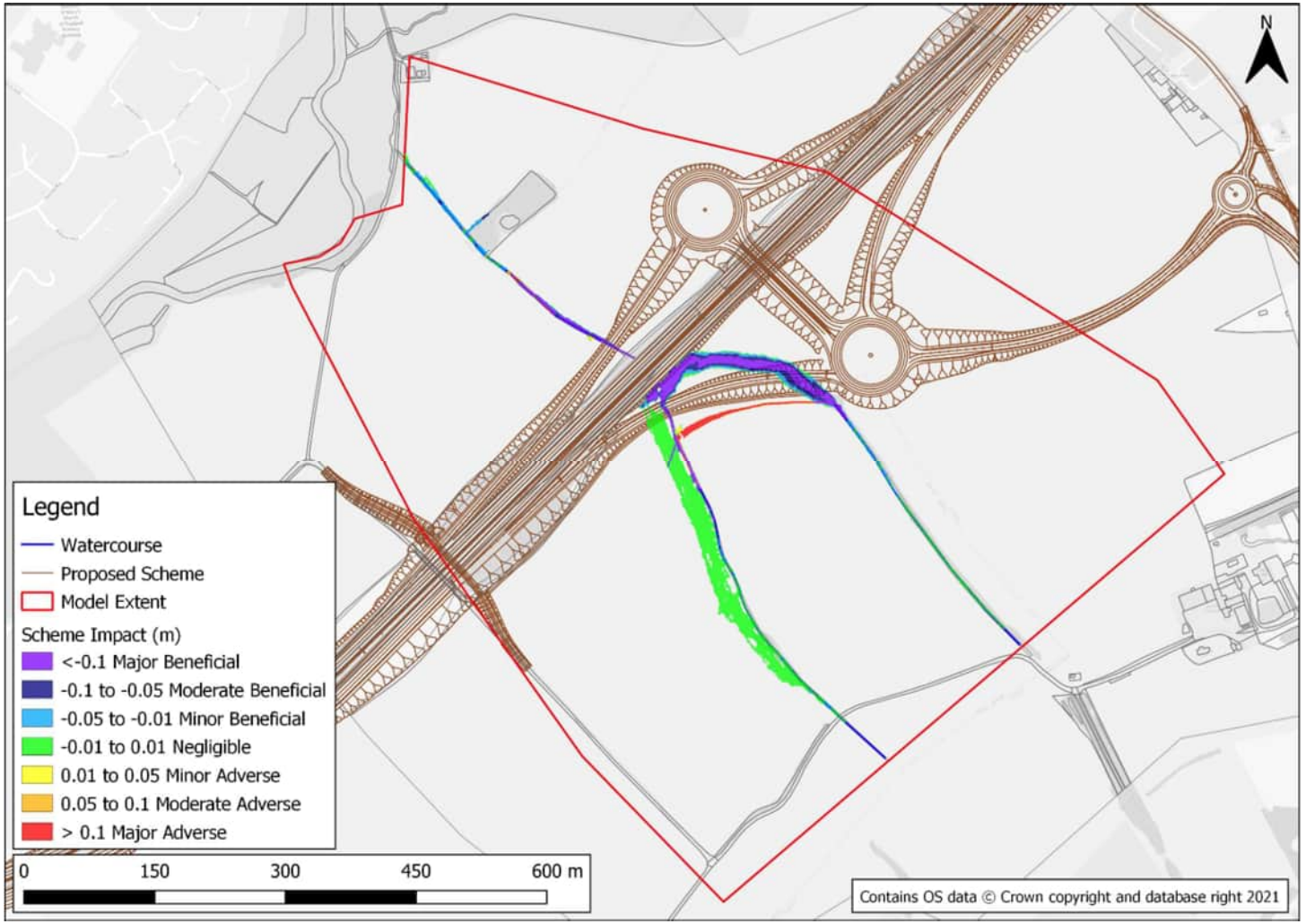


Plate 9.3 Water level difference (mitigation – baseline) for 1% (1 in 100) AEP +40%CC



10 Model assumptions and limitations

10.1 Introduction

10.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological, surveyed, and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

10.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed scheme and are therefore, appropriate for the flood risk assessment.

10.2 Hydrology

10.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- As MWC21A is a small ungauged catchment there is inevitable uncertainty associated with flow estimation for small, ungauged catchments.
- ReFH2.3 is most appropriate for the target catchment.
- FEH Statistical is considered not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 23 / 25.
- The solar farm has little impact on design peak flow estimates.
- Water is assumed to not back up from the River Blackwater during simulated flood events. This is thought to be appropriate given there is approximately a 10m decrease in height between the A12 crossing and the River Blackwater confluence.

10.3 Hydraulic Modelling

10.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for MWC21A are as follows:

- Culvert and model roughness values have been assigned across the model using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the TUFLOW Manual. The dimensions for structures have been based on detailed survey measurements.

- The LiDAR data is assumed to appropriately represent the floodplain.
- A 1m grid has been used. This is deemed a sufficient level of detail to represent floodplain topography and flooding mechanisms predicted by the model in the vicinity of the proposed-scheme.
- No calibration was carried out as the catchment is ungauged.

11 Conclusion

11.1.1 This report has detailed the modelling undertaken to assess the baseline and with-scheme flood risk for Minor Watercourse 21A and its tributaries with respect to the proposed highway scheme.

11.1.2 The results of the baseline modelling have shown that there is floodplain storage at the confluence of the two tributaries just upstream of the A12. However, the A12 carriageway is unaffected by flooding. The introduction of the proposed-scheme results in the flooding of the proposed slip road for all events. Therefore, mitigation works were investigated.

11.1.3 A manmade channel redirecting and attenuating flows from the east tributary with online storage was introduced to the model upstream of the proposed A12 crossing, to prevent flows from the east tributary affecting the proposed-scheme slip road and new junction. The mitigation solution also consists of a toe drain and bund on the left floodplain of the west tributary to redirect flows back into channel.

11.1.4 The proposed mitigation works have successfully managed the design flows through the proposed highway arrangement for all events, and the A12 highway remains free from flooding. Downstream of the proposed-scheme, the risk from flooding remains unchanged from the baseline case with flows remaining in-channel for all simulated storm events..

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
EA	Environment Agency
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
HOST	Hydrology of Soil Types
MWC21A	Minor Watercourse 21A
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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Appendix A **– Minor Watercourse 21A Hydrology Report**

See document HE551497-JAC-EWE-SCHW-RP-LE-0086.

Appendix B **– Sensitivity Testing**

Plate B.1 Water level difference map for the reduced Manning's 'n' sensitivity test

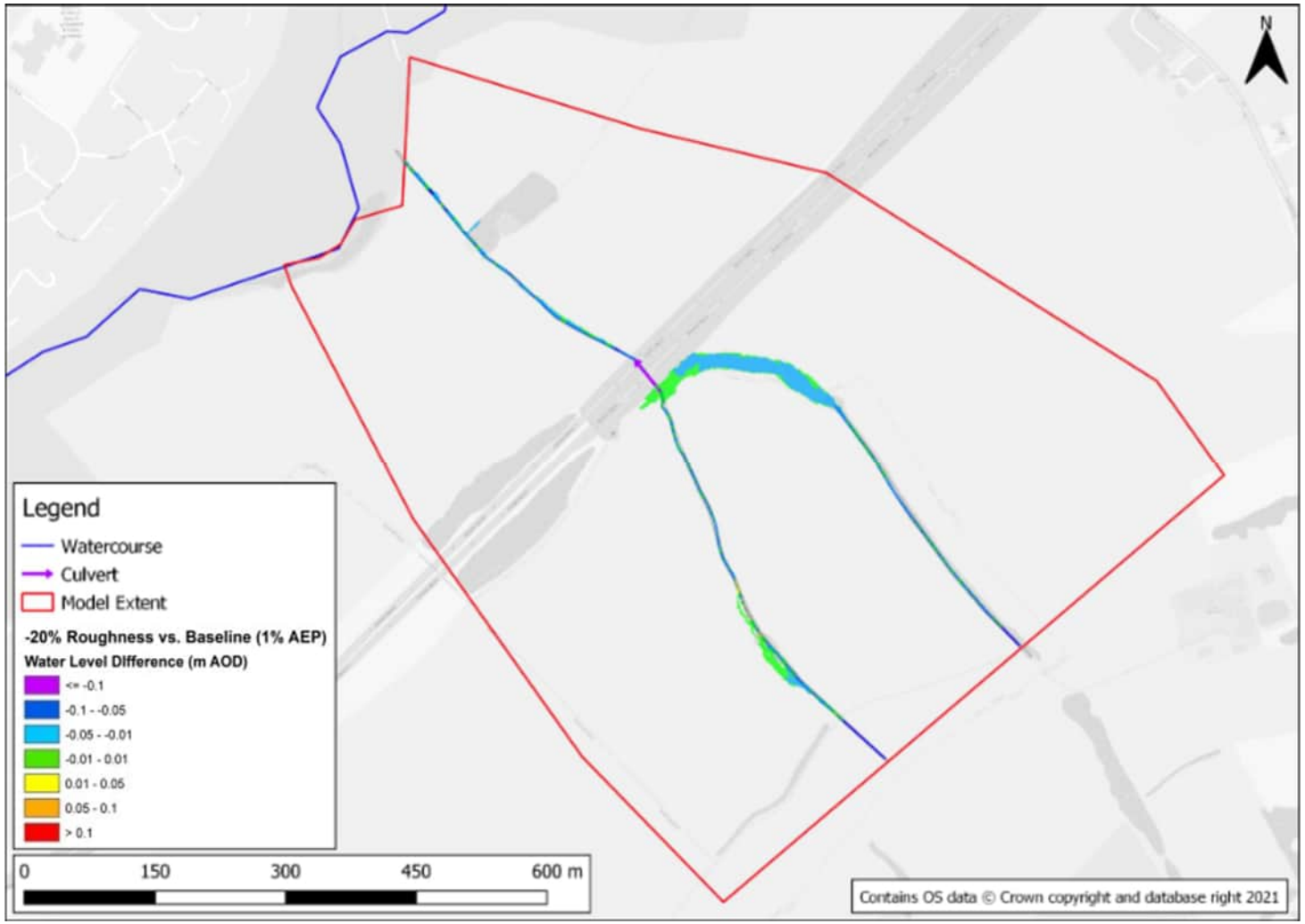


Plate B.2 Water level difference map for the increased Manning's 'n' sensitivity test

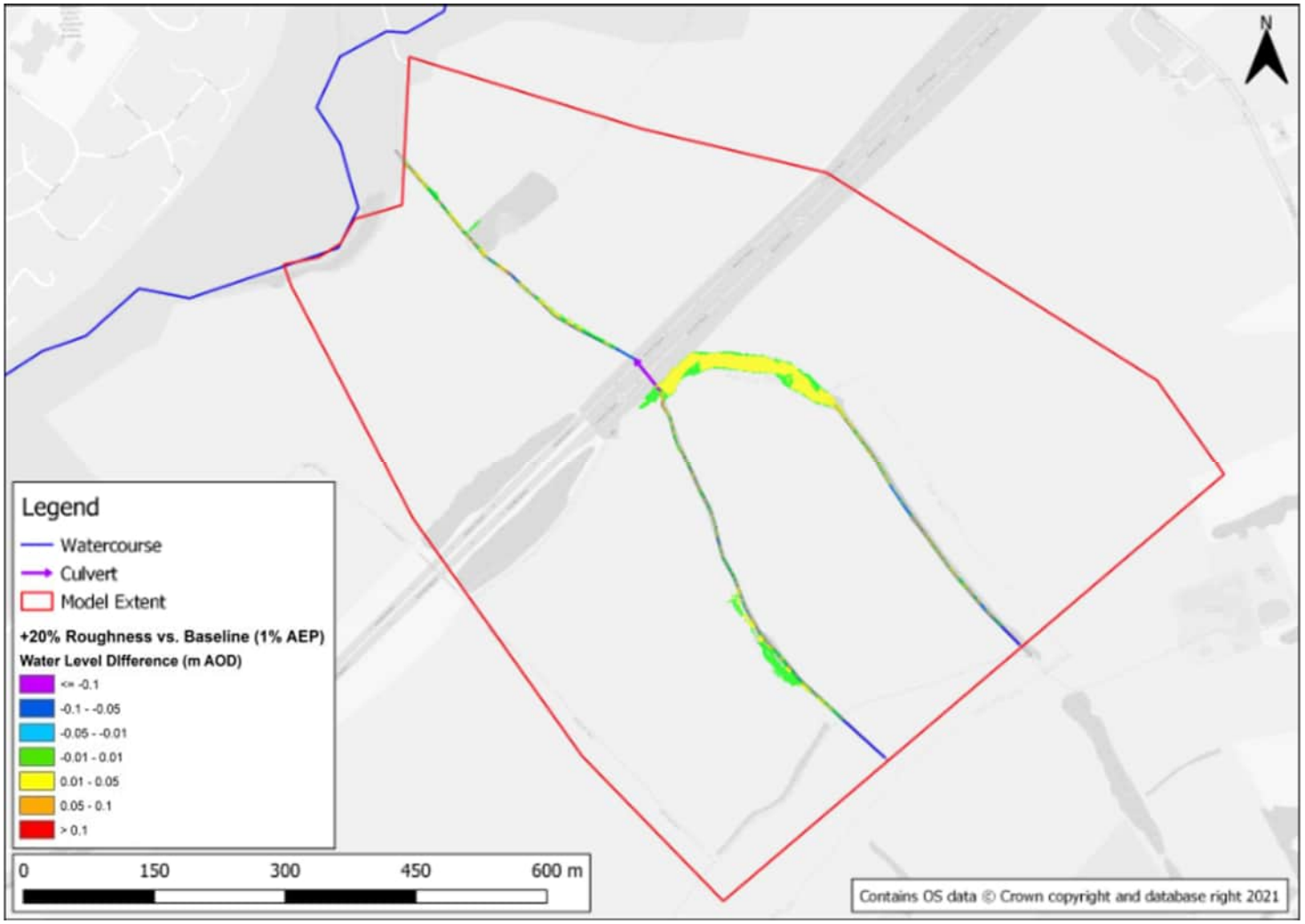


Plate B.3 Water level difference map for the reduced inflow sensitivity test

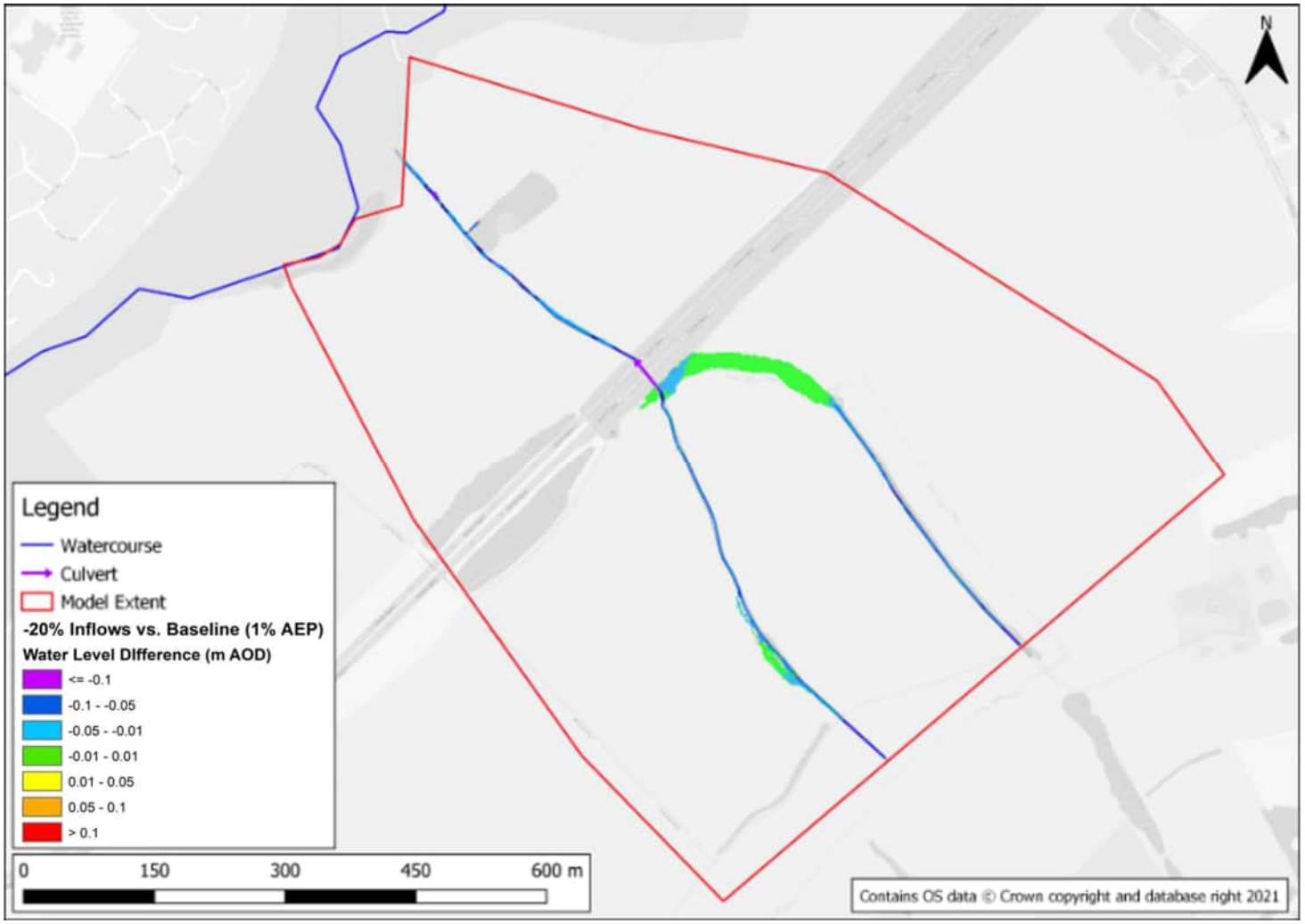
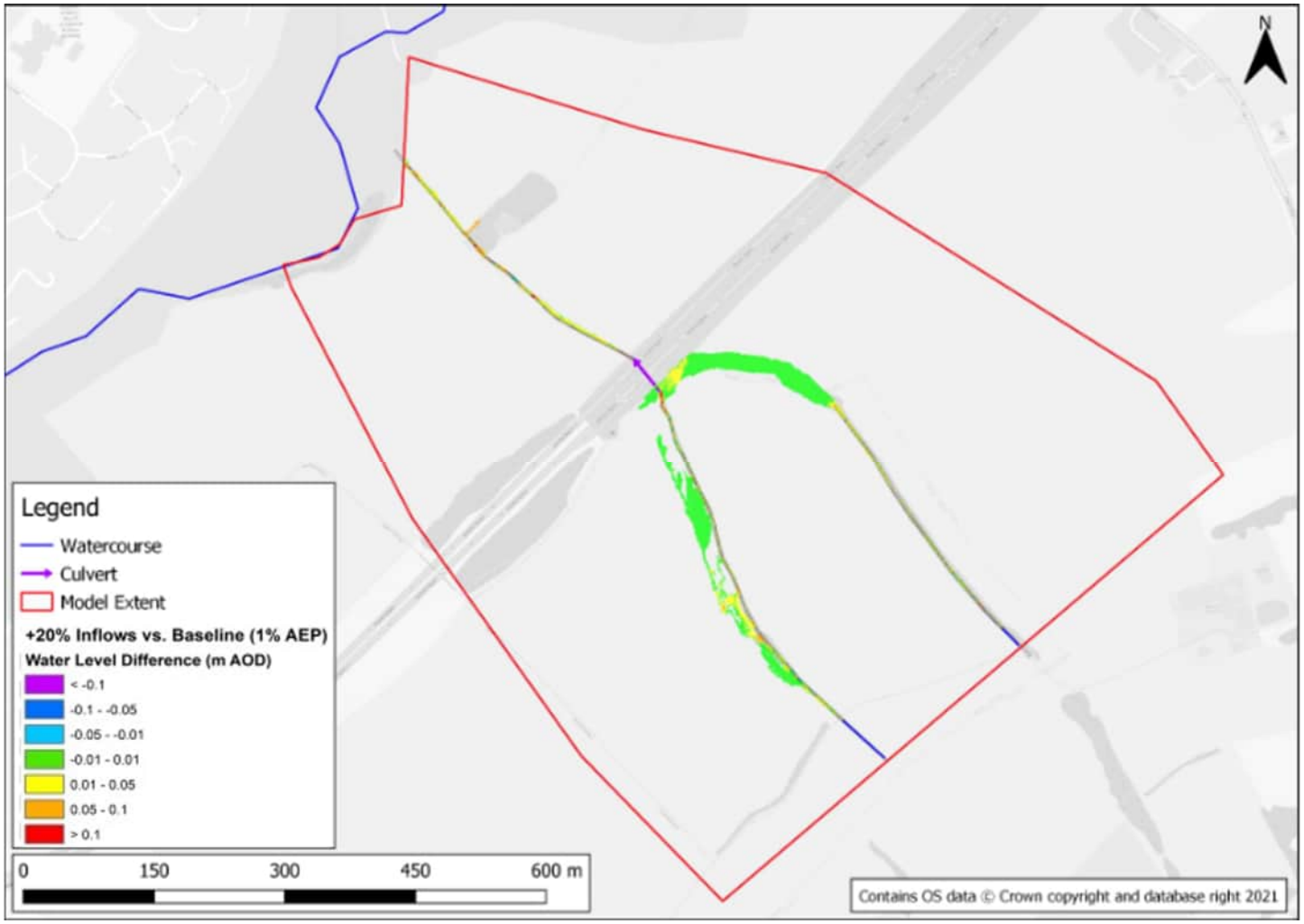


Plate B.4 Water level difference map for the increased inflow sensitivity test



Appendix C **– With-Scheme Flood Mapping**

Plate C.1 5% AEP maximum modelled flood extents for the baseline and with-scheme scenarios.

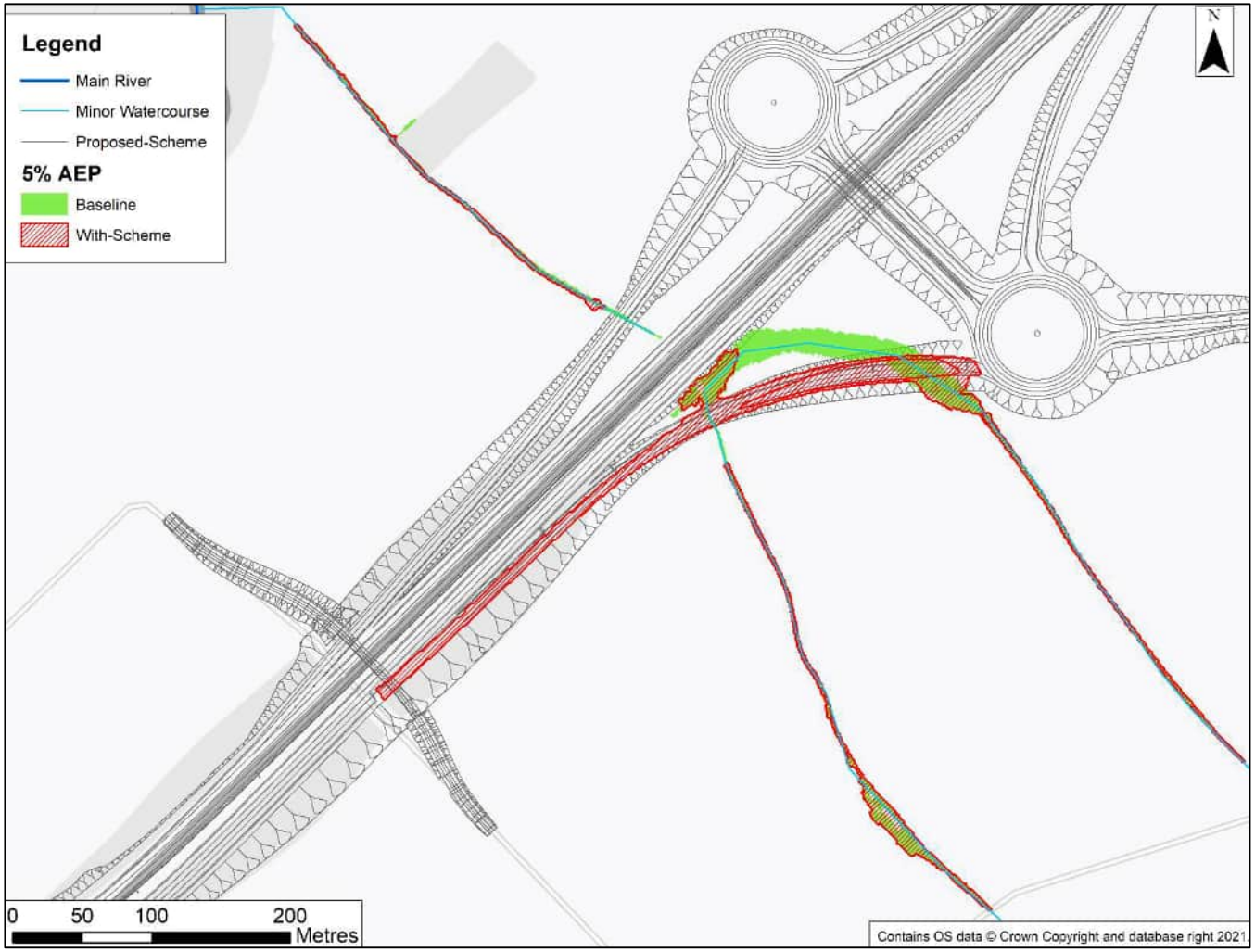


Plate C.2 1% AEP maximum modelled flood extents for the baseline and with-scheme scenarios.

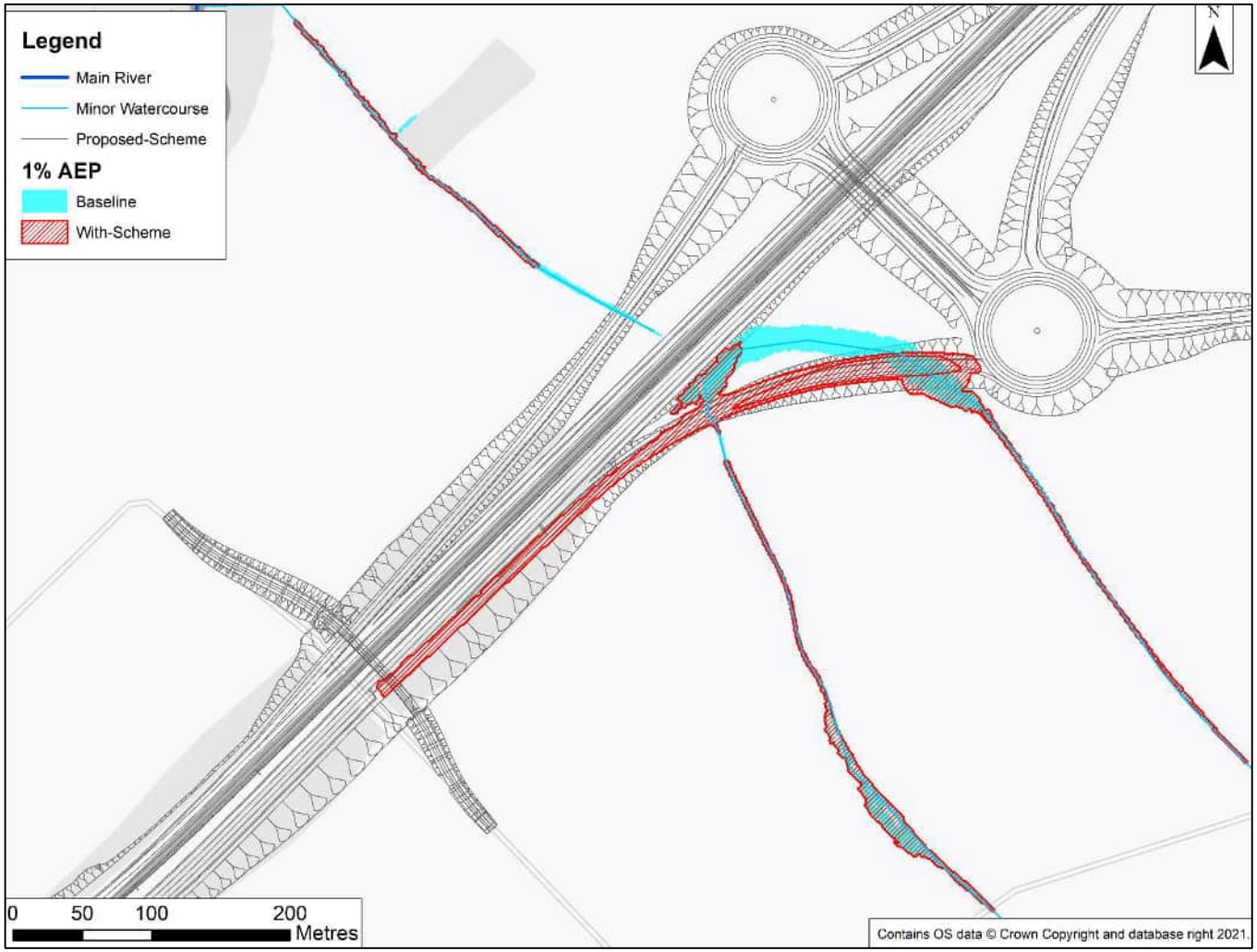


Plate C.3 1% AEP +40%CC maximum modelled flood extents for the baseline and with-scheme scenarios.

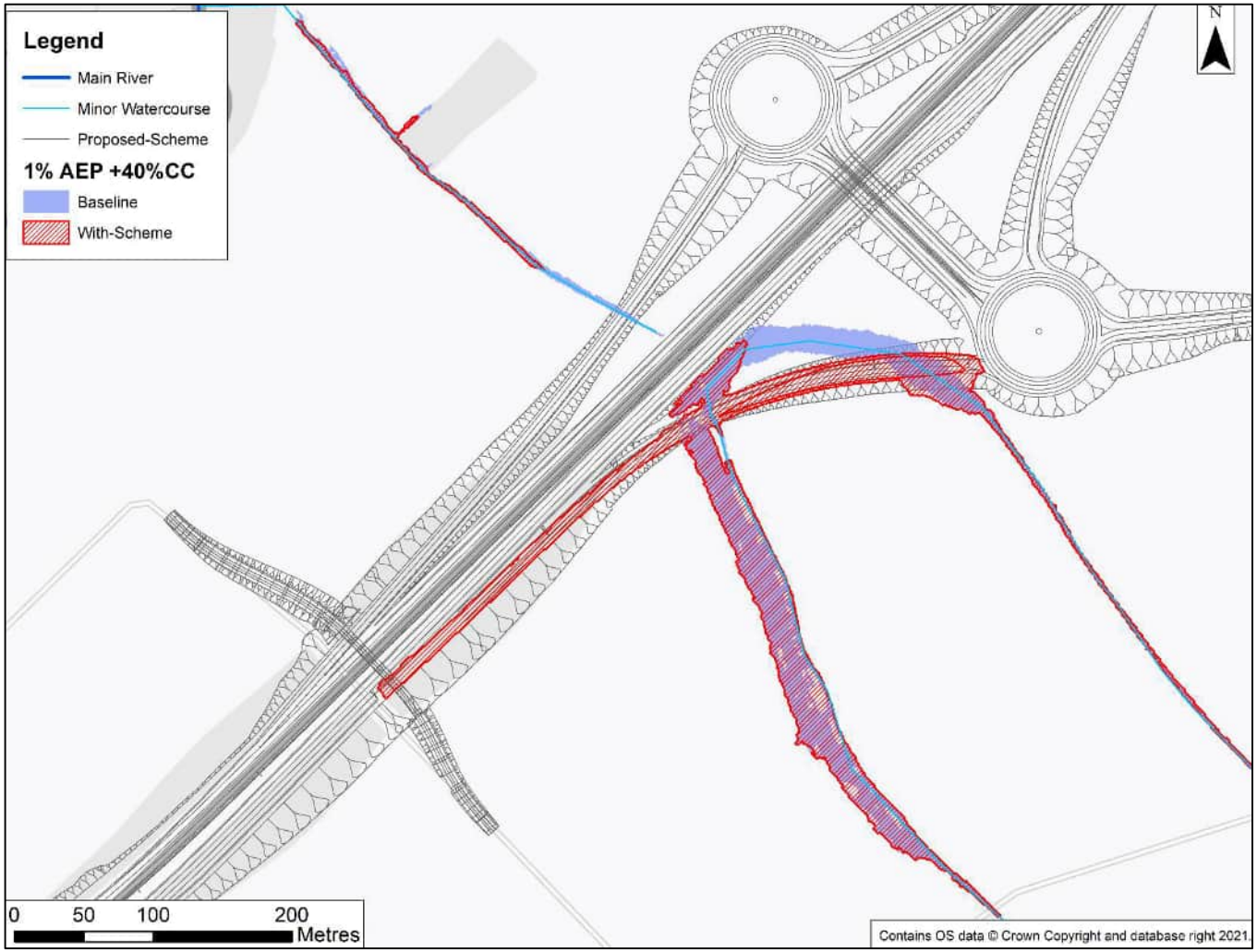


Plate C.4 Water level difference (with-scheme minus baseline) for the 5% AEP event.

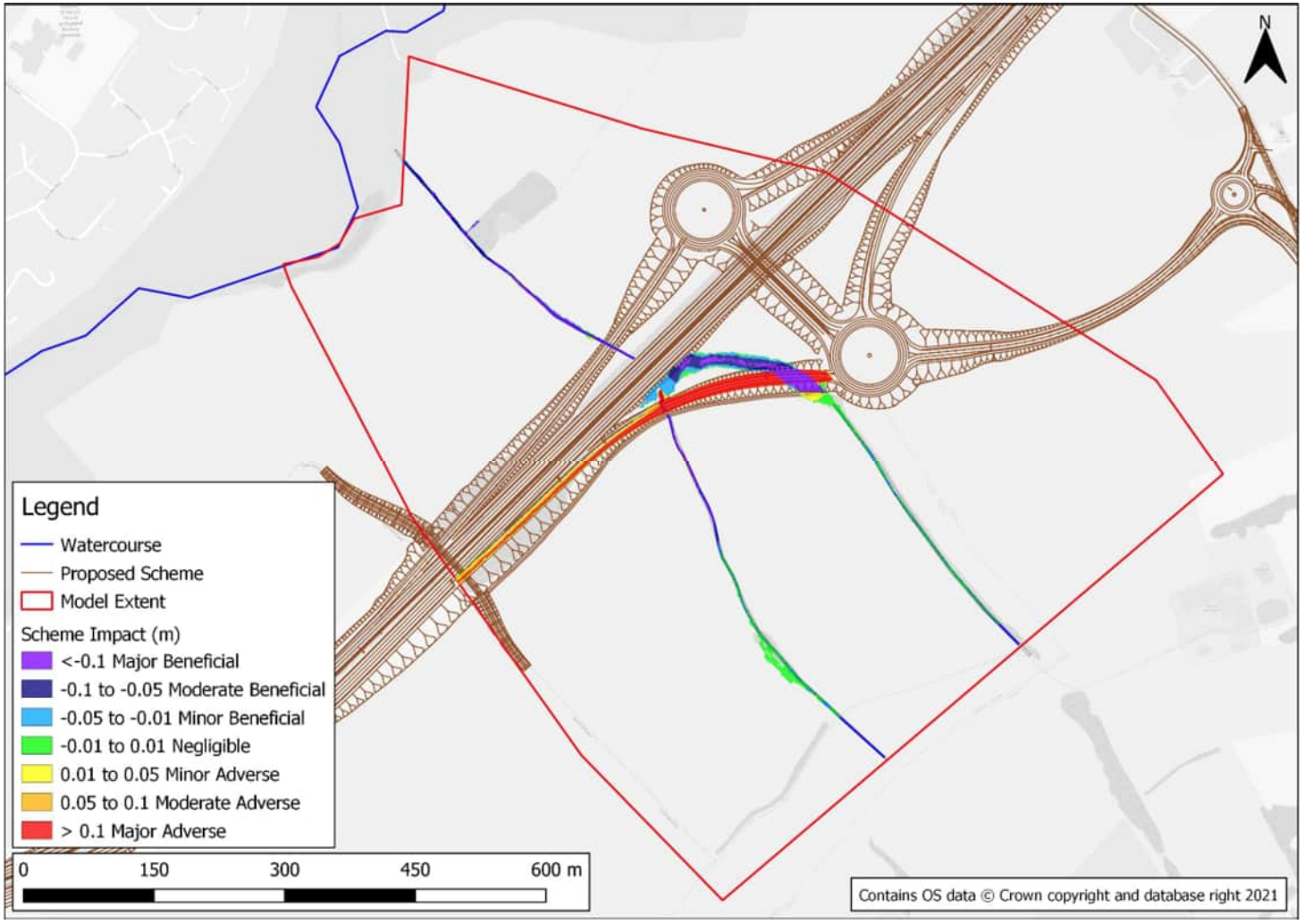


Plate C.5 Water level difference (with-scheme minus baseline) for the 1% AEP event.

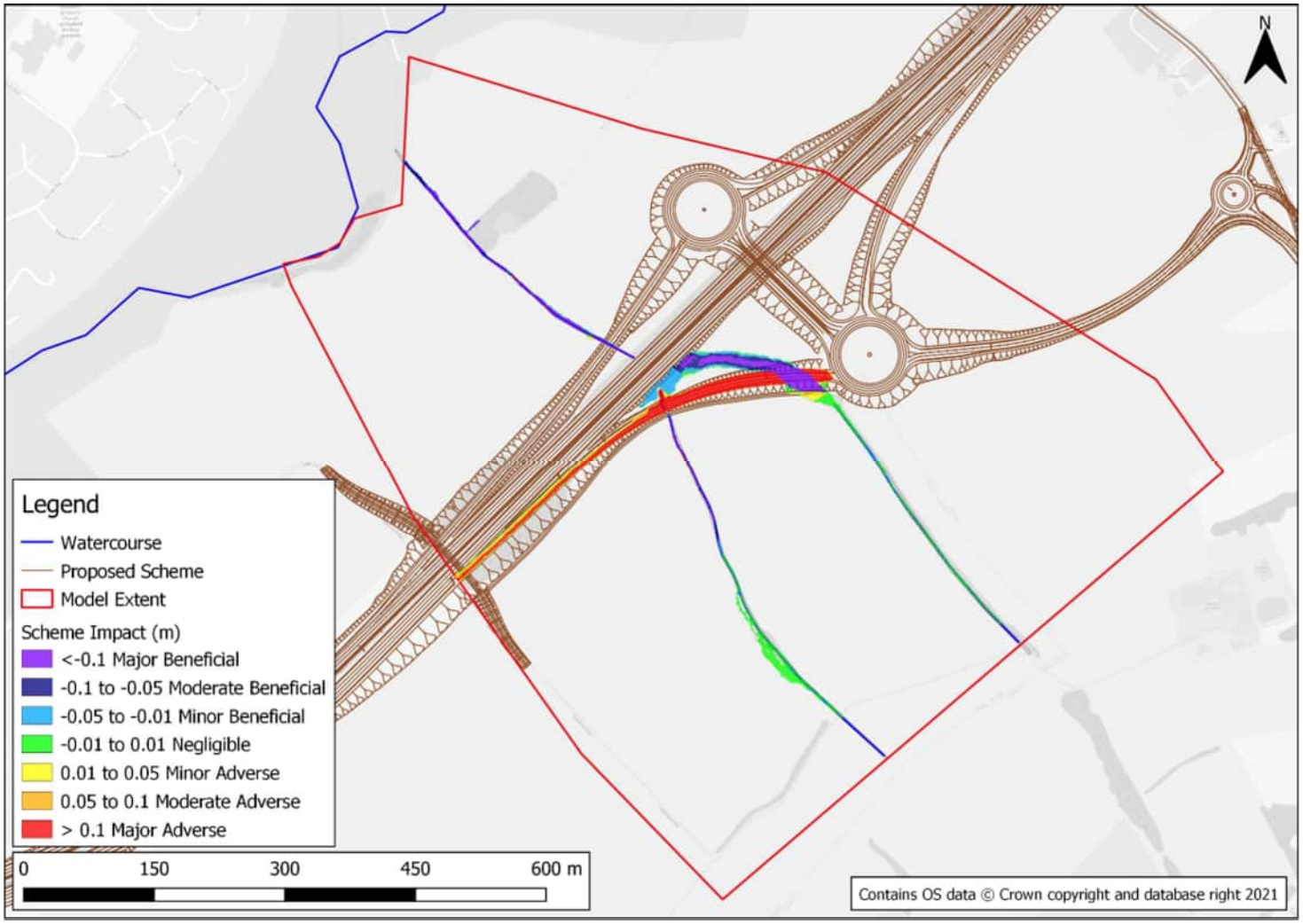
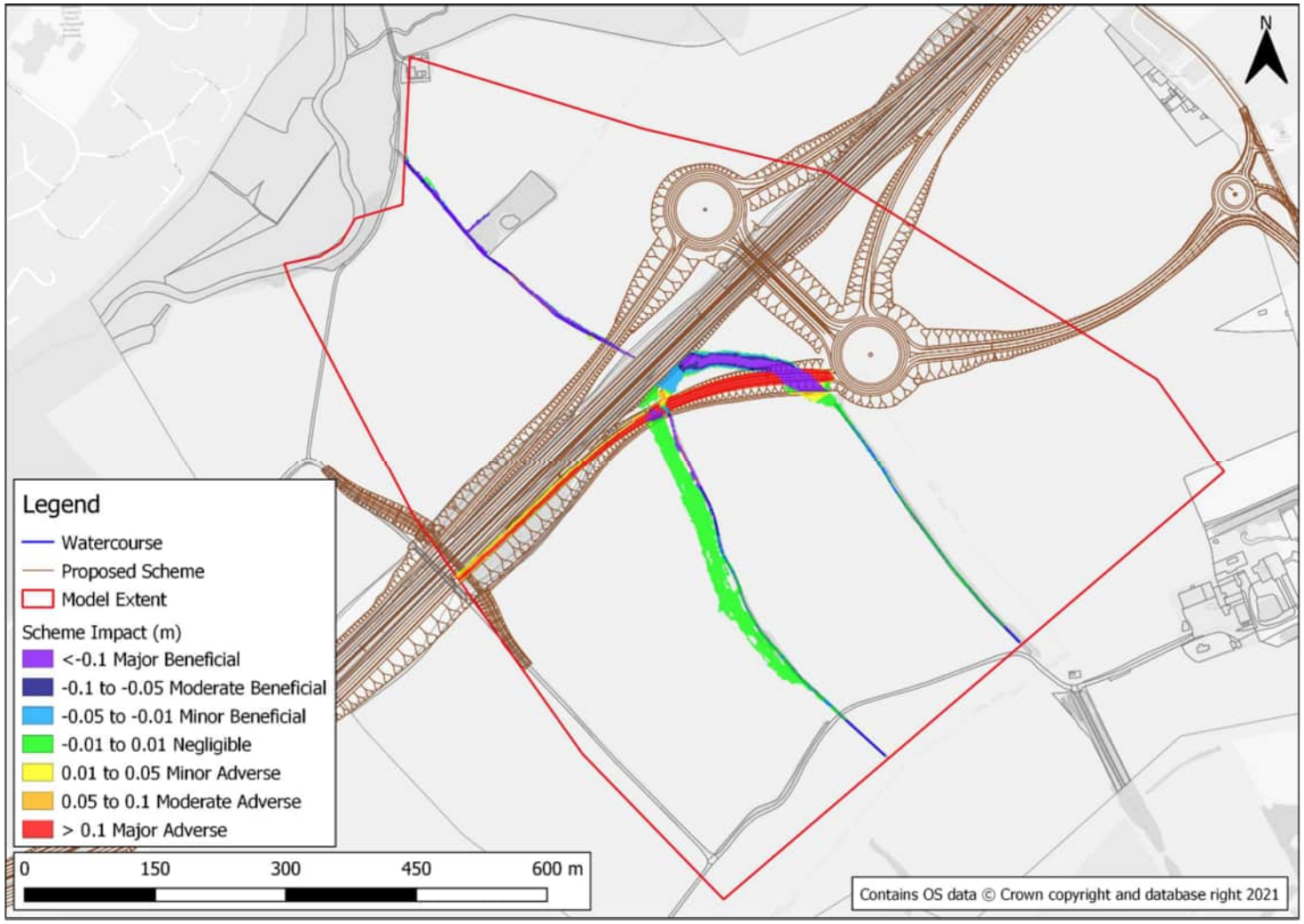


Plate C.6 Water level difference (with-scheme minus baseline) for the 1% AEP +40%CC event.



Appendix D **– Mitigation Flood Mapping**

Plate D.1 5% AEP maximum modelled flood extents for the baseline and mitigation scenarios.

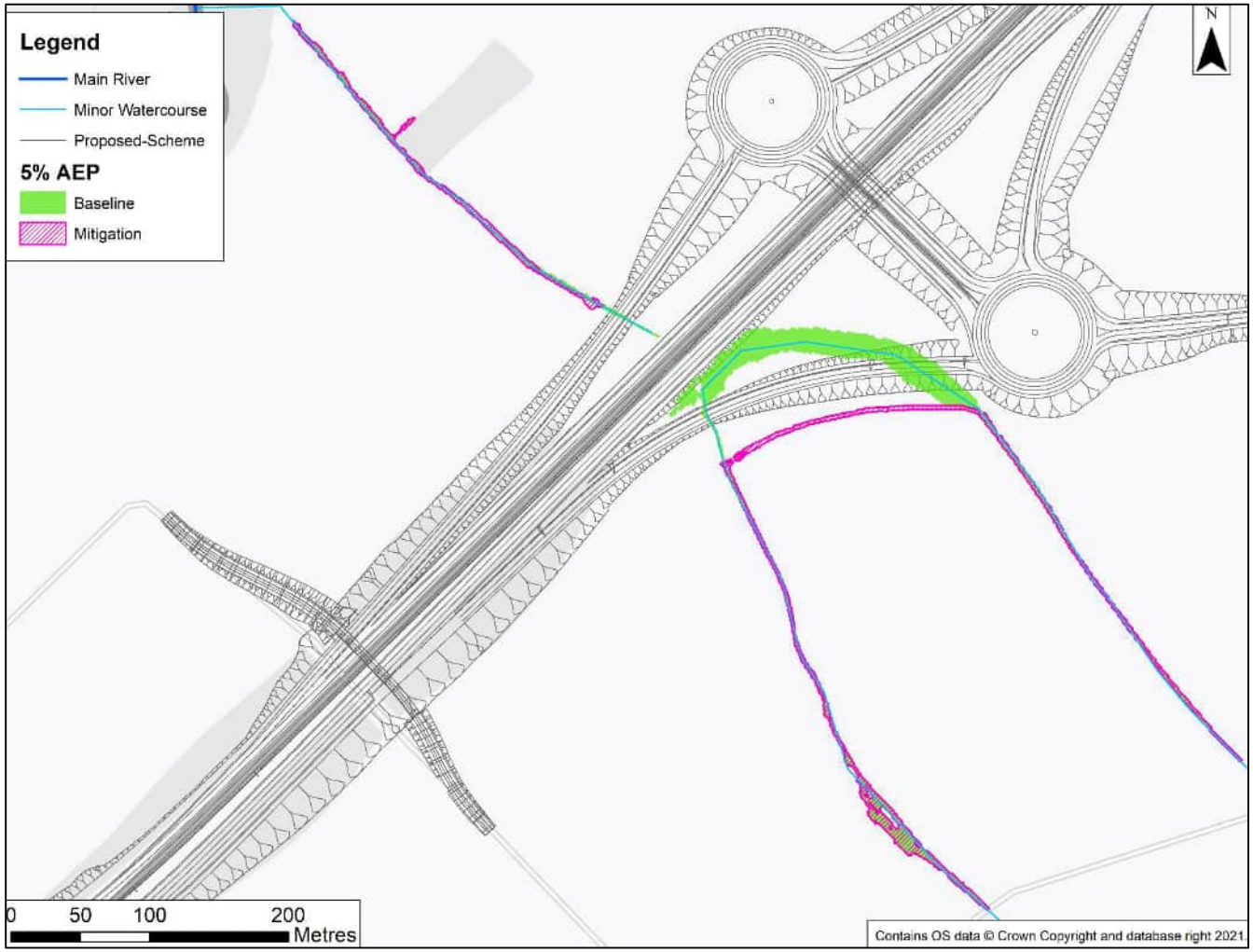


Plate D.2 1% AEP maximum modelled flood extents for the baseline and mitigation scenarios.

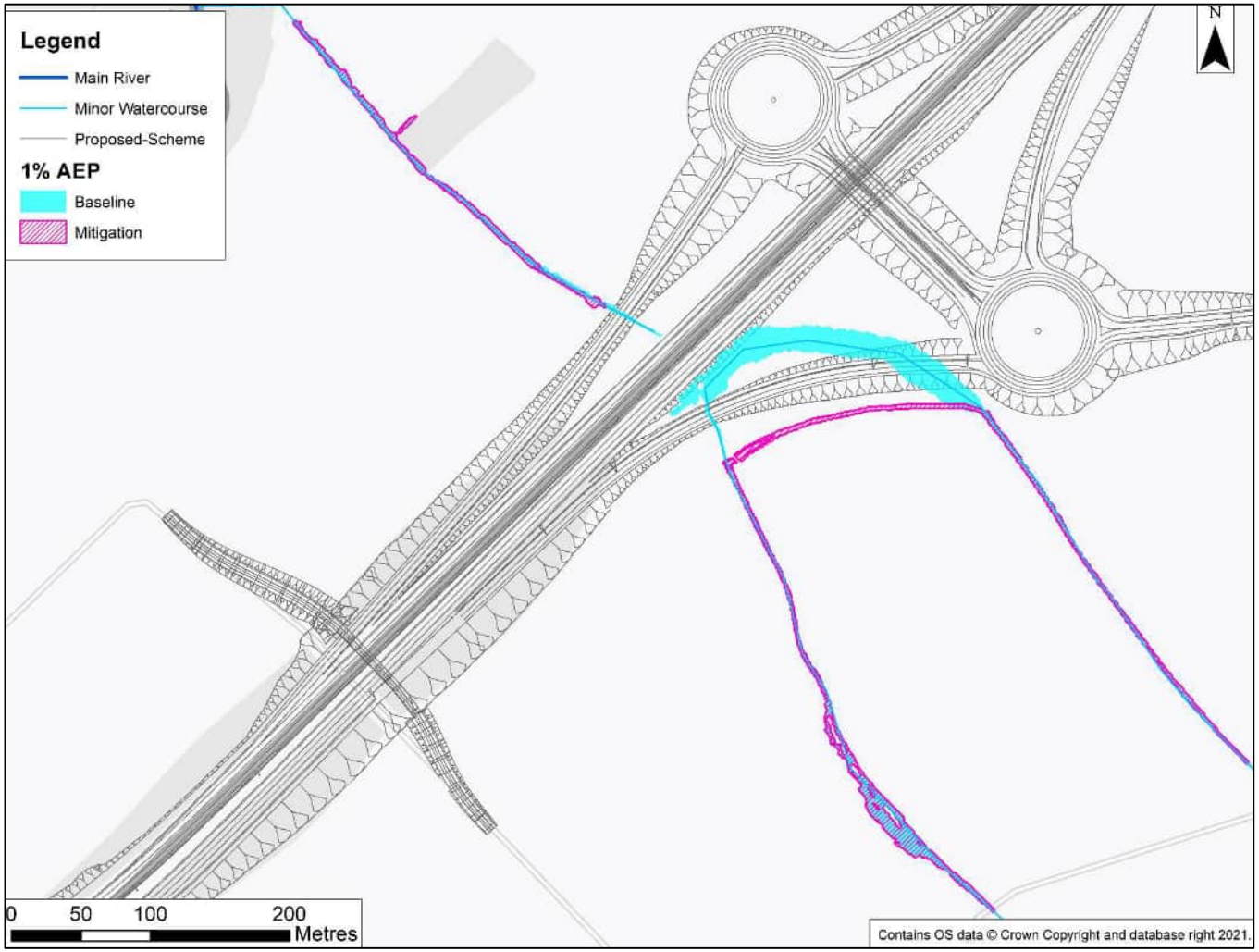


Plate D.3 1% AEP +40%CC maximum modelled flood extents for the baseline and mitigation scenarios.

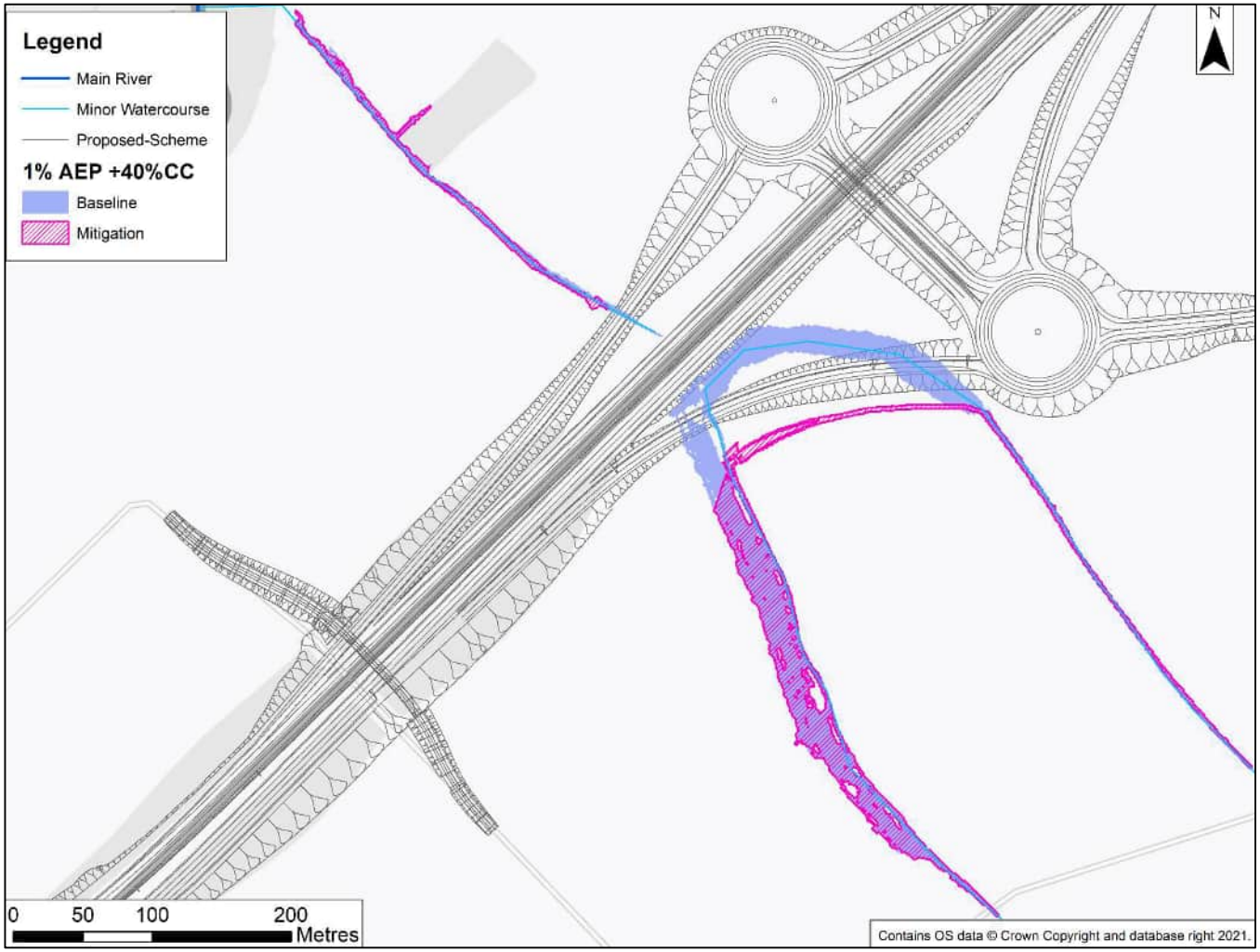


Plate D.4 Water level difference (mitigation minus baseline) for the 5% AEP event.

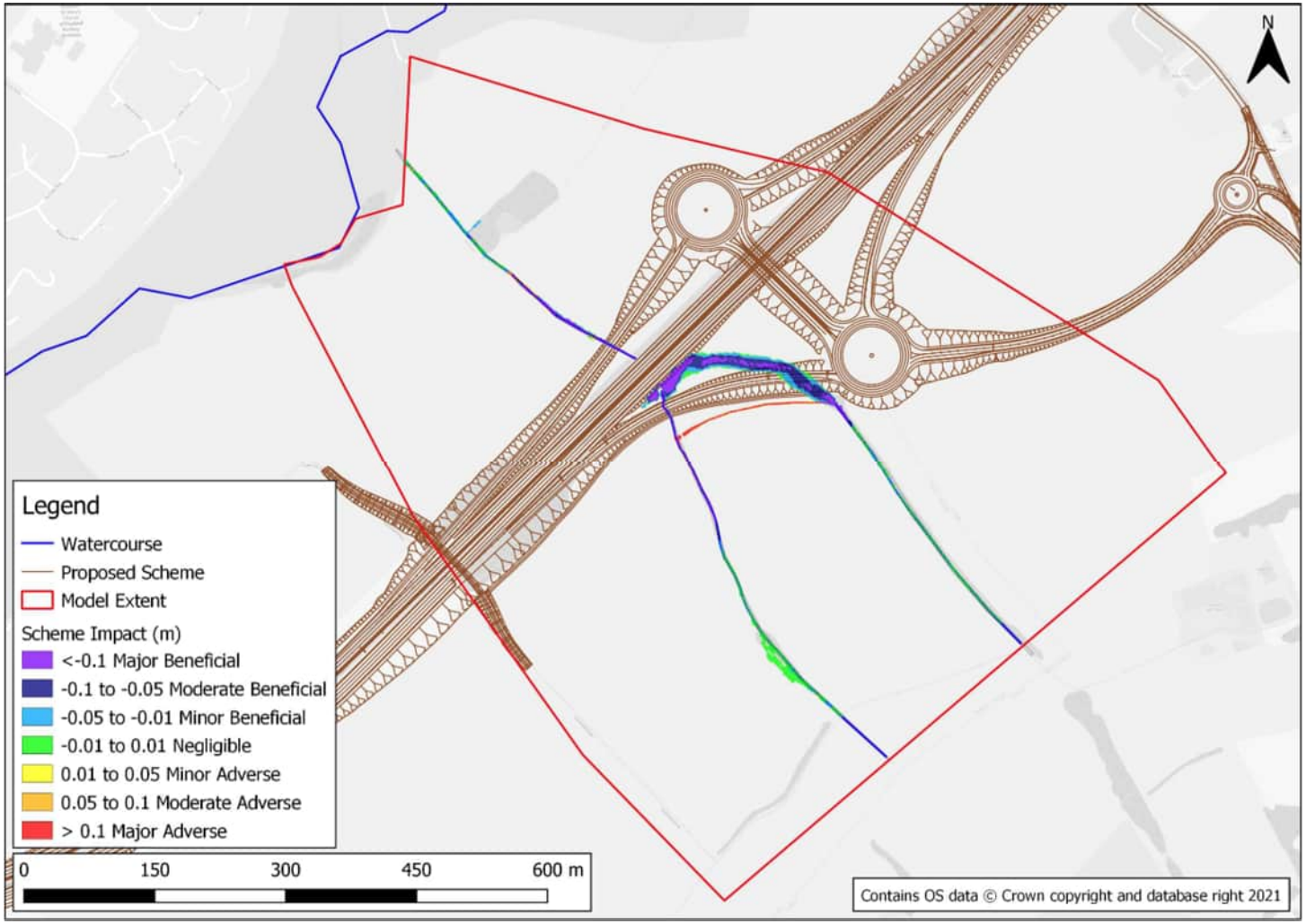


Plate D.5 Water level difference (mitigation minus baseline) for the 1% AEP event.

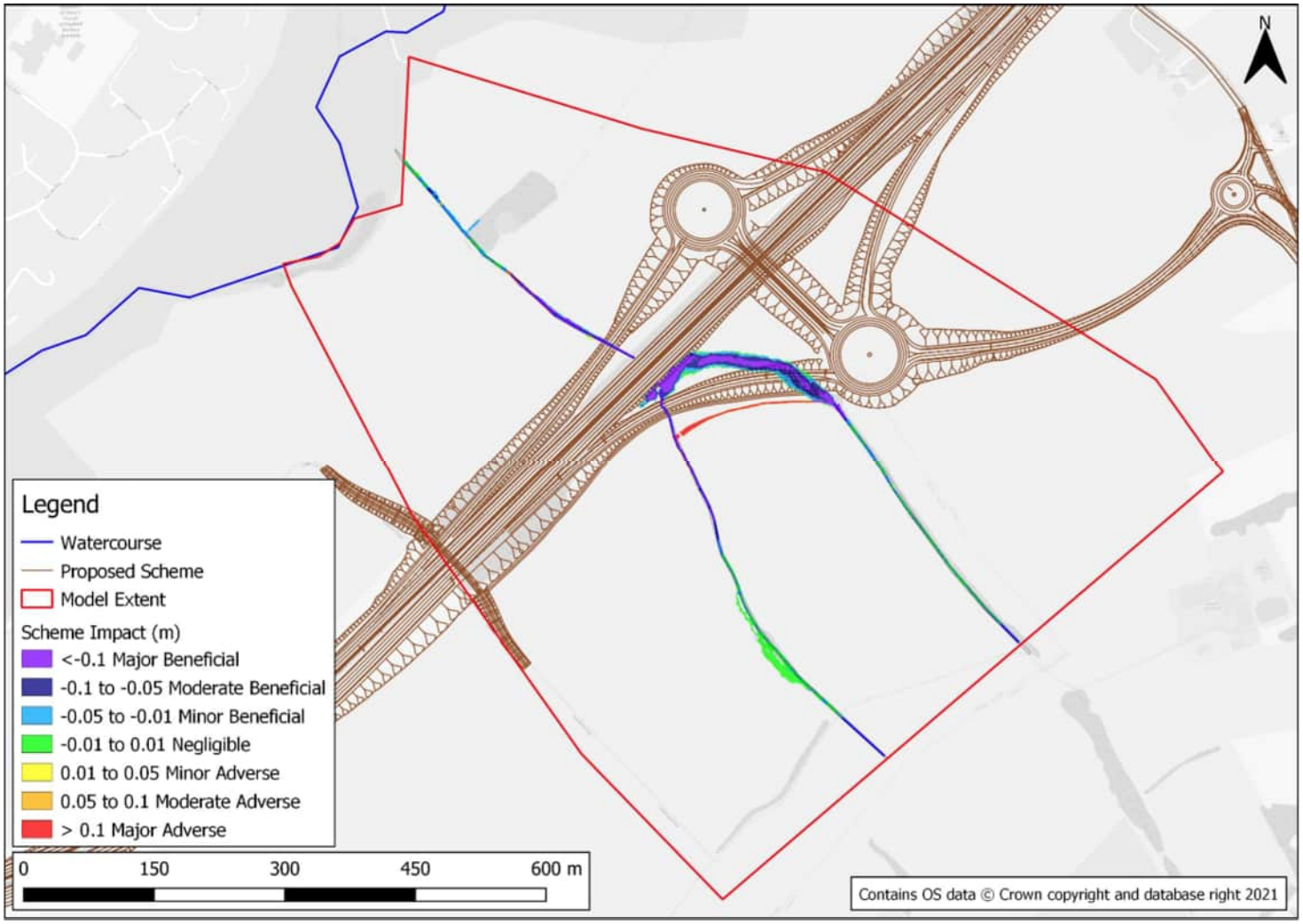
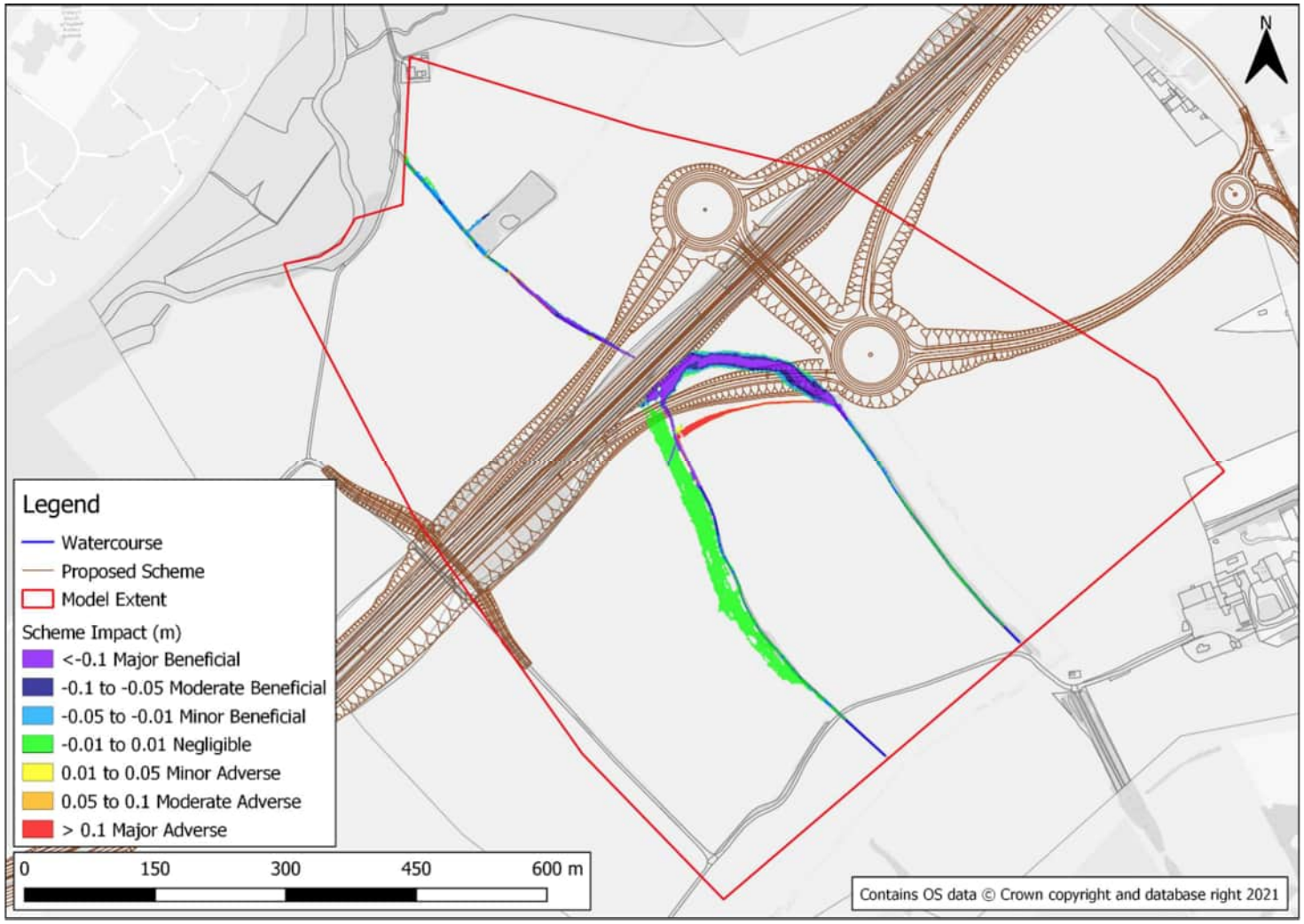


Plate D.6 Water level difference (mitigation minus baseline) for the 1% AEP +40%CC event.



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 21A HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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1 Executive Summary

1.1.1 This document has been prepared to support the A12 Minor Watercourse 21A with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation for the Minor Watercourse 21A (MWC21A) catchment which crosses the A12. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future.

3 Summary of assessment

3.1 Summary

3.1.1 A summary of the key information contained within the detailed hydrological assessment of MWC 21A is provided in the table below.

Catchment location	Minor watercourse 21A (MWC 21A) is located in South East England, just outside Kelvedon and crosses the A12 at approximately NGR: TL 87121 18444. This minor watercourse discharges to the River Blackwater approximately 0.5km downstream of the A12 crossing.
Purpose of study and scope	The A12 is proposed to be upgraded and therefore peak flows and inflow hydrographs are required for the numerical hydraulic modelling of this watercourse. Flows have been derived for the following Annual Exceedance Probability (AEP) events: 5%, 1% and 1% plus climate change (plus cc), equivalent to the 20-year, 100-year and 100-year plus cc return periods events respectively.
Key catchment features	The minor watercourse has a rural catchment. There is a solar farm in the upper catchment. HOST Class 23 and 25 are present within the upper catchment.
Flooding mechanisms	Peak flows / volumes.
Gauged / ungauged	Ungauged.
Final choice of method	ReFH2.3
Key limitations / uncertainties in results	Small ungauged watercourse not picked up by the FEH Web Service (donor FEH catchment descriptors were therefore required). Upper catchment contains HOST Class 23 and 25 and therefore BFIHOST values open to uncertainty.

3.2 Note on flood frequencies

3.2.1 The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

3.2.2 Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval.

3.2.3 The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Table A.1 Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

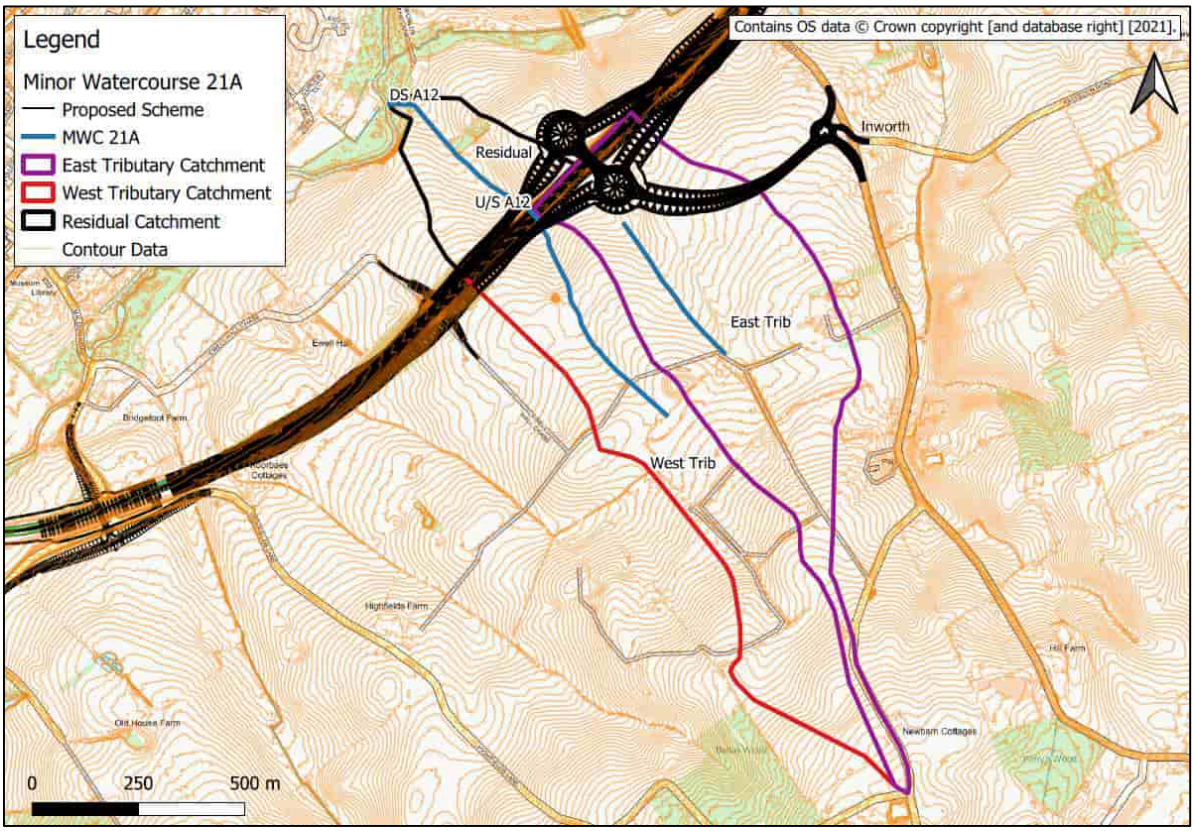
4 Method statement

4.1 Requirements for flood estimates

Overview	The proposed A12 upgrade route crosses the Minor Watercourse 21A (MWC 21A) and therefore peak flows and hydrographs are required for the hydraulic modelling of this watercourse. Design peak flows and hydrographs are required for the following AEP events: 5%, 1% and 1% plus climate change (plus cc). This is equivalent to the 20-year, 100-year and 100-year plus cc return periods). A climate change uplift factor of plus 40% will be applied for the 1% AEP (100-year return period) event. This is based on current guidance[1] (peak rainfall allowance, upper end, 2080s scenario). MWC 21A is a small and ungauged watercourse. For hydraulic modelling purposes two flow estimation points have been adopted, one for the catchment up to the A12 road crossing and the other for the catchment downstream of the A12 crossing to its confluence with the River Blackwater (residual catchment).
Project scope	This is a simple hydrological study for one minor watercourse with a catchment area up to its confluence with the River Blackwater of approximately 0.75km^2. Therefore, ReFH model parameter calibration for this ungauged catchment is outwith the scope of this project.

[1] Environment Agency (2016). Flood risk assessments: climate change allowances (updated 2020).

4.2 The catchment



Description	As shown in the above figure, Minor Watercourse 21A has two branches upstream of the A12, namely, the east and west tributaries which combine just upstream of the A12. MWC 21A discharges to the River Blackwater 0.43km downstream of the A12 crossing. There is approximately a 10-meter decrease in height between the A12 road and the River Blackwater confluence, thus any backwater in the River Blackwater is unlikely to reach the MWC 21A crossing by the A12. The catchment is classified as essentially rural with a few dwellings present throughout the catchment. Land use is predominantly agricultural. There is also a solar farm within the upper reaches of the catchment (accounting for approximately 8% of the total catchment area). The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 813b (Fladbury 1) near the River Blackwater, 571s (Efford 1) and 572q (Ashley) in the middle reaches, with 712c (Windsor) being present in the upper reaches of

	the catchment. The HOST class of the soil groups were assessed and soil type 712c was found to contain HOST class 23 and 25. Flood Estimation Guidelines[2] highlight that BFIHOST should be avoided for clay catchments in south east England associated with HOST class 23 or 25. This is due to the coefficients being assessed as too low by more recent assessment. BFIHOST19 was noted by the guidance as being updated to reflect this. The soils within the catchment were also assessed using Soilscapes[3], which indicates that there are loamy and clayey soils present within the catchment. According to the Geology of Britain Viewer[4] the geology of the catchment is composed of London Clay Formations consisting of clay, silt and sand.

4.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released 24th September 2020. This contains provisional data up to water year 2019-20.

4.4 Gauging stations (flow or level)

Watercourse	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
River Blackwater	Blackwater @ Appleford Bridge		37010	247.3	Double throated trapezoidal flume and high flow rated spillway.	1962 to present
River Ter	Ter @ Crabbs Bridge		37003	77.8	Theoreticall y rated	1932 to present

[2] Flood Estimation Guidelines, Technical guidance 197_08 (Environment Agency, June 2020)

[3] http://www.landis.org.uk/soilscapes/

[4] geology from the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

4.5 Data available at each flow gauging station

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
Blackwater @ Appleford Bridge	1962 to present	N	Yes	Yes	N	Groundwater influenced regime. Intermittently affected by the Ely / Ouse transfer scheme.
Ter @ Crabbs Bridge	1963 to present	N	Yes	No	N	Recently removed from suitable for pooling category. This was noted as due to many AMAX events being out of bank and this is not accounted for in rating.

4.6 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings	N/A			Not required for this assessment.
Historical flood data	N/A			Not Available
Flow or river level data for events	N/A			Not available for this assessment.
Rainfall data for events	N/A			Not required for this assessment.
Potential evaporation data	N/A			Not required for this assessment.
Results from previous studies	N/A			Not available for this assessment.
Other data or information	N/A			Not available for this assessment.

4.7 Hydrological understanding of catchment

Conceptual model	Hydraulic modelling of MWC 21A is required to assess potential impacts of upgrading the A12. The target location is the A12 crossing of MWC 21A. Flooding is likely to be caused by peak flows/volumes at the A12 crossing location.
Unusual catchment features	The catchment is a small rural clay catchment located in South East England. There are noted issues with BFIHOST values for the soil (which contains HOST 23 and 25) within the upper catchment.

4.8 Initial choice of approach

Is FEH appropriate?	Yes, FEH methods are appropriate but care needs to be taken due to BFIHOST issue.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	ReFH2.3 and FEH Statistical methods have been applied to derive the peak flow estimates and hydrographs for this catchment. The hydrographs have been derived using ReFH2.3 method. Flow estimates have been derived for the catchment to the A12. This flow has been area scaled between the east and west tributaries within the hydraulic model. A residual catchment downstream flow has further been provided for the catchment area between the A12 road and the River Blackwater confluence.
Software to be used (with version numbers)	FEH Web Service[5] / WINFAP 4[6] / ReFH2.3

[5] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

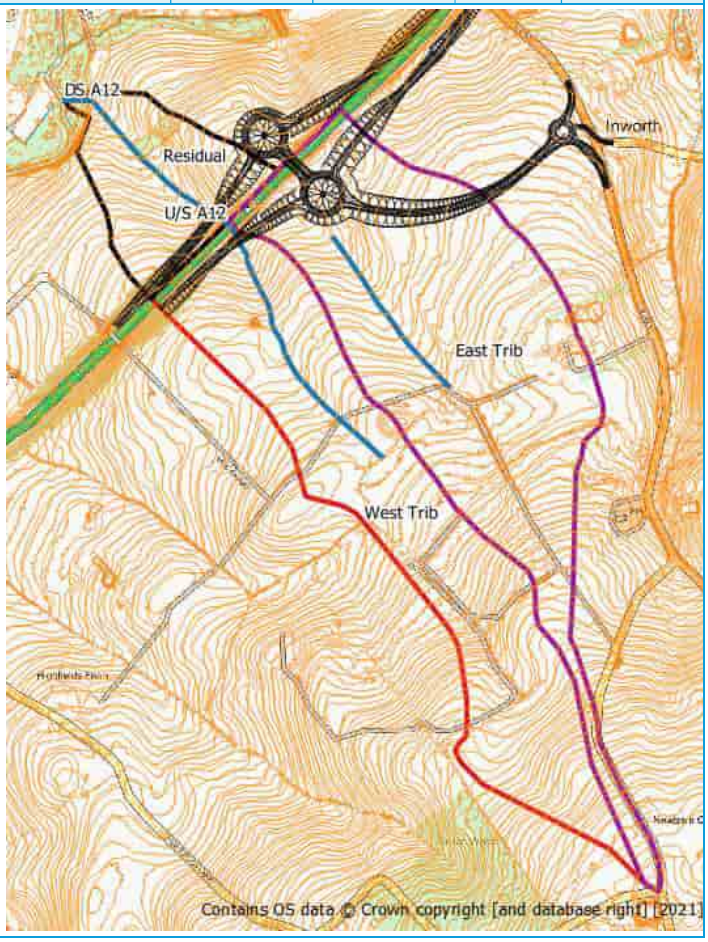
[6] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.9 Locations where flood estimates required

4.9.1 The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

4.10 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
US A12	L	MWC 21A	Catchment upstream of the A12	587118	218448	-	0.676
East Trib	S	MWC 21A	East tributary upstream of A12	587141	218439	-	0.377
West Trib	S	MWC 21A	West tributary upstream of A12	587129	218435	-	0.299
Residual	S	MWC 21A	Catchment downstream of the A12	586887	218652	-	0.097
DS A12	L	MWC 21A	Catchment to River Blackwater confluence	586787	218701	-	0.773

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required. Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced. The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.							

4.11 Important catchment descriptors at each subject site (incorporating any changes made)

4.11.1 As the FEH Web Service does not pick up MWC 21A, a donor catchment to the west of the target watercourse (586150 218150) was therefore selected for the purposes of this assessment. Key FEH catchment descriptors (area, DPSBAR, DPLBAR and URBEXT) were calculated for the target locations using standard FEH methodologies.

Site code	FARL	PROPWET	BFIHOST	BFIHOST 19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
Donor	1	0.23	0.292	0.295	1.31	26.40	562	0	0.088
US A12	1	0.23	0.292	0.295	0.81	23.49	562	0.004	0.088
Residual	1	0.23	0.292	0.295	0.28	28.72	562	0	0.088
DS A12	1	0.23	0.292	0.295	0.87	23.01	562	0.004	0.088

4.12 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	As MWC 21A is not picked up by the FEH Web Service, the catchment areas for this watercourse were assessed using 1m contour data and LIDAR data.
Record how other catchment descriptors were checked and describe any changes.	Donor FEH catchment descriptors were adopted from an adjacent minor watercourse. Key catchment descriptors such as FARL and soils were checked using mapping data and key catchment descriptors were calculated using standard methods (e.g. area, DPSBAR, DPLBAR, URBEXT).
Source of URBEXT	URBEXT1990 and URBEXT2000 calculated using URBAN50K method.
Method for updating of URBEXT	URBEXT calculated using URBAN50K.

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	The FEH Statistical method was used as a comparison against ReFH2.3 peak flow estimates. The FEH Statistical method uses BFIHOST which has known issues in South East England on clay catchments with HOST Classes 23 and 25 within the catchment.

5.2 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjust-ment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor			
						Weight	Weighted ave. adjustment		
US A12	0.163	CDs	-	-	-			1.004	0.164
DS A12	0.183	CDs	-	-	-			1.003	0.184

Are the values of QMED spatially consistent?	Yes
Method used for urban adjustment for subject and donor sites	

Parameters used for WINFAP v4 urban adjustment if applicable

Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment

descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010) in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016).

5.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Donor gauges were assessed for the purposes of QMED adjustment; however, no donor was selected. The River Blackwater @ Appleford Bridge gauge (37010) was assessed as it is the closest gauge, however, the River Blackwater @ Appleford Bridge gauges a much larger catchment with groundwater influence in the upper reaches of the catchment. This gauge was therefore assessed as not suitable as a donor for this small ungauged catchment. The River Ter @ Crabbs Bridge (37003) was also assessed as a donor for QMED adjustment, but this gauge was found to have varying FEH cds from the target site. Both the River Blackwater @ Appleford Bridge and River Ter @ Crabbs Bridge gauges would have resulted in a decrease in QMED and given neither were assessed as suitable donors QMED was not adjusted.

5.4 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)

5.5 Derivation of pooling groups

5.5.1 Given the target watercourse is not picked up by the FEH Web Service, FEH Statistical Pooling was undertaken on the donor catchment (586150 218150).

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
Donor 1 Final Pooling Group	Donor	No	Removed: 27073 / 26016 (due to high BFIHOST), 47022 / 106002 (due to low FARL) and 49005 (due to relatively high BFIHOST / high discordance). Added 206006 / 36010	L-CV: 0.226 L-Skew: 0.268
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

5.6 Derivation of flood growth curves at subject sites

Site code	Method	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
Donor	P	Donor 1 Final Pooling Group	GL: distribution gives an acceptable fit.	Urban Adjustment	Location: 1.000 Scale: 0.221 Shape: -0.268	3.003
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis Urban adjustments are all carried out using the method of Kjeldsen (2010). Growth curves were derived using the procedures from Science Report SC050050 (2008).						

5.7 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	25	50	100	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events									
	50	20	10	5	4	2	1	0.5	0.2	0.1
US A12	0.164	0.225	0.272	0.326	0.346	0.413	0.492	0.588	0.744	0.890
DS A12	0.184	0.252	0.305	0.366	0.387	0.462	0.551	0.658	0.833	0.997

6 Revitalised flood hydrograph 2 (ReFH2) method

6.1 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 has been applied to derive both peak flow estimates and hydrographs for the target watercourse at key locations for input into a hydraulic model.

6.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp}	BL (hours)	BR
US A12	ReFH2.3	3.33	-	260.42	-	27.63	0.71
Residual	ReFH2.3	1.70	-	260.42	-	21.92	0.71
DS A12	ReFH2.3	3.49	-	260.42	-	28.07	0.71
Brief description of any flood event analysis carried out							
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

6.3 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
US A12	Rural	Winter	5.5
DS A12	Rural	Winter	5.5

6.4 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
Residual	Winter	5.5	0.982	Target location A12 therefore US A12 duration adopted for residual.

6.5 Flood estimates from the ReFH2 method

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
US A12	0.299	0.414	0.491	0.569	0.595	0.682	0.791	1.142	0.950	1.282	1.581
Residual	0.064	0.090	0.107	0.124	0.130	0.149	0.173	0.252	0.209	0.283	0.350
DS A12	0.330	0.456	0.541	0.626	0.655	0.750	0.869	1.254	1.044	1.408	1.735

7 Discussion and summary of results

7.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak			
	Return period 20-years / 5% AEP		Return period 100-years / 1% AEP	
	FEH Statistical	ReFH2.3	FEH Statistical	ReFH2.3
US A12	0.326	0.569	0.492	0.791
DS A12	0.366	0.626	0.551	0.869

7.2 Final choice of method

Choice of method and reasons	The final choice of method is ReFH2.3 as it incorporates updates to BFIHOST19 and uses latest rainfall data (FEH13).
How will the flows be applied to a hydraulic model?	The ReFH2.3 design hydrographs will be applied in the hydraulic model. The US A12 hydrographs will be applied within the hydraulic model at the upstream east and west tributary with appropriate scaling factors applied to represent the catchment area of the tributaries. Peak flow estimates will be checked for consistency at the A12. The residual ReFH2.3 hydrographs will be applied as a lateral flow within the hydraulic model between the A12 and the River Blackwater confluence.

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are as follows: • ReFH2.3 is most appropriate for the target catchment. • FEH Statistical is not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 23 / 25. • The solar farm has little impact on design peak flow estimates. • Water is assumed to not back up from the River Blackwater during simulated flood events.
Discuss any particular limitations	None.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	The ReFH2.3 method has been used to derive design peak flows and hydrographs. This method is appropriate but there is inherent uncertainty regarding using default model parameters. The standard approach in this situation is to use default model parameters. If the catchment was gauged the ReFH parameters could be adjusted to reduce the

	uncertainty.
Comment on the suitability of the results for future studies	The design peak flows have been estimated for the purposes of the A12 road improvements project only with the target location being the A12 crossing. Future studies should assess the appropriateness of these peak flow estimates / hydrographs for the purposes of their investigation.
Give any other comments on the study	

7.4 Checks

Are the results consistent, for example at confluences?	Flows are consistent.
What do the results imply regarding the return periods / frequency of floods during the period of record?	N/A ungauged catchment.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The 1 % AEP growth factor at the A12 crossing is 3.0. This is within the expected range.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the 0.1% AEP / 1% AEP is 1.81.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No other studies were available for this minor watercourse.
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	The peak flow estimates will be checked within the model at the A12 crossing (the target location).

7.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
US A12	0.299	0.414	0.491	0.569	0.595	0.682	0.791	1.142	0.950	1.282	1.581
Residual	0.064	0.090	0.107	0.124	0.130	0.149	0.173	0.252	0.209	0.283	0.350
DS A12	0.330	0.456	0.541	0.626	0.655	0.750	0.869	1.254	1.044	1.408	1.735

7.6 Uncertainty bounds

7.6.1 The Flood Estimation Guidelines 197_08 (2020) notes that it is more difficult to quantify uncertainty in design flows estimated from the ReFH rainfall-runoff model than for the FEH Statistical method which it reports approximate uncertainty bounds for. It is therefore highlighted that the final flow estimates are open to uncertainty, but uncertainty bounds have not been provided for this assessment.

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hydrographs are required for the hydraulic modelling of MWC 21A. See: MWC 21A ReFH2.3 Hydrographs 1

Acronyms

Abbreviation	Term
AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

References

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Annex A

A.1 Pooling Group Details

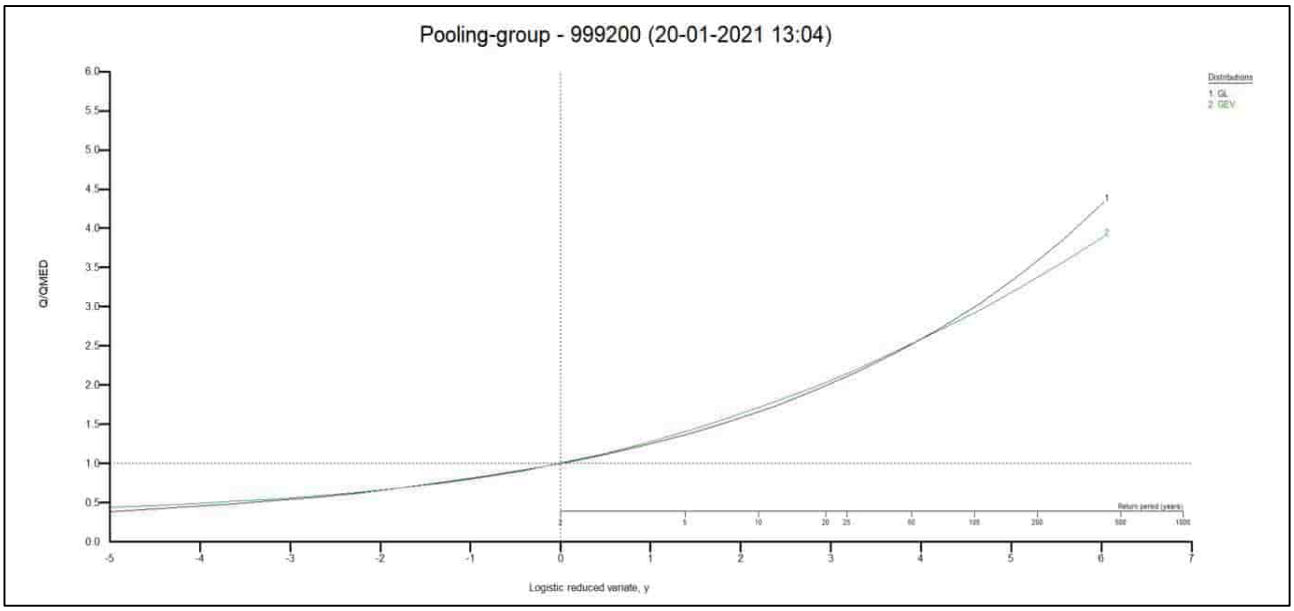
A.1.1 Initial Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.523	42	1.84	0.163	0.301	0.577
27051 (Crimple @ Burn Bridge)	3.278	47	4.524	0.218	0.156	0.357
45816 (Haddeo @ Upton)	3.285	26	3.456	0.3	0.406	0.925
27073 (Brompton Beck @ Snainton Ings)	3.508	39	0.812	0.215	0.035	2.071
28033 (Dove @ Hollinsclough)	3.569	44	4.177	0.228	0.371	0.563
25019 (Leven @ Easby)	4.067	41	5.09	0.342	0.386	1.547
26016 (Gypsey Race @ Kirby Grindalythe)	4.085	22	0.1	0.321	0.266	1.432
91802 (Allt Leachdach @ Intake)	4.107	34	6.35	0.153	0.257	0.677
25011 (Langdon Beck @ Langdon)	4.207	33	15.647	0.232	0.328	0.923
71003 (Croasdale Beck @ Croasdale Flume)	4.225	37	10.9	0.212	0.323	0.233
47022 (Tory Brook @ Newnham Park)	4.227	25	6.176	0.257	0.191	0.559
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	4.247	9	5.777	0.271	0.151	3.026
25003 (Trout Beck @ Moor House)	4.266	46	15.142	0.168	0.29	0.52
106002 (Laxdale @ Laxdale)	4.322	12	17.449	0.103	0.132	1.308
54022 (Severn @ Plynlimon Flume)	4.338	38	14.988	0.156	0.171	1.19
27010 (Hodge Beck @ Bransdale Weir)	4.455	41	9.42	0.224	0.293	0.092
Total		536				
Weighted means				0.221	0.26	

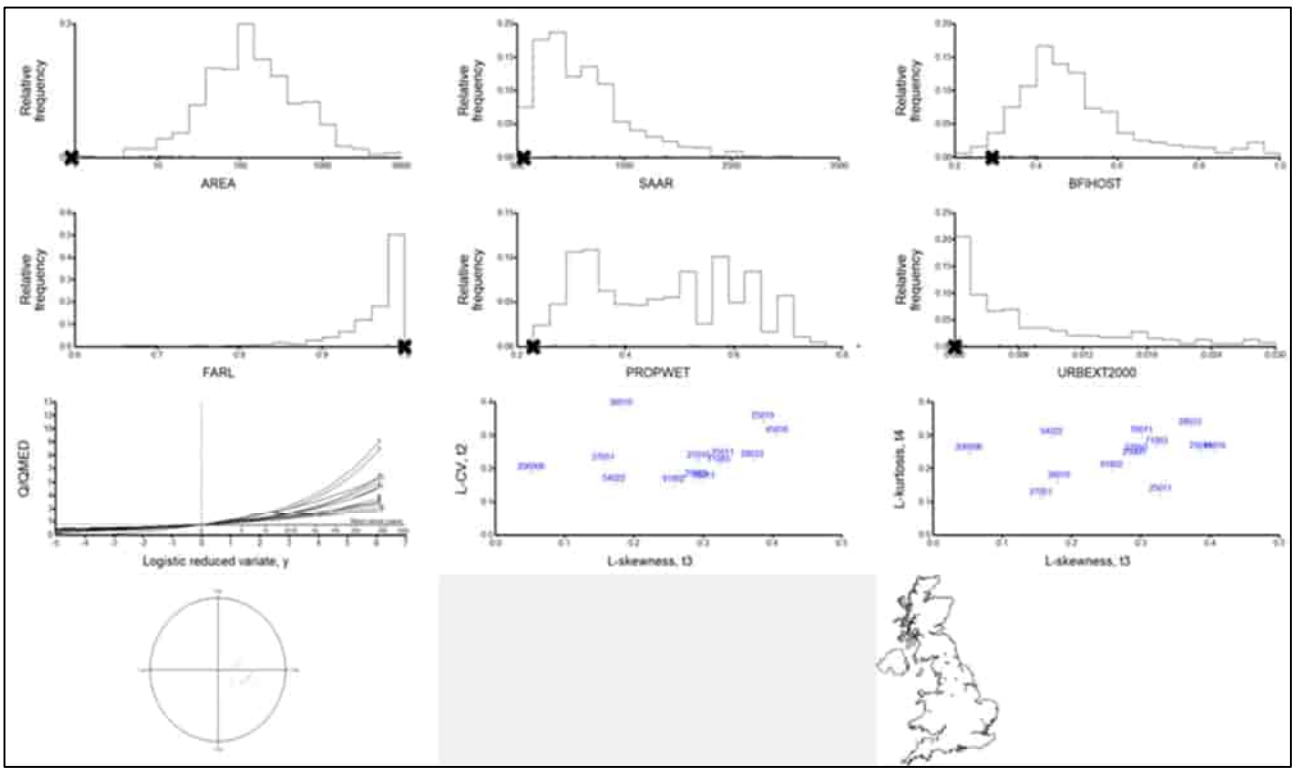
A.1.2 Final Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.523	42	1.84	0.163	0.301	0.579
27051 (Crimple @ Burn Bridge)	3.278	47	4.524	0.218	0.156	1.394
45816 (Haddeo @ Upton)	3.285	26	3.456	0.3	0.406	0.802
28033 (Dove @ Hollinsclough)	3.569	44	4.177	0.228	0.371	0.844
25019 (Leven @ Easby)	4.067	41	5.09	0.342	0.386	1.182
91802 (Allt Leachdach @ Intake)	4.107	34	6.35	0.153	0.257	0.718
25011 (Langdon Beck @ Langdon)	4.207	33	15.647	0.232	0.328	1.6
71003 (Croasdale Beck @ Croasdale Flume)	4.225	37	10.9	0.212	0.323	0.174
25003 (Trout Beck @ Moor House)	4.266	46	15.142	0.168	0.29	0.361
54022 (Severn @ Plynlimon Flume)	4.338	38	14.988	0.156	0.171	0.982
27010 (Hodge Beck @ Bransdale Weir)	4.455	41	9.42	0.224	0.293	0.028
206006 (Annalong @ Recorder)	4.468	48	15.33	0.189	0.052	1.925
36010 (Bumpstead Brook @ Broad Green)	4.793	52	7.395	0.382	0.181	2.412
Total		529				
Weighted means		529		0.226	0.268	

A.2 Final Growth Curve



A.3 Diagnostic Plots



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 21 MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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0 Executive summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at Minor Watercourse 21 (MWC21), NGR grid reference TL863178. The modelling and hydrology analyses have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 2-dimensional (2D) type using TUFLOW software (BMT 2022) and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 40% climate change (CC). No flow reconciliation was required to be carried out for the design hydrology. No data was available for calibration, but the model was verified against the Environment Agency (EA) flood maps. The model was used to simulate baseline and with-scheme scenarios for the three design events mentioned. The with-scheme scenario involved a new side road to be built to the west of the A12 crossing with MWC21, as well as associated earthworks on the upstream side of the carriageway. The extension of the existing culvert, CL21 was required, and the discontinuance of two existing culverts to redirect flow away from the proposed-scheme. Sensitivity analysis of roughness and inflows was carried out to further validate the results.

0.1.3 The baseline (existing) modelling scenario shows the A12 carriageway is widely inundated by flooding. The proposed-scheme impacts were deemed a significant change to baseline scenario; therefore, mitigation works were required. The mitigation measures put in place prevent flooding of the carriageway and new side road, with beneficial impacts to flooded areas to the north of the proposed-scheme near the confluence to the River Blackwater.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios, both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) are:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Minor Watercourses 7, 21 (this report), 21A, 23, 26

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out to assess the baseline and with-scheme scenarios for an unnamed tributary of the River Blackwater. The watercourse is designated as Minor Watercourse 21 based on the A12 drainage design numbering system for the main culvert crossings of the highway. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 The existing A12 crossing of the Minor Watercourse 21 (MWC21) will be retained in the proposed-scheme. Existing highway drainage that connects the two eastern tributaries to the A12 crossing at MWC21, will

be disconnected and reconfigured. Immediately west of the A12 crossing, a new side road will be built, with its associated earthworks on the upstream side of the carriageway. Full details of the with-scheme arrangements are given in Section 7.1.

This report should be read in conjunction with the FRA [R10060/APP/6.6] (National Highways 2022).

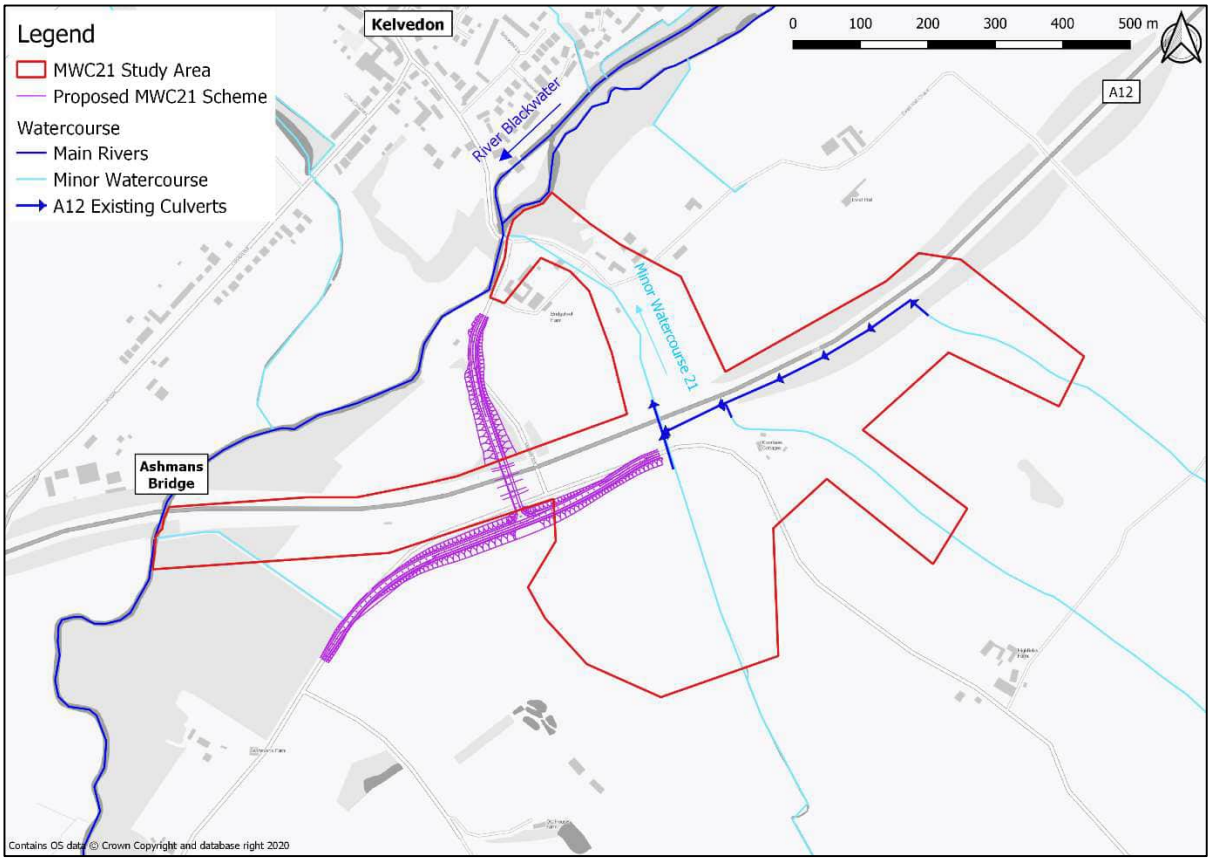
1.2 Study area

1.2.1 MWC21 is situated immediately south of Kelvedon, Essex. The watercourse is composed of three upper reaches, referred to as the western, middle and eastern tributaries, originating approximately 1 to 3km upstream (south-east) of the A12 and flowing in a north-westerly direction (see Plate 1.1). Upstream of the A12, the two smallest (middle and east) tributaries are carried along the toe of the A12 embankment via a culvert of 375mm and then 450mm diameter, before discharging into a confluence chamber and joining the main (western tributary) A12 culvert crossing.

1.2.2 The existing MWC21 A12 crossing conveys flow from south to north under the carriageway and discharges to the downstream open channel reach of MWC21. MWC21 runs northwest for approximately 375m before discharging into the River Blackwater.

1.2.3 The model extent covers an area of 0.33km^2 and includes a total reach length of approximately 2000m.

Plate 1.1 MWC21 Study area



2 Input data

2.1.1 The data used to construct the hydraulic model for MWC21 is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model

Data	Description	Source
LiDAR	1m resolution composite Digital Terrain Model (DTM) from LiDAR (Light Detection and Ranging) 2019 data. Used to inform the hydraulic model with ground level information.	Department for Environment, Food & Rural Affairs (DEFRA), 2021
Ordnance Survey (OS) Mastermap	Land use data used to specify roughness values across the 2D floodplain.	National Highways
Hydrological Analysis	Calculation of inflow hydrographs into the model.	National Highways, 2020
As-built survey	Survey of key hydraulic structures in model	MKSurveys December 2019
CCTV survey	Survey photos and videos of key hydraulic structures in model	National Highways, 2020
Outline Design Drawings	Drawings of the proposed-scheme design.	National Highways, 2021
Flood Zone Mapping	EA Risk of Flooding Surface Water (RoFSW) flood maps. Used for model verification.	Environment Agency

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of MWC21 for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event is also required including an allowance for climate change. For full details of the flow derivation methods and background hydrology refer to Appendix A Flood Risk Assessment Minor Watercourse 21 Hydrology Annex (HE551497-JAC-EWE-SCHW-RP-LE-0088)

3.2 Hydrological considerations

3.2.1 MWC21 is located in South East England just outside Kelvedon and crosses the A12 at approximately NGR: TL 86300 17850. This minor watercourse discharges to the River Blackwater approximately 0.4km downstream of the A12 road crossing. MWC21 has a relatively small catchment area of approximately 3.33km^2 to the downstream model extent which is located just upstream of the River Blackwater confluence. The catchment is classified as essentially rural with a number of dwellings present throughout the catchment. Land use is predominantly agricultural. There is also a solar farm within the upper eastern reaches of the catchment.

3.2.2 The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 571s (Efford 1) and 572q (Ashley) in the lower to middle reaches of the catchment, with 712c (Windsor) being present in the middle to upper reaches of the catchment. The HOST class of the soil groups were assessed and soil type 712c was found to contain HOST class 23 and 25. There are noted issues within the Flood Estimation Guidelines with BFIHOST values for soil which contains HOST 23 and 25 which has been corrected for BFIHOST19. The geology of the catchment was assessed using the Geology of Britain Viewer[1] and found to be composed of London Clay formations consisting of clay, silt and sand.

3.3 Peak flow derivation

3.3.1 MWC21 is a relatively small ungauged catchment and therefore flow estimation is open to great uncertainty. Both ReFH2.3 and FEH Statistical methods were therefore applied to estimate design peak flows for this watercourse. ReFH2.3 resulted in higher design peak flow estimates for MWC21 than the FEH Statistical method. Given ReFH2.3 is more

[1] British Geological Survey. The Geology of Britain Viewer accessed 2021 at: http://mapapps.bgs.ac.uk/geologyofbritain/home.html

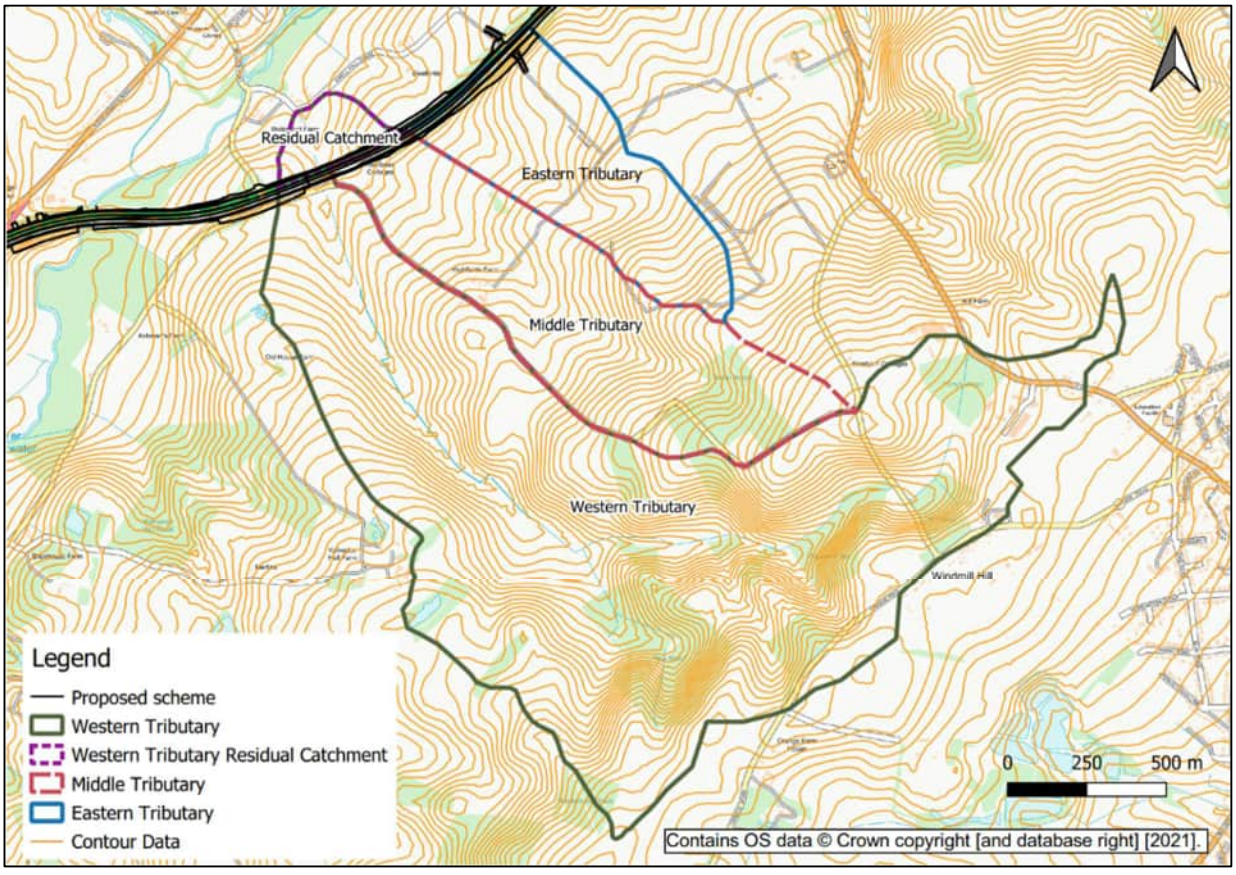
conservative and as the FEH Statistical method uses BFIHOST which has known issues where the soils contain HOST Class 23 and 25, the ReFH2.3 method was selected for final peak flows estimates.

3.4 Hydraulic model inflows

3.4.1 For the purposes of hydraulic modelling the MWC21 catchment was split into three flow estimation points. Flows were derived for the western catchment, the middle catchment and the eastern catchment. The flow estimates for the western catchment were then area scaled so 97% of the total inflow was applied upstream of the A12 at the start of the hydraulic model and the remaining 3% of the flow was applied as a lateral flow downstream of the A12 crossing. Point inflow were also applied within the hydraulic model at the start of both the eastern and middle catchment modelled reaches.

3.4.2 A schematisation of the MWC21 catchment is presented in Plate 3.1.

Plate 3.1 Schematisation of the MWC21 catchment



3.4.3 The adopted final design peak flow estimates for the western tributary, middle tributary and eastern tributary catchments for the individual catchment critical storm duration (CSD) are presented in Table 3.1.

Table 3.1 MWC21 Peak flow estimates for the western, middle and eastern tributaries for the individual catchment CSD

Inflow	Peak Flow (m^3/s)										
	50% AEP (1 in 2)	20% AEP (1 in 5)	10% AEP (1 in 10)	5% AEP (1 in 20)	3.33% AEP (1 in 30)	2% AEP (1 in 50)	1.33% AEP (1 in 75)	1% AEP (1 in 100)	1% AEP +CC 40% uplift (1 in 100)	0.5% AEP (1 in 200)	0.1% AEP (1 in 1000)
Western Tributary (CSD = 9hrs)	0.80	1.08	1.27	1.46	1.58	1.74	1.89	2.02	2.91	2.46	4.08
Middle Tributary (CSD = 4.5hrs)	0.28	0.38	0.46	0.53	0.58	0.64	0.69	0.73	1.06	0.88	1.45
Eastern Tributary (CSD = 4.5hrs)	0.18	0.26	0.30	0.35	0.38	0.43	0.46	0.49	0.71	0.59	0.98

3.5 Model runs and critical storm duration

3.5.1 Introduction

3.5.2 To assess the CSD for MWC21 two model runs were required. Run 1 was undertaken using the CSD, ARF and SCF to the downstream model extent. Run 2 was undertaken using the individual catchment CSD areal reduction factor (ARF) and seasonal correction factor (SCF). Further details on the hydrology of the model runs and associated peak flow estimates are provided below.

3.5.3 Run 1

For Run 1 all inflows for all sub-catchments were derived using the CSD, ARF and SCF relevant to the catchment to the downstream model extent. This run resulted in the following peak flow estimates for the modelled events.

Table 3.2 Run 1 Peak Flow Estimates (Storm Duration = 9hrs)

Inflows	Peak flows (m^3/s)		
	AEP 5% (1 in 20)	AEP 1% (1 in 100)	AEP 1% + 40% CC uplift (1 in 100)
Western Tributary	1.46	2.02	2.91
Middle Tributary	0.52	0.73	1.05
Eastern Tributary	0.34	0.48	0.69

Table 3.2 shows that the western tributary peak flows match with those in Table 3.1 as these are for the CSD of this tributary (9 hrs). However, the peaks for the other two tributaries (middle / lower tributaries) are lower than those shown in Table 3.1 as these flows are derived for a longer storm duration than their individual CSD.

3.5.4 Run 2

For Run 2 all inflows were derived using the individual CSD relevant to the individual tributaries. Run 2 resulted in the following peak flow estimates for the modelled events.

Table 3.3 Run 2 Peak Flow Estimates

Inflows	Peak flows (m^3/s)		
	AEP 5% (1 in 20)	AEP 1% (1 in 100)	AEP 1% + 40% CC uplift (1 in 100)
Western Tributary (CSD = 9hrs)	1.46	2.02	2.91
Middle Tributary (CSD = 4.5hrs)	0.53	0.73	1.06
Eastern Tributary (CSD = 4.5hrs)	0.35	0.49	0.71

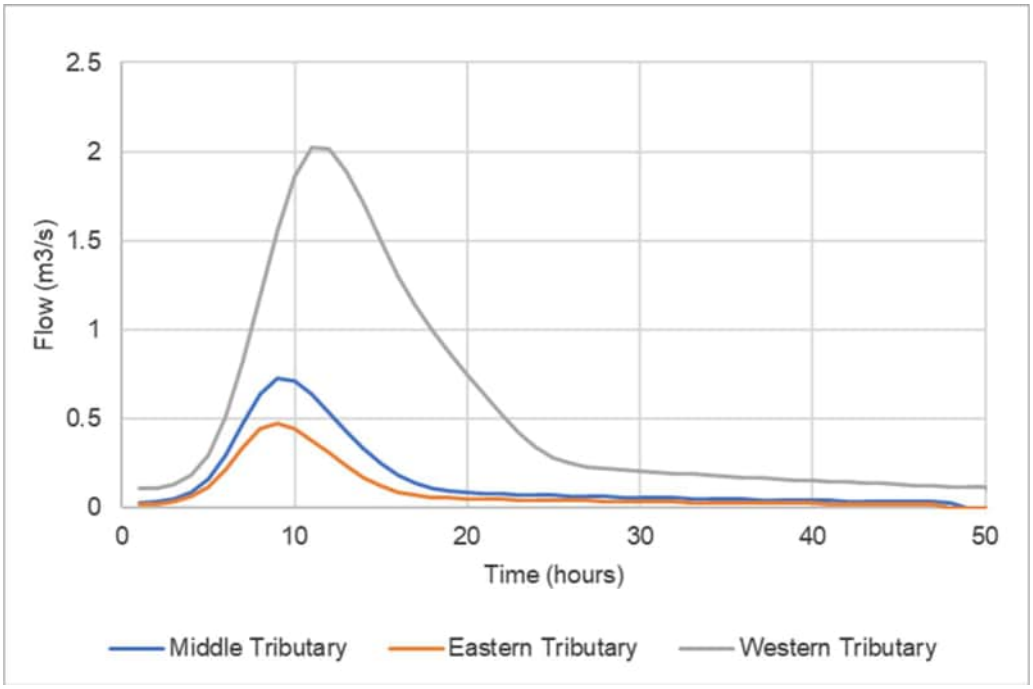
Table 3.3 shows the same peak flow estimates to those shown in Table 3.1 as these peak flow estimates are both derived for the individual CSD.

3.6 Inflow Hydrograph Shape

3.6.1 As the MWC21 catchment is ungauged it was assessed as appropriate to derive hydrograph shapes for input into the hydraulic model using ReFH2.3. For Run 1 the ReFH2.3 hydrograph shapes were derived for all catchments using the CSD, ARF and SCF to the downstream model extent. This resulted in all sub-catchments having a CSD of 9-hours. For Run 2 the ReFH2.3 hydrograph shapes were derived at the A12 crossing (the target location) for the middle and eastern tributaries and to the downstream model extent (just downstream of the A12 crossing) for the western tributary. This resulted in a 4.5-hour CSD for the middle and eastern catchments and 9-hour CSD for the western catchment.

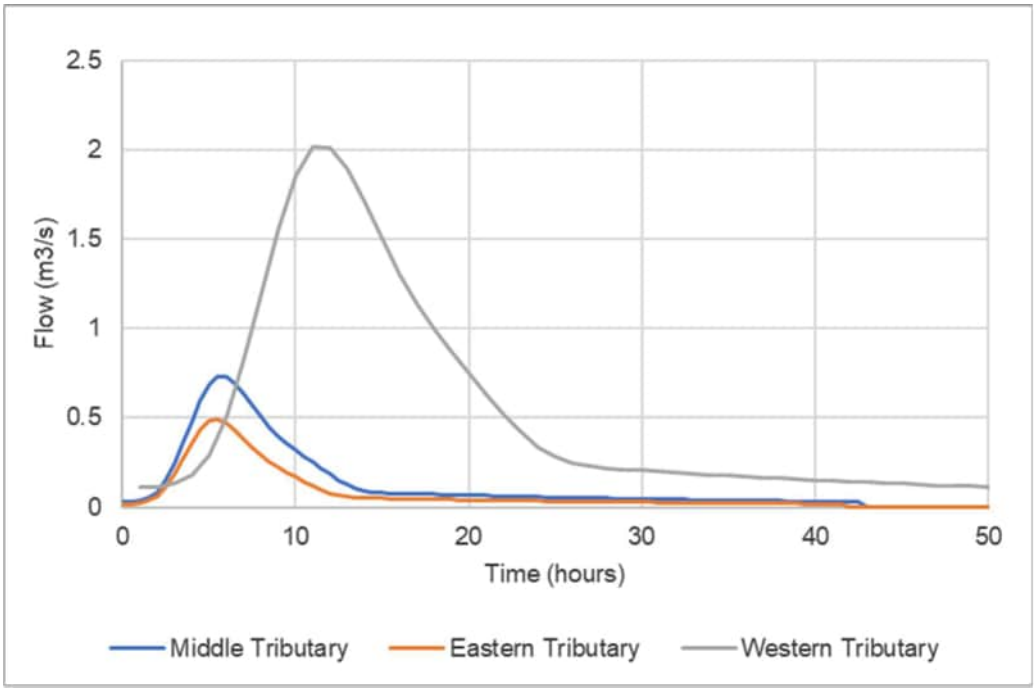
3.6.2 The Run 1 1% (1 in 100) AEP event hydrographs for the western, middle, and eastern catchments are presented in Plate 3.2.

Plate 3.2 Run 1 - 1% (1 in 100) AEP hydrographs



3.6.3 The Run 2 1% (1 in 100) AEP event hydrographs for the western, middle and eastern catchments are presented in Plate 3.3.

Plate 3.3 Run 2 - 1% (1 in 100) AEP hydrographs



3.7 Climate change

3.7.1 A climate change allowance for peak rainfall intensity was obtained from the latest government guidance at the time of assessment (Environment Agency, 2021). The peak rainfall intensity allowance is appropriate for small catchments (less than 5km^2). An uplift of 40% (the upper end, 2080s scenario) was therefore applied. For Run 1 the peak for the 40% uplift is 2.91m^3/s for the western tributary, 1.05m^3/s for the middle tributary and 0.689m^3/s for the eastern tributary. For Run 2 the peak for the 40% uplift is 2.91m^3/s for the western tributary, 1.06m^3/s for the middle tributary and 0.709m^3/s for the eastern tributary.

3.8 Flow reconciliation

3.8.1 Given the model inflows were estimated to or in close proximity to the A12 crossing (target location) of MWC21 using the ReFH2.3 method, flow reconciliation is not considered necessary for this minor watercourse.

3.9 Summary

3.9.1 Design peak flows and hydrographs for MWC21 were derived using ReFH2.3, using the FEH catchment descriptors available from the FEH Web Service and adopting climate change allowance as per latest EA guidance. Design inflow hydrographs were derived for two hydraulic model runs, namely Run 1 – for the longer critical storm duration to the downstream model extent and Run 2 – for the individual critical storm duration for each tributary. After undertaking the hydraulic analysis with both sets of inflow hydrographs, it was noted that Run 1 resulted in a moderate increase in water levels downstream of the A12 in comparison to Run 2 (however with a slight (-0.05m^3/s) reduction in peak flow through the A12 cross culverts). The results of Run 1 have therefore been presented. For further details refer to Section 5.

4 Baseline modelling

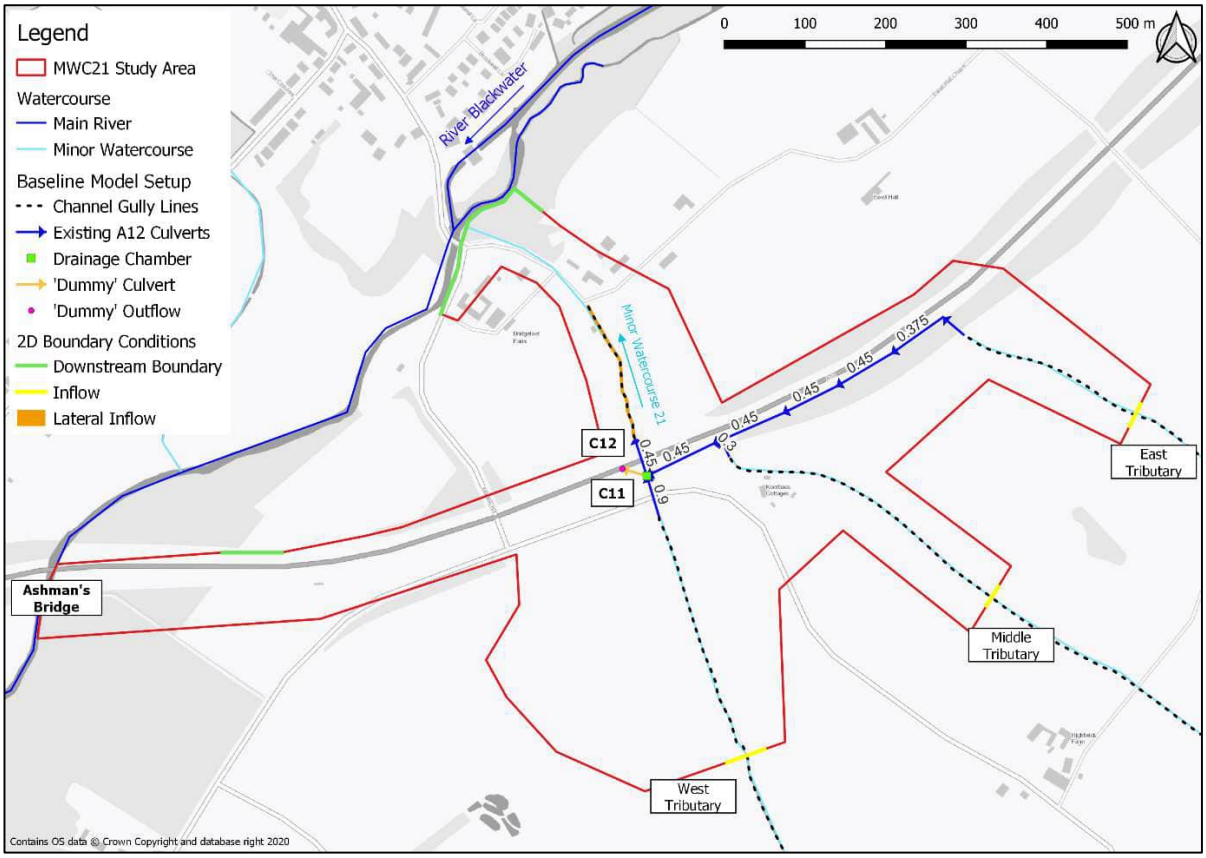
4.1.1 The MWC21 hydraulic model uses a two-dimensional (2D) schematisation, built in TUFLOW version 2020-10-AA (BMT, 2020). 1D 'ESTRY' elements were used to represent the culverts within the TUFLOW model and linked to the 2D domain via SX links. The 2D representation of the channel was considered appropriate as the highway culvert controls pass forward flows and upstream water levels, therefore the detailed representation of the channel capacity is not required.

4.1.2 The model was built from scratch and used to simulate the baseline, with-scheme and mitigation scenarios. Three design events were simulated for each scenario: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) AEP + 40%CC.

4.2 Model extent

4.2.1 The 2D model domain covers an area of 0.33km^2. The model extent has been drawn in a way to capture the floodplains of each tributary and the downstream watercourse, and an overland flow route running west along the A12 highway. Representation of the model extent is shown in Plate 4.1, and model configuration is explained in detail in Section 4.3.

Plate 4.1 MWC21 TUFLOW model setup



4.3 Model resolution and topography

4.3.1 1m horizontal resolution LiDAR DTM data from 2019 was available for the model extent and was downloaded from the DEFRA website to inform the hydraulic model with ground level information.

4.3.2 A 1m grid cell resolution was deemed appropriate for the model, as this is sufficient to represent the open channel, the floodplain topography and the active floodplain details while avoiding a long simulation run time. The model grid has been oriented to aligned with key flow paths.

4.3.3 The open channel sections were represented in the model using 2m thick gully lines to define channel width, whereby the riverbed levels have been reinforced using selected LiDAR data, based on inspection of the 1m LiDAR DTM and aerial photography. Bridge structures downstream of the A12 have been ignored, as investigation shows that these features would not affect the model results within the area of interest.

4.3.4 The watercourse crosses the A12 via three separate inlets. The eastern tributary enters a 375mm pipe before joining a 450mm highway surface carrier drain. Further west (downstream) the middle tributary goes into culvert via a 300mm pipe and connects to the main 450mm carrier pipe. The Main carrier pipe finally discharges into a large ($10m^3$) confluence chamber at the proximity of the western tributary crossing. The western tributary culverts consist of:

- A 900mm culvert (C11) on the northern side of A12, which discharges into the confluence chamber described above.
- Leaving the chamber, flows are conveyed north via a circular culvert (CL12) of 450mm diameter before discharging into MWC21.

4.3.5 At the chamber, an additional 600mm diameter bypass culvert conveys flows exceeding CL12 capacity into the main highway drainage, which runs westwards until discharging into the River Blackwater upstream of Ashman Bridge. No survey information was available for the continued main highway drainage. Therefore, a representative ‘dummy’ 600mm circular culvert and a 1D outflow boundary was used, as shown in Plate 4.1. This allows for the secondary outflow to be represented without explicit representation of the wider drainage network.

4.3.6 MWC21 runs north-westerly for approximately 375m before discharging into the River Blackwater. Culverts were schematised in the model using embedded 1D ‘Estry’ elements with a Manning’s ‘n’ roughness value of 0.013, which is a mid-value for either metal or concrete culvert types (Chow 1959). Culvert losses are applied based on standard recommended values; 0.5 and 1 for the inlet and outlet loss, respectively.

4.3.7 The confluence chamber dimensions modelled in ‘Estry’ were estimated from CCTV photos and video provided by National Highways, as no drawings were available.

4.4 Hydraulic Friction

4.4.1 OS MasterMap data was used to identify land use type and inform the TUFLOW model with different hydraulic roughness (Manning's 'n' coefficient) values. Hydraulic roughness coefficients are applied over each grid cell of the 2D domain depending on the land use taken from the MasterMap data, as shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow, 1959).

4.4.2 As mentioned in Section 4.3 above, all culverts were modelled assuming a roughness value of 0.013, taken from Chow (1959) as a mid-value for either concrete or metal culverts that are straight and free of debris.

Table 4.1 Manning's 'n' coefficients – 2D domain

Land use	Manning's 'n'
Buildings	0.2
Inland water	0.025
Channel	0.04
General surface	0.05
Natural environment (coniferous/non coniferous trees) heavy woodland and forest	0.075
Roads, tracks and paths	0.02
Roadside	0.04
Structures	0.025
Land (unclassified), industrial yards, car parks	0.035
Slope	0.040
Stability Patch	0.1

4.5 Boundary conditions

4.5.1 Three types of boundary conditions have been implemented in the model and can be seen on Plate 4.1.

- QT boundaries were applied to the 2D domain at the upstream ends of the three tributaries. A lateral inflow was applied to the channel immediately downstream of the existing A12 crossing. The catchment hydrology analysis is explained in Section 3.
- A constant water level (HT) boundary equal to the River Blackwater levels for the 5% (1 in 20) AEP event was implemented at the downstream end of the MWC21 to allow flow out of model.
- A water level vs flow (HQ) boundary was implemented to the west of the model extent adjacent to the A12, at the low point of the highway, to allow channel flows out of the model and prevent ‘glass walling’ of water. The boundary applies a normal depth assumption with slope determined by inspection of the LiDAR data.

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 40% uplift in accordance with the latest EA guidance as detailed in Section 3.

5.1.2 As discussed in Section 3, two model runs were required to assess the critical storm duration for MWC21. The two runs were compared for all three events using the baseline model. Run 1 resulted in a moderate increase in water levels downstream of the A12, however with a slight (-0.05m^3/s) reduction in peak flow through the A12 cross culverts. Model results were not very sensitive to the "run scenario". However, for a conservative assessment, Run 1 was utilised in the analysis going forward.

5.1.3 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, and hydrological inflows (see Section 6.4).

Table 5.1 Modelled events for MWC21

Scenario	5%AEP (1 in 20)	1%AEP (1 in 100)	1%AEP +40%CC (1 in 100)
Baseline	✓	✓	✓
Design	✓	✓	✓
Mitigation	✓	✓	✓
Roughness Sensitivity		✓	
Hydrological Inflow Sensitivity		✓	

6 Model proving

6.1 Introduction

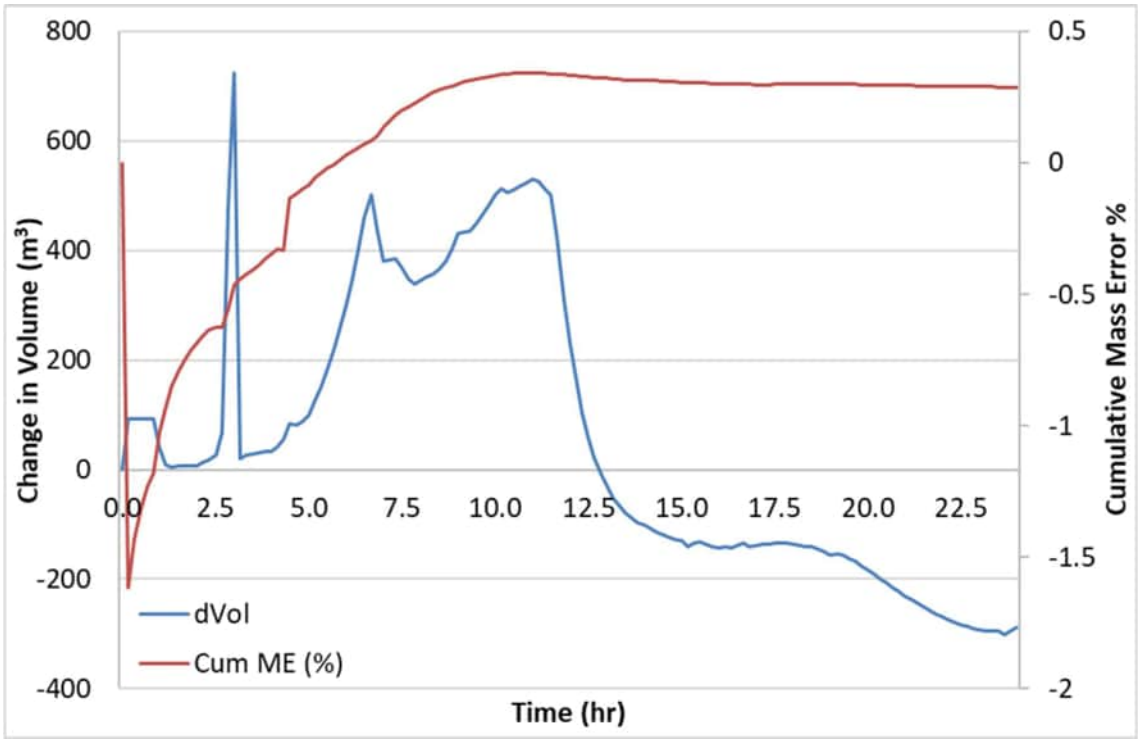
6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model computational performance was achieved. The cumulative mass error diagnostics output from the model have been checked for each model simulation. Plate 6.1 shows the cumulative mass error (Cum ME) and the change in volume (dVol) for the baseline 1% (1 in 100) AEP event simulation. This plot is typical for all the events and scenarios simulated. The tolerance range recommended by the software manual is +/-1% mass balance error.

6.2.2 The cumulative mass error tolerance is exceeded at the onset of the floodplain wetting but stabilises to less than 0.5% error well before the peak of the inflow hydrographs (Plate 3.2) and the corresponding dVol.

Plate 6.1 Model proving cumulative ME and dVol plot for 1% (1 in 100) AEP event



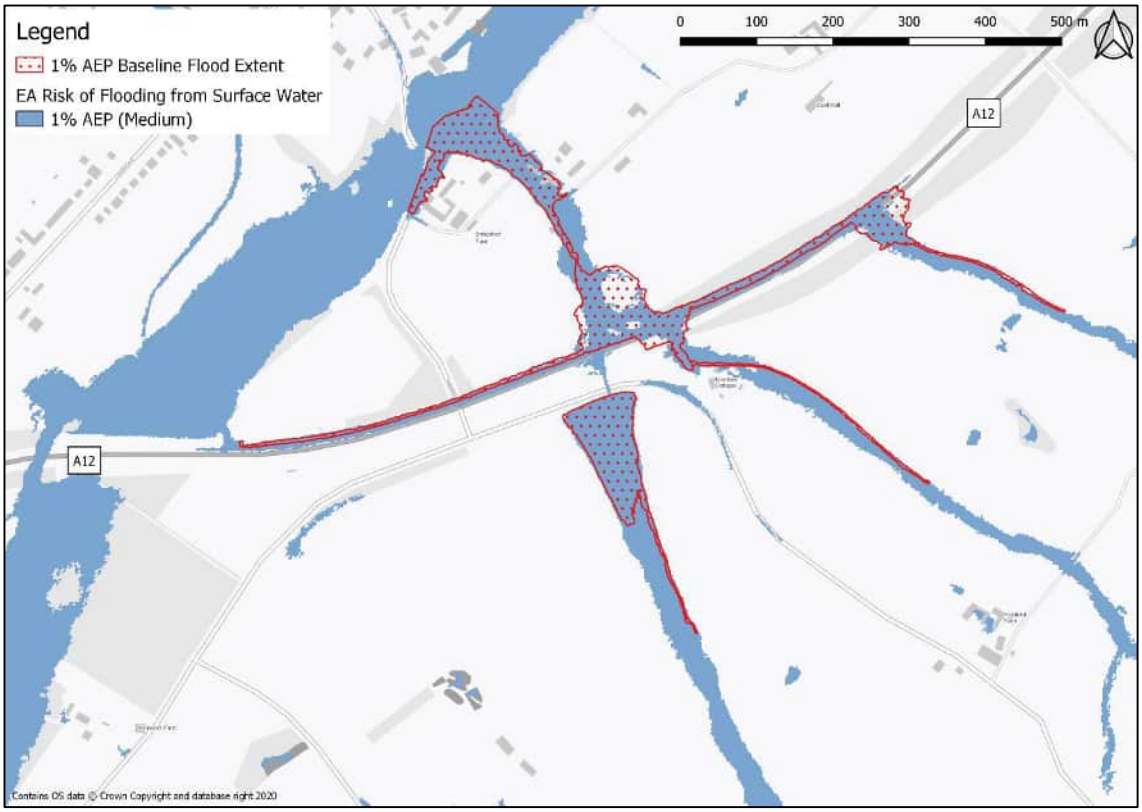
6.3 Comparison with EA flood maps

6.3.1 MWC21 has not been included in the Environment Agency (EA) published Flood Zone mapping of fluvial flood risk. Therefore, a direct comparison between model results and EA flood maps is not possible. However, the EA Risk of Flooding from Surface Water (RoFSW) mapping can be used to identify areas of pooling.

6.3.2 Plate 6.2 shows the modelled 1% (1 in 100) AEP event flood extent and the EA RoFSW mapping extents. The EA RoFSW map shows greater flooding from the tributary watercourses within the study area. Similar inundation along the northern verge of A12 and the downstream reach of MWC21 is present in both modelled flood extent and EA flood map.

6.3.3 There is limited data available for verification of the MWC21 model, as the Environment Agency flood map shows a larger flood extent than the model. Furthermore, overtopping of the A12 carriage is evident in the RoFSW mapping because the existing culverts across the A12 have not been represented. Considering this, verification of the model cannot be based on the comparison of results with EA flood maps. The analysis is therefore reliant on the use of industry standard modelling and hydrology tools, as is common for small watercourse studies.

Plate 6.2 EA risk of flooding from surface water and 1% (1 in 100) AEP extent



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness and hydrological inflows. All sensitivity tests were performed using the 1% (1 in 100) AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Appendix B shows the impact of changing the model roughness on predicted peak water levels. The results show that upstream of the west tributary, the floodplain is relatively sensitive to manning's changes, with a change in modelled water level of +100mm and -200mm when the Manning's value is increased and decreased respectively. The in-channel water levels downstream of the A12, and the A12 road drain show moderate sensitivity to changes in roughness. A 20% increase in roughness slightly increases the 1% (1 in 100) AEP flood extent of the storage upstream on the west tributary.

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Appendix B shows the impact of changing the model inflows on predicted peak water levels. The results show that the west tributary floodplain upstream of A12 is very sensitive to change in inflows, with a water level change of +400mm and -600mm respectively.

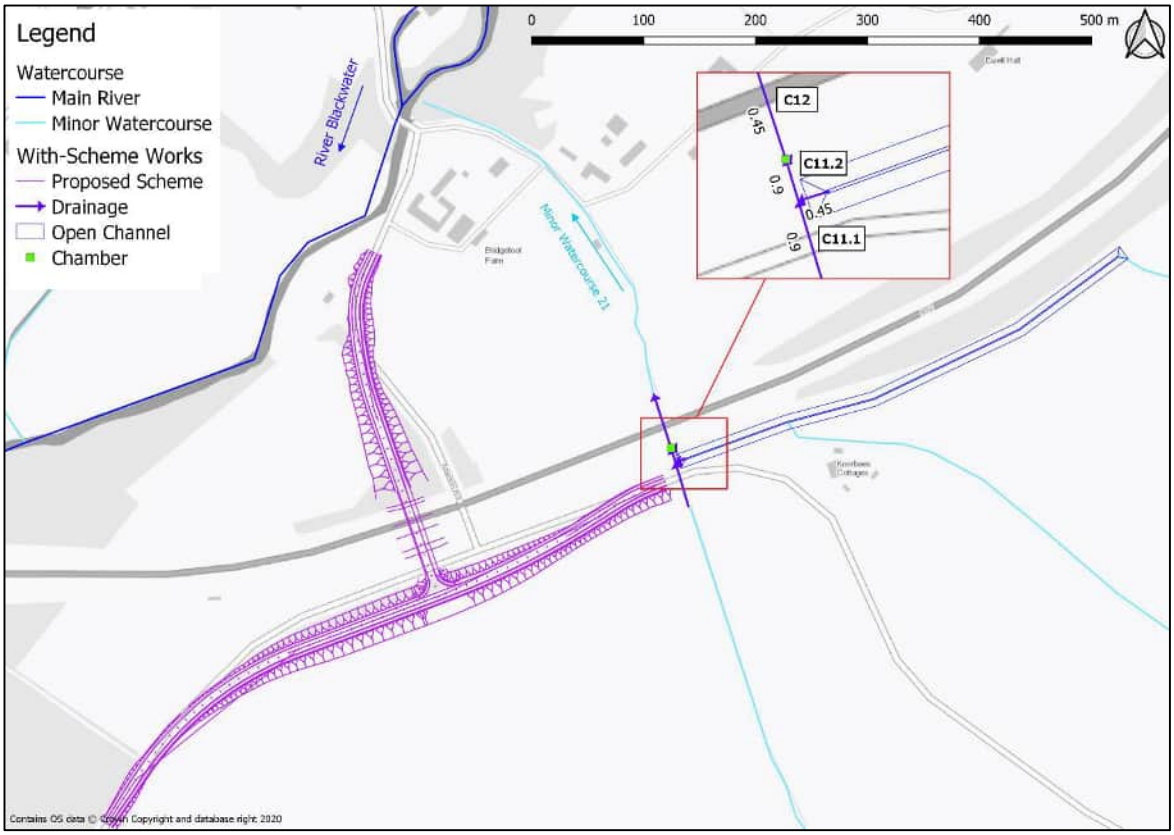
7 Design modelling

7.1 Design model build

7.1.1 The baseline model was updated to include the proposed A12 scheme. Changes to the model were as follows and illustrated in Plate 7.1:

- The proposed A12 side road design was included in the model as modification to the baseline model topography using an Ascii DTM surface overlain.
- The middle and eastern tributaries were disconnected from existing drainage network and redirected via a 2m wide (base) to 10m wide (top) trapezoidal open channel running along the toe of the A12 embankment. Elevations for the channel determined by inspection of the LiDAR data, and bed slopes were set to match the existing arrangements as much as possible. The new channel outfalls via a 450mm concrete circular culvert into a new drainage chamber located further south (upstream). The new chamber is also in line with the C11 culvert.
- The existing confluence chamber and the southern A12 crossing culvert C12, remain unchanged.
- Where the existing drainage system continues west along A12, the ‘dummy’ culvert and outflow boundary were removed.
- The Manning’s roughness layer was updated to reflect the proposed open channel connecting the east and middle tributaries to the A12 crossing. Manning’s ‘n’ values were updated as per Table 4.1.

Plate 7.1 Design model updates



8 Model results

8.1 Baseline results

8.1.1 Plate 8.1 shows the maximum flood extent for all modelled events in the baseline scenario. The flood extents for each of the modelled events can be found in Appendix C.

8.1.2 There is flooding on the A12 from the middle and eastern tributaries in all modelled events. Flooding of the carriageway creates a flow path on the northern verge of the A12 which flows west towards the River Blackwater.

8.1.3 Significant floodplain storage occurs upstream of the A12 crossing on the western tributary, where flows are locally constricted by Highfields Lane and C11 for all AEP events. In the 1% (1 in 100) AEP +40%CC event, flooding overtops Highfields Lane to the east, converging with the middle tributary, where it overtops the A12 carriage way.

8.1.4 It is evident in the overtopping of the A12 carriageway, that the culvert structures at the middle and eastern tributaries are undersized and insufficient in conveying in-channel flows. The middle culvert is seen surcharging in all modelled flood events.

8.1.5 There is consistent out of channel flow for all modelled events immediately downstream of the A12 crossing. At Ewell Hall Chase flooding occurs to two private properties at the confluence of MWC21 and the River Blackwater, by up to 100mm depth.

Plate 8.1 Baseline Flood Extents



8.2 With-scheme model results

8.2.1 Plate 8.3 shows the changes to the water level due to the proposed A12 scheme for the 1% (1 in 100) AEP +40%CC event. The maximum flood level difference between baseline and with-scheme are categorised into levels of impact due to with-scheme. The impact of the proposed-scheme for all other flood events can be found in Appendix C. In all events, the extent of flooding has been increased due to the removal of the connections to the existing A12 drainage culverts. This has caused multiple run-on effects due to the decrease in available downstream capacity across the highway, causing the existing C11 culverts, to reach maximum capacity at a lower flow rate.

8.2.2 Due to decreased culvert capacity, the maximum modelled water level at the western tributary storage is increased, by up to 1.2m in the 5% (1 in 20) AEP event. Overtopping of Highfield Lane to the east occurs in all modelled events, which flows towards the middle tributary exacerbating the flood risk to the A12, inundating the road carriageway up to 200mm deep.

8.2.3 Where the proposed design channel meets with the A12 crossing, the outfall structure is surcharging in all modelled events, resulting in mass overtopping of the A12, with depths up to 150mm, in all events.

8.2.4 Removal of the ‘dummy’ outlet, and figuratively the western A12 drainage network, has resulted in a large increase in surface flows draining west along the A12 carriageway towards the River Blackwater as shown in Plate 8.2. The A12 roadway is inundated up to 250mm deep.

8.2.5 In-channel modelled water levels downstream from the scheme have increased by approximately 100mm, with impacts shown to extent 300m north to the River Blackwater. Private properties that were previously inundated in the baseline scenario, show an increase in water levels in all with-scheme events. Increases of up to 45mm are seen in the 1% + 40% CC (1 in 100) AEP event. There are no adverse impacts seen at the downstream boundary at the River Blackwater.

8.2.6 The implementation of the scheme removes the flooding of the A12 highway from the eastern tributary that was shown in the baseline case.

Plate 8.2 With-scheme flood extents

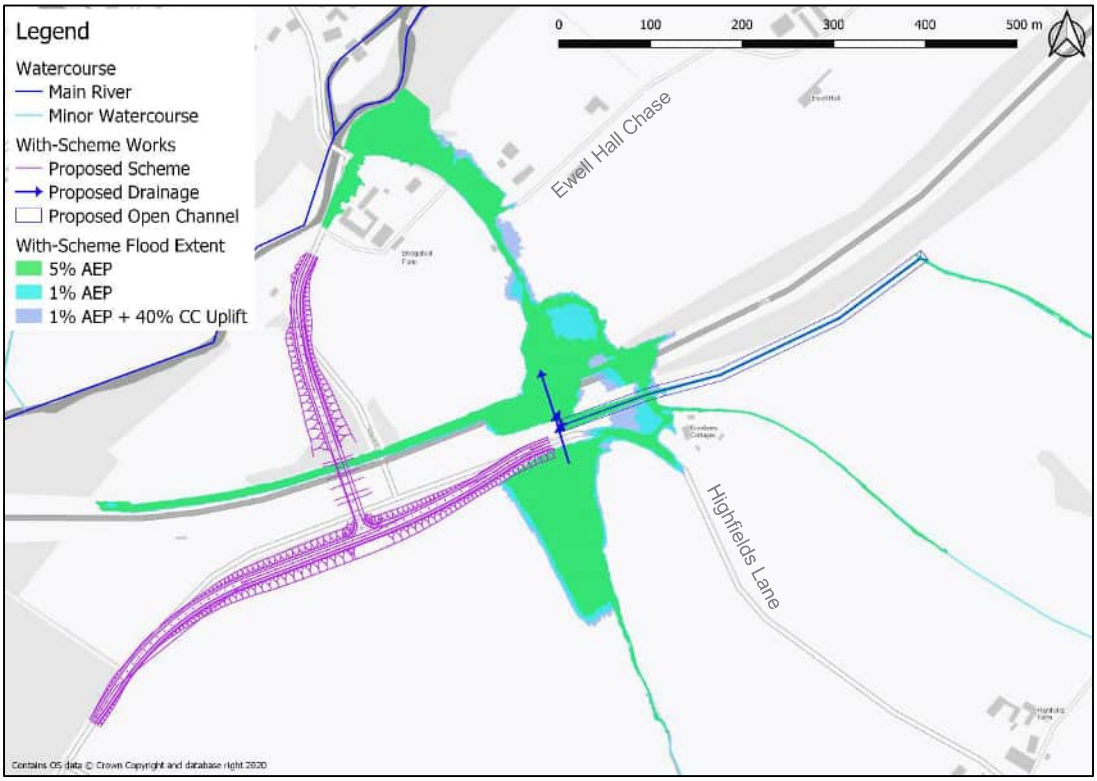


Plate 8.3 1% (1 in 100) AEP +40%CC Water level difference (with-scheme – baseline)



9 Mitigation modelling

9.1 Mitigation model build

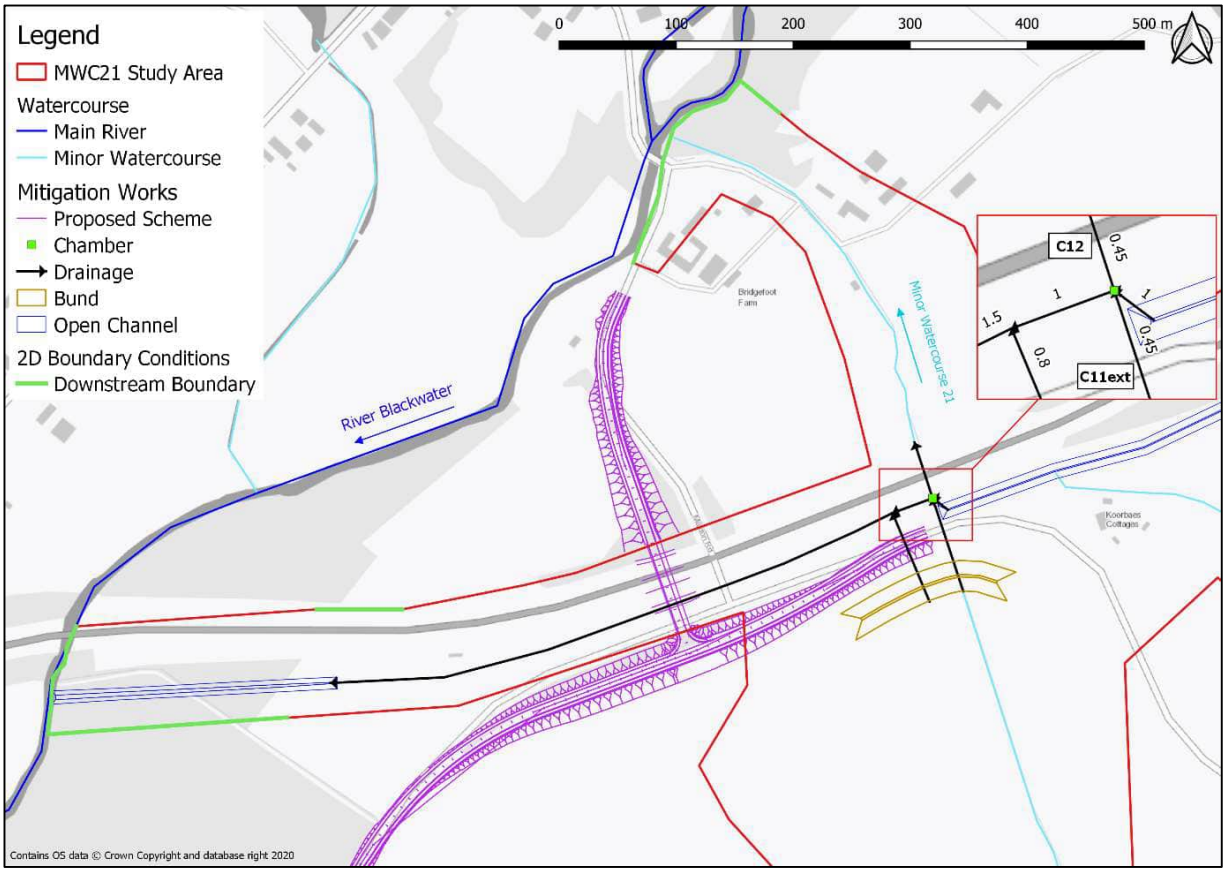
9.1.1 The with-scheme model was updated to include the required mitigation measures to address the impacts resulting from the proposed-scheme.

9.1.2 Many mitigation options were considered and trialled. This presented the option that was found to provide both A12 flooding immunity and prevent adverse impacts to private properties downstream.

9.1.3 Changes to the model were as follows and illustrated in Plate 9.1:

- A bund, with a crest elevation of 26.5m AOD, was added across the west tributary channel and floodplain, upstream of the proposed A12 crossing, to prevent flows from the west tributary ponding against the A12 embankment or overtopping the scheme.
- C11 was extended 30m to the south, on its upstream end, so that the inlet is at the upstream face of the proposed bund, and its dimension remain unchanged (900mm diameter). The with-scheme (new) chamber and 450 mm diameter CL12 pipe connected downstream of it remain unchanged in the mitigation model.
- A further concrete outlet pipe with a diameter of 800mm is added to allow drainage from the left floodplain, this acts as a high-level bypass to the River Blackwater, taking excess water from storage during high peak flow events.
- The connection pipe for the proposed eastern open channel into the new chamber has been increased to a 1m diameter concrete pipe. No modifications were made to the with-scheme open channel geometry.
- A new flood mitigation drainage system was introduced. A 1m diameter concrete circular culvert connects the new chamber to the flood mitigation reservoir bypass pipe system. Downstream (west) of this connection, the new drainage system consists of a proposed 1.5m diameter concrete circular pipe of 500m length, that travels west alongside the south edge of the A12 Highway. The pipe outfalls into a man-made channel of approximately 200m length running westwards through the River Blackwater left floodplain and discharges into the Blackwater immediately downstream (south) of Ashman Bridge. The model topography was modified to represent the channel required to discharge flow into River Blackwater.
- Two additional downstream boundaries were added at the western model edge to allow for flow out of the model and into the River Blackwater. Along the River Blackwater, a constant water level (HT) boundary equal to the River Blackwater levels for the 5% (1 in 20) AEP event was implemented.

Plate 9.1 MWC21 mitigation setup



9.2 Mitigation model results

9.2.1 Plate 9.3 shows the changes to the water level due to the mitigation works for the 1% (1 in 100) AEP +40%CC event. The maximum flood level difference between baseline and “with-scheme plus mitigation” are categorised into levels of impact. The impact of the proposed-scheme for all other flood events can be found in Appendix D.

9.2.2 The proposed-scheme remains dry for all AEP events as demonstrated by Plate 9.2.

9.2.3 Most notably are the impacts seen at the River Blackwater to the west. Mitigation modelling is presenting adverse impacts due to a combination of downstream boundary conditions from the River Blackwater 5% (1 in 20) AEP water levels, and MWC21 additional flows from the proposed mitigation pipe. This drain and accompanying floodplain are represented within the Middle Blackwater model and are seen to be inundated in all modelled events, therefore the MWC21 scheme impacts in this area are expected to be minor. The River Blackwater is a large catchment of approximately 350m2, a small diversion of 2m3/s to a location on the river 800m downstream of the existing confluence will have negligible impact on water levels in the River Blackwater. For more detailed discussion, refer to the FRA [R10060/APP/6.6] (National Highways 2022).

9.2.4 The bund and additional 800mm outlet pipe directing stored flows west towards the River Blackwater prevent ponding against the proposed A12 embankment in all events. These mitigation measures also prevent the overtopping of Highfields Lane, and ultimately the A12 adjacent to the middle tributary.

9.2.5 Peak flow through the existing culvert C11 is lower than in the baseline and with-scheme scenarios. This is because water in the western tributary storage area is also being drained by the proposed high level transfer pipe, diverting a portion of flows directly westwards to the River Blackwater.

9.2.6 The northern boundary of western tributary storage area has been displaced 34m south by the proposed mitigation bund. However, the mitigation works do not cause any significant change to the flood extent further south. Overall, the mitigation works act to reduce flood extents in the larger 1% and 1% +40%CC (1 in 100) AEP events.

9.2.7 Within the northern River Blackwater downstream boundary floodplain there is a reduction in flood levels including the property locations, which have reduced by up to 0.05m, and one property footprint is no longer inundated in any of the modelled events. Negligible impacts are seen at the downstream boundary with the River Blackwater.

Plate 9.2 Mitigation flood extents

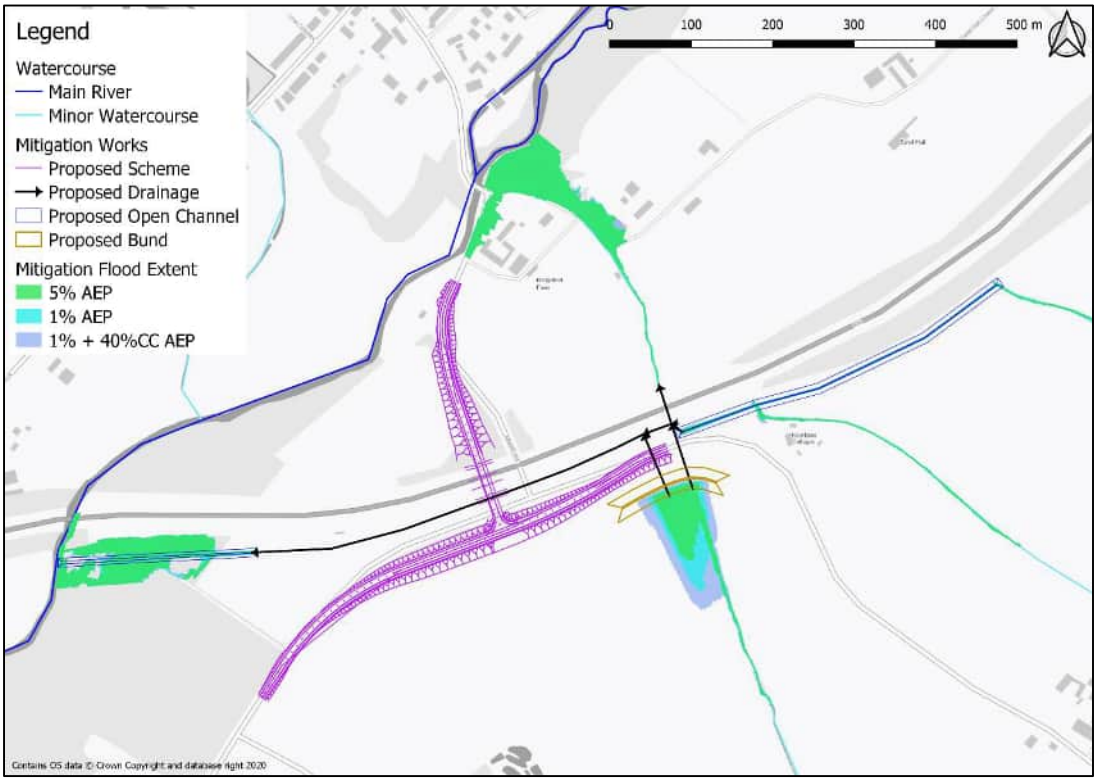
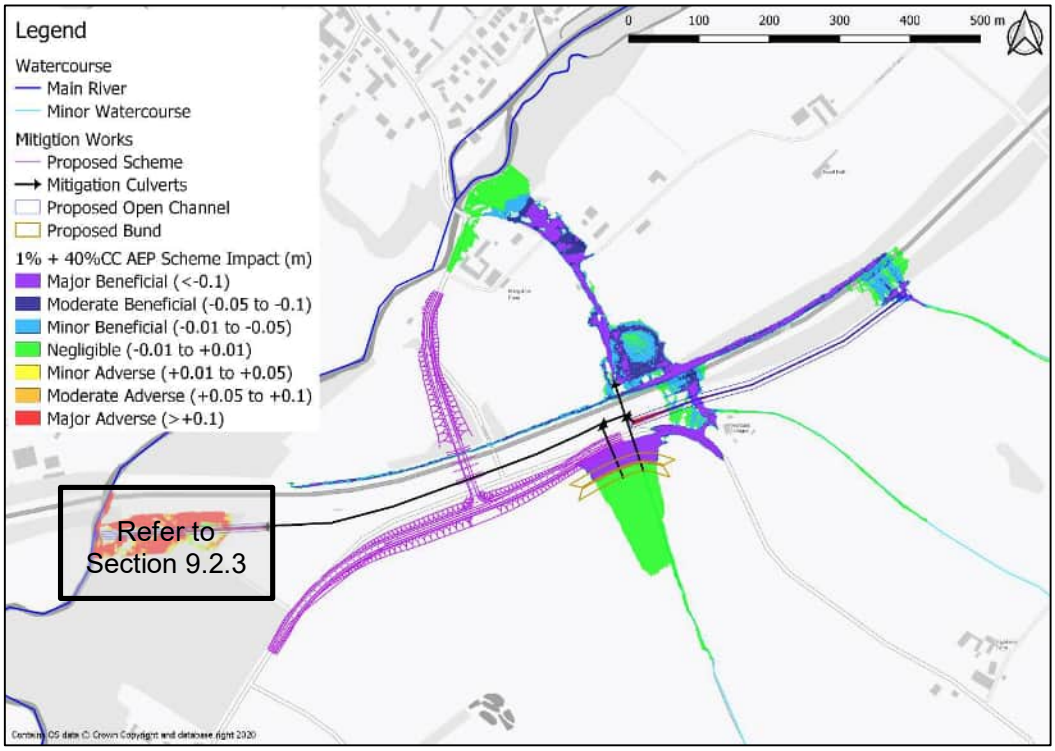


Plate 9.3 Water level difference (mitigation – baseline) for 1%(1 in 100) AEP +40%CC



10 Model assumptions and limitations

10.1 Introduction

10.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological, surveyed, and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

10.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed-scheme and are therefore, appropriate for the flood risk assessment.

10.2 Hydrology

10.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- As MWC21 is a small ungauged catchment there is inevitable uncertainty associated with flow estimation for small, ungauged catchments.
- ReFH2.3 is most appropriate for the target catchment as it uses BFIHOST19 and more recent (FEH13) rainfall data. FEH Statistical is not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 23 and 25 soils.

10.3 Hydraulic modelling

10.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for MWC21 are as follows:

- Culvert and model roughness values has been assigned across the model using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the TUFLOW Manual. The dimensions for structures have been based on detailed survey measurements.
- The LiDAR data is assumed to appropriately represent the floodplain.
- A 1m grid has been used. This is deemed a sufficient level of detail to represent floodplain topography and flooding mechanisms predicted by the model in the vicinity of the proposed-scheme.

- As this model study is on MWC21, modelling details at the western discharge point have not been represented, such as proposed A12 scheme embankment works and the River Blackwater's complex hydraulic behaviour. It is considered that the downstream boundary representation in this area is of sufficient detail to allow the assessment of the scheme scenarios for MWC21.
- No design drawings of the storage chamber were received, arrangement is based the survey files provided, which comprise photos and videos. In the with-scheme and mitigation scenarios, the location and dimensions of this chamber, as well as the length of pipes connected to it, are approximative and are subject to change at detailed design stage. It should be noted that the exact dimensions and location will not impact results as pipe capacity and upstream (south) inverts are the key hydraulic controllers of the A12 crossing.
- No calibration was carried out as the catchment is ungauged.

11 Conclusion

11.1.1 This report has detailed the modelling carried out to assess the baseline and with-scheme flood risk for Minor Watercourse 21 and its tributaries with respect to the proposed highway scheme.

11.1.2 The results of the baseline modelling have shown that there is significant floodplain storage on the western tributary upstream of A12, and that there is overtopping of the A12 in all modelled events at the middle and eastern tributaries. The introduction of the proposed-scheme results in increased flooding extent, as well as increased overtopping and inundation of the A12. Therefore, mitigation works were deemed necessary.

11.1.3 A bund was introduced to the model upstream of the proposed A12 crossing to prevent flows from the west tributary ponding against the A12 embankment and help increase storage upstream of the A12, along with an additional drainage network added to divert flows west towards the River Blackwater. A flow bypass system was added comprising a 1.5m diameter pipeline carrying a portion of high flows from all three tributaries out to the River Blackwater downstream of Ashmans Bridge, thus reducing flood risk around the MWC21 A12 crossing, and on the open channel downstream of the highway. The diversion pipeline represents the shift of a portion of storm flows into the River Blackwater from upstream to downstream of Ashmans Bridge, this is considered inconsequential for flood risk on the River Blackwater itself.

11.1.4 Private properties situated at the northern boundary of the model are shown to have had a reduction in flood risk due to the mitigation scenario.

11.1.5 The proposed mitigation works have reduced the flooding extents for all events, and the A12 highway remains free from flooding for all the mitigation simulations.

Acronyms

Abbreviation	Term
AEP	Annual Exceedance Probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

References

Chow, Ven Te (1959). Open-Channel Hydraulics. McGraw-Hill.

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Appendix A – Minor Watercourse 21 Hydrology

See document HE551497-JAC-EWE-SCHW-RP-LE-0088

Appendix B – Sensitivity Testing

Plate B.1 Water level difference (Manning's 'n' Roughness changed by -20%) for the 1% AEP event



Plate B.2 Water level difference (Manning's 'n' Roughness changed by +20%) for the 1% AEP event

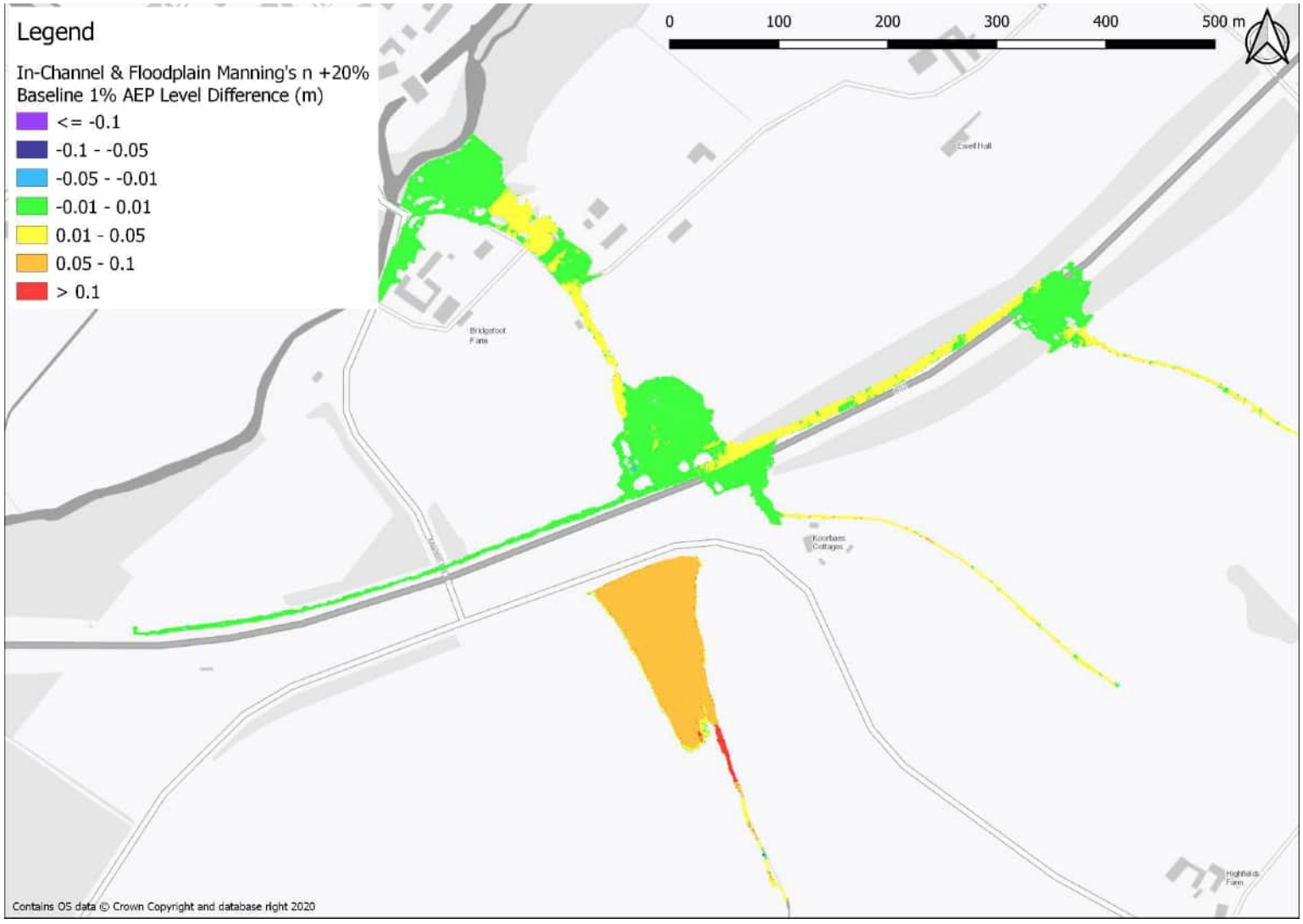
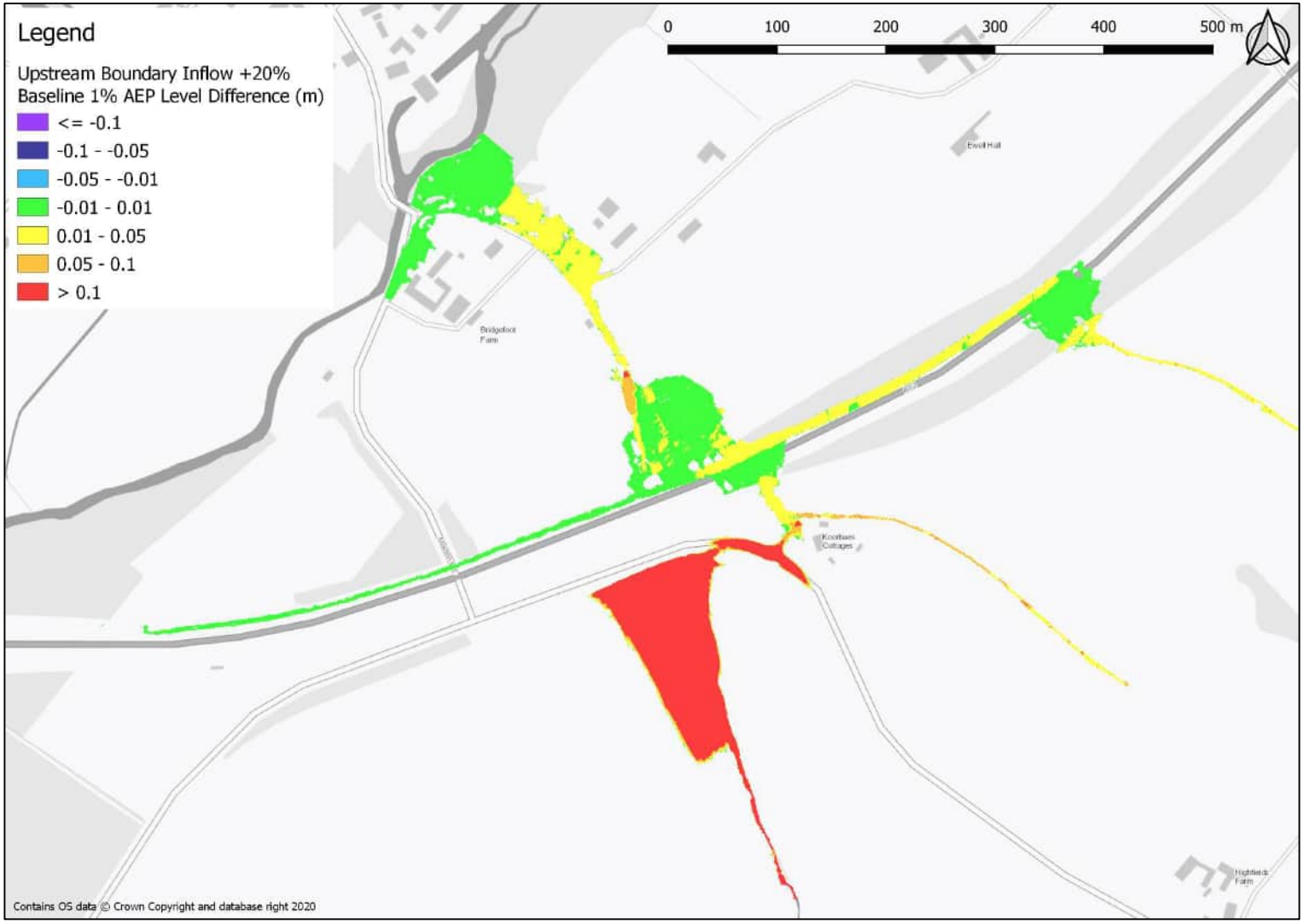


Plate B.3 Water level difference (Hydrological Inflows into model changed by -20%) for the 1% AEP event



Plate B.4 Water level difference (Hydrological Inflows into model changed by +20%) for the 1% AEP event



Appendix C – With-Scheme Flood Mapping

Plate C.1 5% AEP maximum flood extent for baseline and with-scheme scenarios

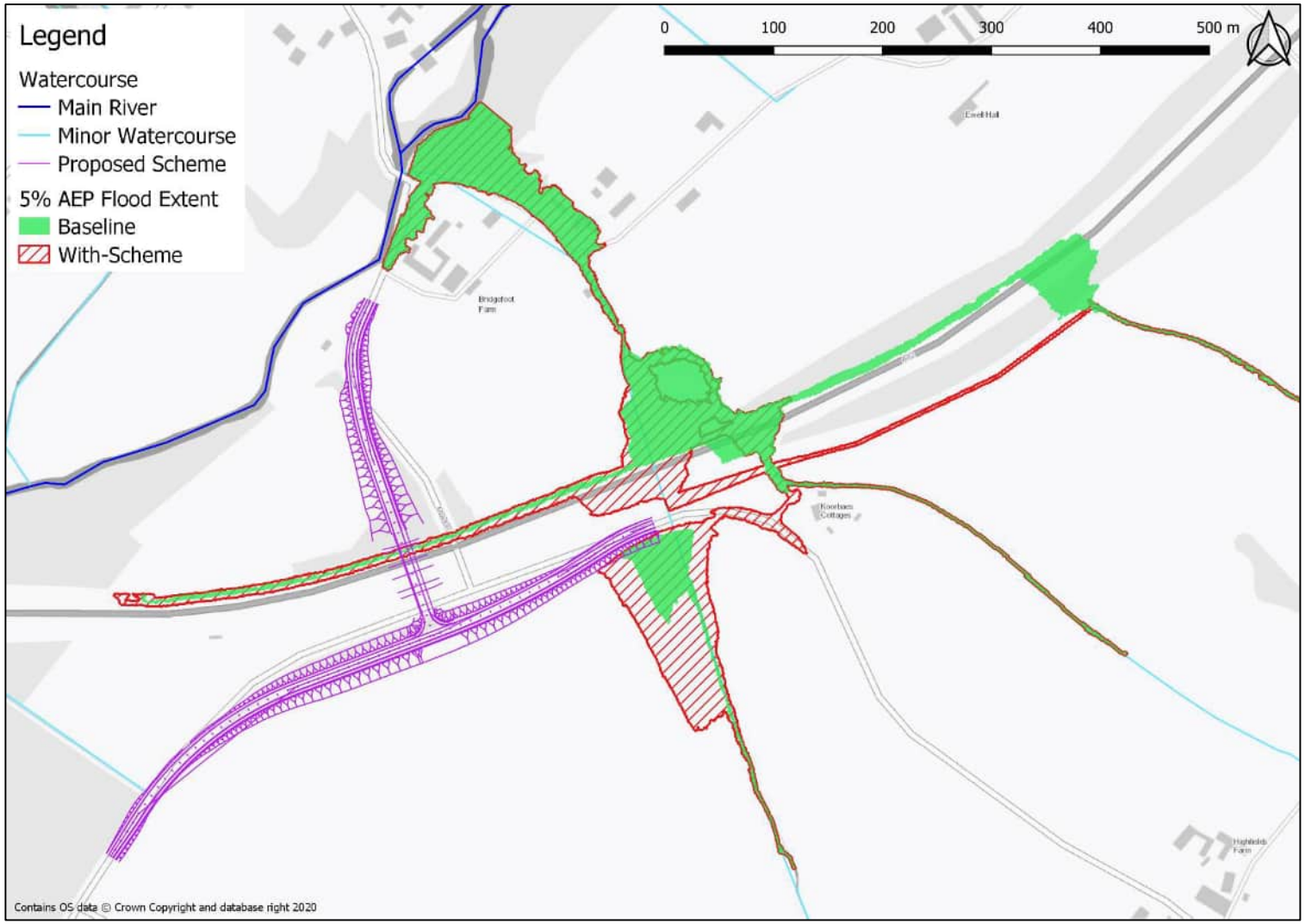


Plate C.2 1% AEP maximum flood extent for baseline and with-scheme scenarios



Plate C.3 1% AEP +40%CC maximum flood extent for baseline and with-scheme scenarios

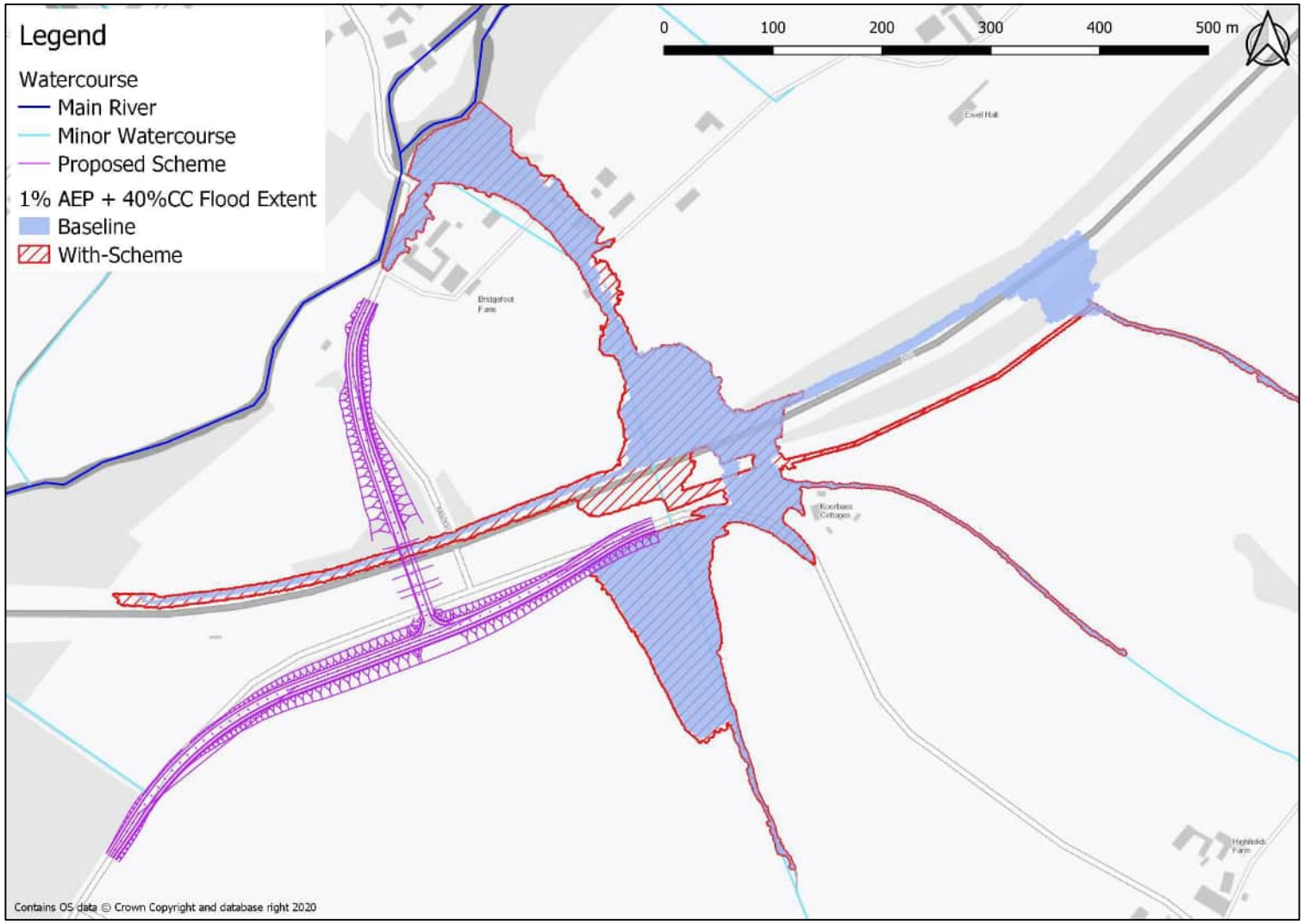


Plate C.4 Water level difference (with scheme - baseline) for the 5% AEP event



Plate C.5 Water level difference (with scheme - baseline) for the 1% AEP event



Plate C.6 Water level difference (with scheme - baseline) for the 1% AEP + 40%CC event



Appendix D – Mitigation Flood Mapping

Plate D.1 5% AEP maximum flood extent for baseline and mitigation scenarios



Plate D.2 1% AEP maximum flood extent for baseline and mitigation scenarios



Plate D.3 1% AEP +40%CC maximum flood extent for baseline and mitigation scenarios



Plate D.4 Water level difference (mitigation - baseline) for the 5% AEP event

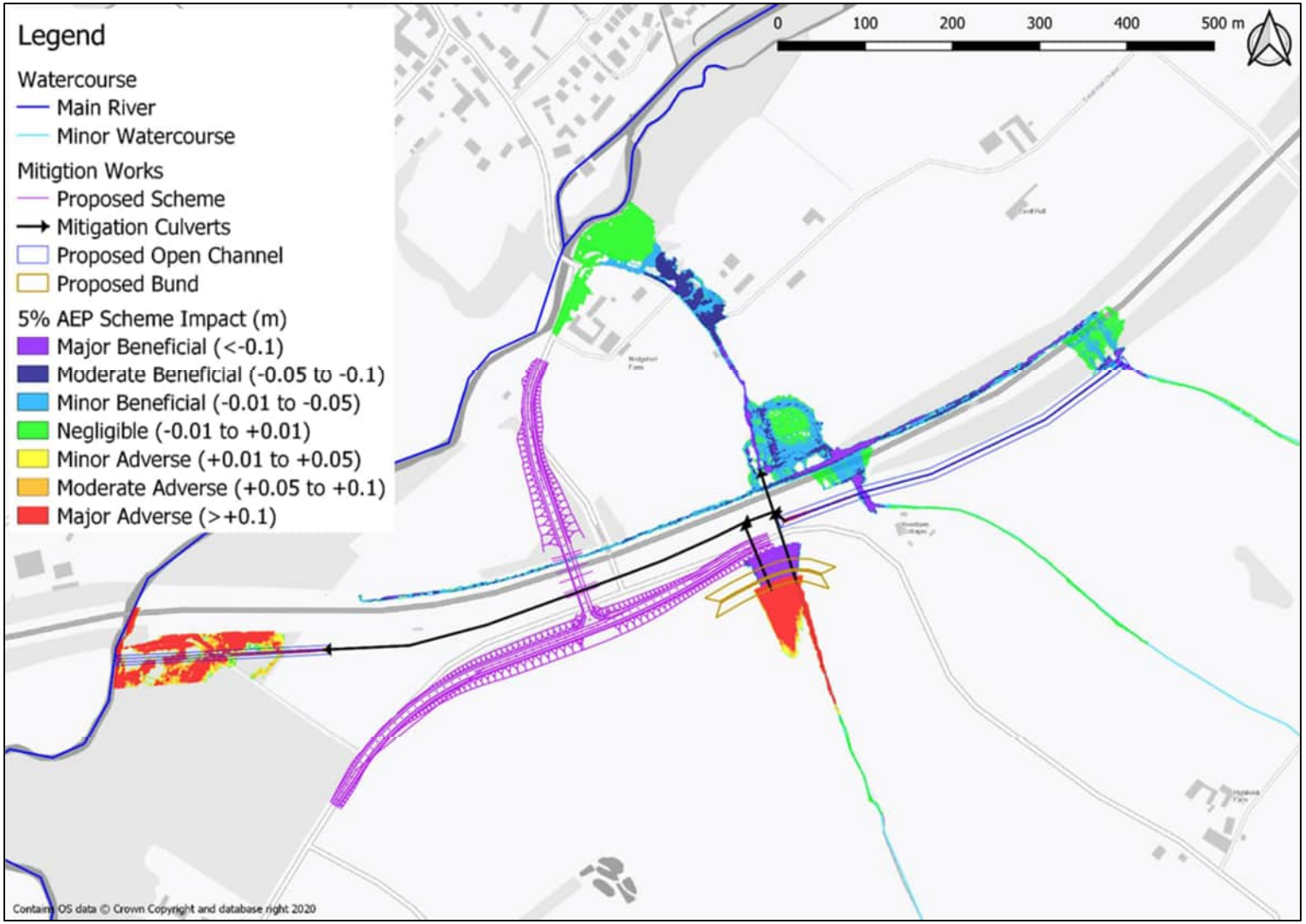


Plate D.5 Water level difference (mitigation - baseline) for the 1% AEP event



Plate D.6 Water level difference (mitigation - baseline) for the 1% AEP + 40%CC event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 21 HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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Table 3.1 Annual exceedance probability (AEP) and related return period reference table.6

1 Executive Summary

1.1.1 This document has been prepared to support the A12 Minor Watercourse 21 with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation for the Minor Watercourse 21 (MWC21) catchment which crosses the A12. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future.

3 Summary of assessment

3.1 Summary

3.1.1 A summary of the key information contained within the detailed hydrological assessment of MWC21 is provided in the table below.

Catchment location	Minor watercourse 21 (MWC21) is located in South East England, just outside Kelvedon and crosses the A12 at approximately NGR: TL 86300 17850. This minor watercourse discharges to the River Blackwater approximately 0.4km downstream of the A12 crossing.
Purpose of study and scope	The A12 is proposed to be upgraded and therefore peak flows and inflow hydrographs are required for the numerical hydraulic modelling of this watercourse. Flows have been derived for the following Annual Exceedance Probability (AEP) events: 5%, 1% and 1% plus climate change (plus cc), equivalent to the 20-year, 100-year and 100-year plus cc return period events respectively.
Key catchment features	The minor watercourse has a rural catchment. There is a solar farm in the upper eastern catchment. HOST Class 23 and 25 soils are present within the middle to upper catchment.
Flooding mechanisms	Peak flows / volumes.
Gauged / ungauged	Ungauged.
Final choice of method	ReFH2.3
Key limitations / uncertainties in results	Small ungauged watercourse. Middle to upper catchment contains HOST Class 23 and 25 soils and therefore BFIHOST values open to some uncertainty.

3.2 Note on flood frequencies

3.2.1 The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

3.2.2 Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval.

3.2.3 The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Table 3.1 Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

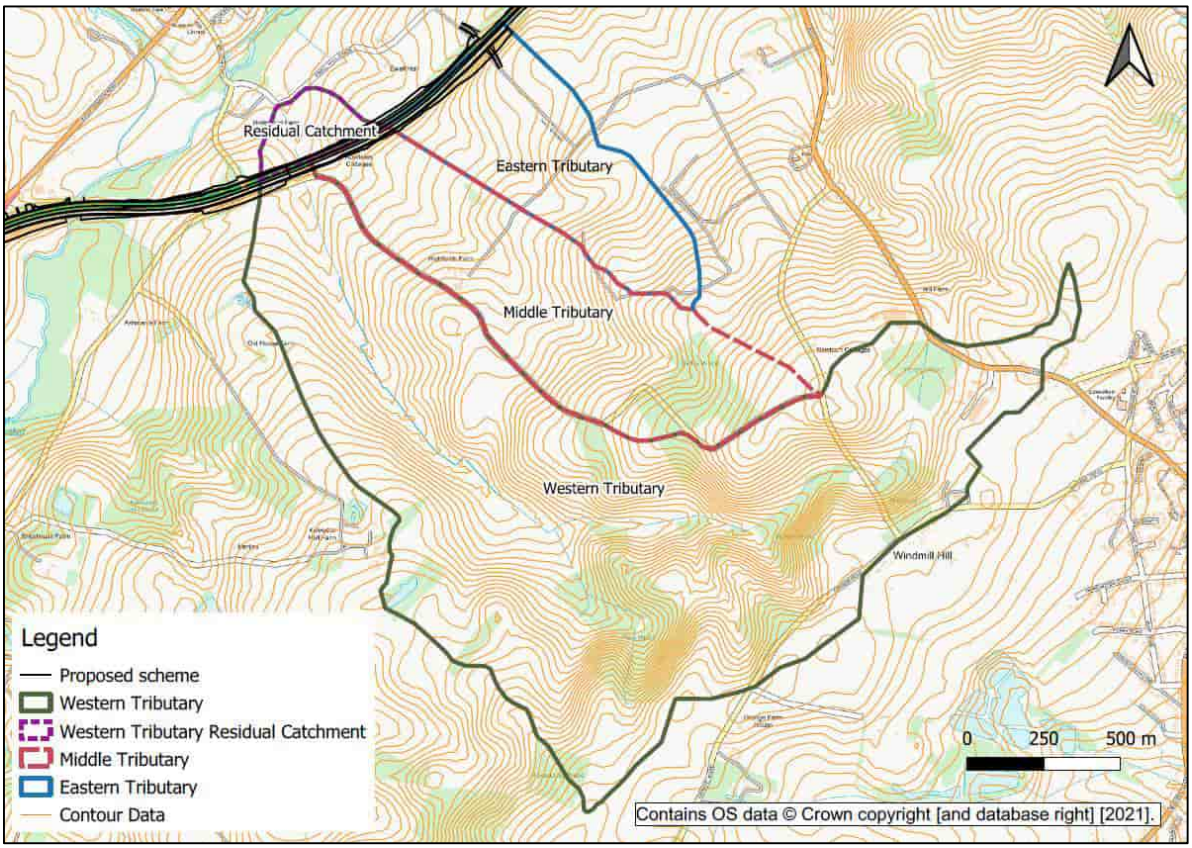
4 Method statement

4.1 Requirements for flood estimates

Overview	The proposed A12 upgrade route crosses Minor Watercourse 21 (MWC21) and therefore peak flows and hydrographs are required for the hydraulic modelling of this watercourse. Design peak flows and hydrographs are required for the following AEP events: 5%, 1% and 1% plus climate change (plus cc). This is equivalent to the 20-year, 100-year and 100-year plus cc return periods). A climate change uplift factor of plus 40% will be applied for the 1% AEP (100-year return period) event. This is based on current guidance[1] (peak rainfall allowance, upper end, 2080s scenario). MWC21 is a small and ungauged watercourse. For hydraulic modelling purposes three flow estimation points have been adopted, one for the western catchment to the downstream model extent, one for the middle catchment and the other for the eastern catchment both to the A12 crossing location.
Project scope	This is a simple hydrological study for one minor watercourse with a catchment area up to the downstream model extent of 3.27km^2. Therefore, ReFH model parameter calibration for this ungauged catchment is outwith the scope of this project.

[1] Environment Agency (2016). Flood risk assessments: climate change allowances (updated 2020).

4.2 The catchment



Description	As shown in the above figure, Minor Watercourse 21 has three tributaries upstream of the A12, namely, the western tributary (the largest), the middle tributary and eastern tributaries which combine just upstream of the A12. MWC21 discharges to the River Blackwater approximately 0.4km downstream of the A12 crossing. The catchment is classified as essentially rural with a number of dwellings present throughout the catchment. Land use is predominantly agricultural. There is also a solar farm within the upper eastern reaches of the catchment. The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 571s (Efford 1) and 572q (Ashley) in the lower to middle reaches of the catchment, with 712c (Windsor) being present in the middle to upper reaches of the catchment. The HOST class of the soil groups were assessed and soil type 712c was found to contain HOST class 23 and 25. There

	are noted issues within the Flood Estimation Guidelines[2] with BFIHOST values for soil which contains HOST 23 and 25 which has been corrected for BFIHOST19. The geology of the catchment was assessed using the Geology of Britain Viewer[3] and found to be composed of London Clay formations consisting of clay, silt and sand.

4.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released 24th September 2020. This contains provisional data up to water year 2019-20.

4.4 Gauging stations (flow or level)

Watercourse	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
River Blackwater	Blackwater @ Appleford Bridge		37010	247.3	Double throated trapezoidal flume and high flow rated spillway.	1962 to present
River Ter	Ter @ Crabbs Bridge		37003	77.8	Theoretically rated	1932 to present

4.5 Data available at each flow gauging station

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
Blackwater @ Appleford Bridge	1962 to present	N	Yes	Yes	N	Groundwater influenced regime. Intermittently affected by the Ely / Ouse transfer scheme.

[2] Flood Estimation Guidelines, Technical guidance 197_08 (Environment Agency, June 2020)

[3] The Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
Ter @ Crabbs Bridge	1963 to present	N	Yes	No	N	Recently removed from suitable for pooling category. This was noted as due to many AMAX events being out of bank and this is not accounted for in rating.

4.6 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings	N/A			Not required for this assessment.
Historical flood data	N/A			Not Available
Flow or river level data for events	N/A			Not available for this assessment.
Rainfall data for events	N/A			Not required for this assessment.
Potential evaporation data	N/A			Not required for this assessment.
Results from previous studies	N/A			Not available for this assessment.
Other data or information	N/A			Not available for this assessment.

4.7 Hydrological understanding of catchment

Conceptual model	Hydraulic modelling of MWC21 is required to assess potential impacts of upgrading the A12. The target location is the A12 crossing of MWC 21. Flooding is likely to be caused by peak flows/volumes at the A12 crossing location.
Unusual catchment features	The catchment is a small rural clay catchment located in South East England. There are noted issues with BFIHOST values for the soil (which contains HOST 23 and 25) within the middle to

	upper catchment.

4.8 Initial choice of approach

Is FEH appropriate?	Yes, FEH methods are appropriate but care needs to be taken due to BFIHOST issue.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	ReFH2.3 and FEH Statistical methods have been applied to derive the peak flow estimates and hydrographs for this catchment. The hydrographs have been derived using ReFH2.3 method. Flow estimates have been derived for the western catchment, the middle catchment, and the eastern catchment. The flow estimates for the western catchment were then area scaled so 97% of the total inflow was applied upstream of the A12 crossing at the start of the hydraulic model reach and the remaining 3% of the flow was applied as a lateral flow downstream of the A12 crossing. Point inflow were also applied within the hydraulic model at the start of both the eastern and middle catchment modelled reaches.
Software to be used (with version numbers)	FEH Web Service[4] / WINFAP 4[5] / ReFH2.3

4.9 Locations where flood estimates required

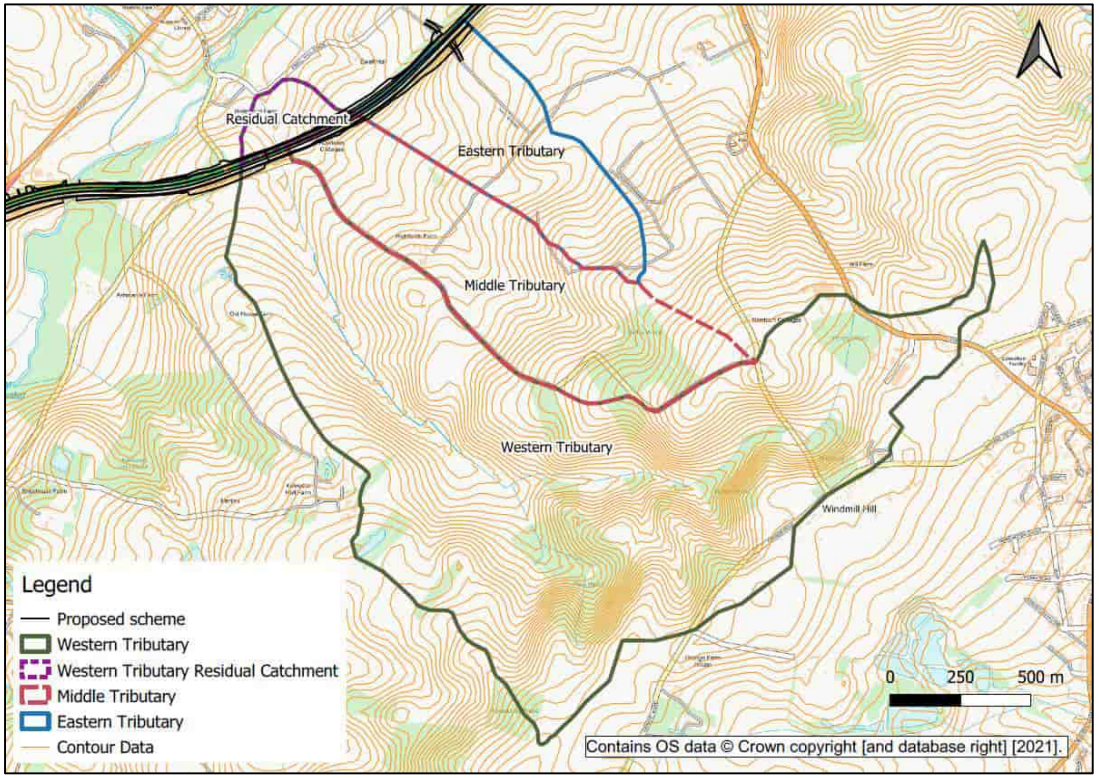
4.9.1 The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

[4] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

[5] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.10 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Western Tributary	L	MWC21	Western tributary to downstream model extent.	586249	217998	2.62	2.272
Middle Tributary	L	MWC21	Middle tributary to A12 crossing.	586418	217823	-	0.619
Eastern Tributary	L	MWC21	Eastern tributary to A12 crossing.	586702	217973	-	0.377

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required. Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced. The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.							

4.11 Important catchment descriptors at each subject site (incorporating any changes made)

4.11.1 As the FEH Web Service does not pick up the MWC21 catchments correctly and therefore key FEH catchment descriptors (area, DPSBAR, DPLBAR and URBEXT) were checked and calculated where necessary using standard FEH methodologies.

Site code	FARL	PROPWET	BFIHOST	BFIHOST 19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
Western Tributary	1	0.23	0.262	0.280	2.16	34.30	564	0.005	0.048
Middle Tributary	1	0.23	0.292	0.295	0.77	27.30	562	0.004	0.088
Eastern Tributary	1	0.23	0.292	0.295	0.59	24.67	562	0.000	0.088

4.12 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	The MWC21 catchment areas were checked using 1m contour data and LIDAR data.
Record how other catchment descriptors were checked and describe any changes.	Key catchment descriptors such as FARL and soils were checked using mapping data and key catchment descriptors were calculated using standard methods where appropriate as the FEH Web Service did not pick up the catchments correctly (e.g., area, DPSBAR, DPLBAR, URBEXT).
Source of URBEXT	URBEXT1990 and URBEXT2000 calculated using URBAN50K method.
Method for updating of URBEXT	URBEXT calculated using URBAN50K.

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	The FEH Statistical method was used as a comparison against ReFH2.3 peak flow estimates. The FEH Statistical method uses BFIHOST which has known issues in South East England on clay catchments with HOST Classes 23 and 25 within the catchment.

5.2 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjustment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, (A/B)[a]	If more than one donor			
						Weight	Weighted ave. adjustment		
Western Tributary	0.488	CDs	-	-	-	-	-	1.004	0.490
Middle Tributary	0.151	CDs	-	-	-	-	-	1.003	0.152
Eastern Tributary	0.092	CDs	-	-	-	-	-	1.000	0.092

Are the values of QMED spatially consistent?	N/A
Method used for urban adjustment for subject and donor sites	

Parameters used for WINFAP v4 urban adjustment if applicable		
Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010) in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016).

5.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Donor gauges were assessed for the purposes of QMED adjustment; however, no donor was selected. The River Blackwater @ Appleford Bridge gauge (37010) was assessed as it is the closest gauge, however, the River Blackwater @ Appleford Bridge gauges a much larger catchment with groundwater influence in the upper reaches of the catchment. This gauge was therefore assessed as not suitable as a donor for this small ungauged catchment. The River Ter @ Crabbs Bridge (37003) was also assessed as a donor for QMED adjustment, but this gauge was found to have varying FEH cds from the target site. Both the River Blackwater @ Appleford Bridge and River Ter @ Crabbs Bridge gauges would have resulted in a decrease in QMED and given neither were assessed as suitable donors QMED was not adjusted.

5.4 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)

5.5 Derivation of pooling group - middle and eastern tributaries

5.5.1 FEH Statistical Pooling was undertaken on the catchment 586150 218150 for the middle and eastern tributaries.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
Final Pooling Group	Eastern and Middle Tributaries	No	Removed: 27073 / 26016 (due to high BFIHOST), 47022 / 106002 (due to low FARL) and 49005 (due to relatively high BFIHOST / high discordance). Added 206006 / 36010	L-CV: 0.226 L-Skew: 0.268
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

5.6 Derivation of flood growth curves – middle and eastern tributaries

Site code	Method	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
Eastern and Middle Tributaries	P	Final Pooling Group	GL: distribution gives an acceptable fit.	Urban Adjustment	Location: 1.000 Scale: 0.221 Shape: -0.268	3.003
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis Urban adjustments are all carried out using the method of Kjeldsen (2010). Growth curves were derived using the procedures from Science Report SC050050 (2008).						

5.6.1 For further details of the middle / eastern tributary pooling group refer to Annex A.1 to A.3.

5.7 Derivation of pooling group - western tributary

5.7.1 FEH Statistical Pooling was also undertaken on catchment 586300 217850 for the western tributary.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
Final Pooling Group	Western Tributary	No	Removed: 26016 / 27073 / 44008 (due to high BFIHOST), 47022 (due to low FARL) and 49005 (due to relatively high BFIHOST / high discordance). Added 206006 / 36010	L-CV: 0.227 L-Skew: 0.269
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

5.8 Derivation of flood growth curves – western tributary

Site code	Method	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
Western Tributary	P	Final Pooling Group	GL: distribution gives an acceptable fit.	Urban Adjustment	Location: 1.000 Scale: 0.223 Shape: -0.269	3.02
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis Urban adjustments are all carried out using the method of Kjeldsen (2010). Growth curves were derived using the procedures from Science Report SC050050 (2008).						

5.8.1 For further details of the western tributary pooling group refer to Annex A.4 to A.6.

5.9 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	25	50	100	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events									
	50	20	10	5	4	2	1	0.5	0.2	0.1
Western Tributary	0.490	0.673	0.817	0.980	1.04	1.24	1.48	1.77	2.24	2.68
Middle Tributary	0.152	0.208	0.253	0.303	0.321	0.383	0.456	0.545	0.689	0.825
Eastern Tributary	0.092	0.127	0.153	0.184	0.195	0.232	0.277	0.331	0.419	0.501

6 Revitalised flood hydrograph 2 (ReFH2) method

6.1 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 has been applied to derive both peak flow estimates and hydrographs for the target watercourse at key locations for input into a hydraulic model.

6.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp}	BL (hours)	BR
Western Tributary	ReFH2.3	5.17	-	250.47	-	33.16	0.6
Middle Tributary	ReFH2.3	3.08	-	260.42	-	27.33	0.72
Eastern Tributary	ReFH2.3	2.73	-	260.42	-	25.79	0.72
Brief description of any flood event analysis carried out							
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

6.3 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
Western Tributary	Rural	Winter	9.0
Middle Tributary	Rural	Winter	4.5
Eastern Tributary	Rural	Winter	4.5

6.4 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm

6.5 Flood estimates from the ReFH2 method

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
Western Tributary (CSD = 9hrs)	0.800	1.08	1.27	1.46	1.52	1.74	2.02	2.91	2.46	3.34	4.08
Middle Tributary (CSD = 4.5hrs)	0.275	0.383	0.456	0.529	0.554	0.635	0.734	1.06	0.876	1.17	1.45
Eastern Tributary (CSD = 4.5hrs)	0.183	0.255	0.304	0.354	0.370	0.425	0.491	0.709	0.587	0.788	0.975

7 Discussion and summary of results

7.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak			
	Return period 20-years / 5% AEP		Return period 100-years / 1% AEP	
	FEH Statistical	ReFH2.3	FEH Statistical	ReFH2.3
Western Tributary	0.980	1.46	1.48	2.02
Middle Tributary	0.303	0.529	0.456	0.734
Eastern Tributary	0.184	0.354	0.277	0.491

7.2 Final choice of method

Choice of method and reasons	The final choice of method is ReFH2.3 as it incorporates updates to BFIHOST19 and uses latest rainfall data (FEH13).
How will the flows be applied to a hydraulic model?	The ReFH2.3 design hydrographs will be applied in the hydraulic model. The hydrographs for the western catchment will be area scaled within the model and 97% of the total inflow will be applied upstream of the A12 at the start of the hydraulic model reach. The remaining 3% of the western tributary flow will be applied as a lateral flow downstream of the A12 crossing. Point inflows will be applied within the hydraulic model at the start of both the eastern and middle tributary modelled reaches.

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are as follows: • ReFH2.3 is most appropriate for the target catchment. • FEH Statistical is not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 23 / 25. • The solar farm has little impact on design peak flow estimates. • Water is assumed to not back up from the River Blackwater during simulated flood events.
Discuss any particular limitations	None.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	The ReFH2.3 method has been used to derive design peak flows and hydrographs. This method is appropriate but there is inherent uncertainty regarding using default model parameters. The standard approach in this situation is to use default model parameters. If the catchment was gauged the

	ReFH parameters could be adjusted to reduce the uncertainty.
Comment on the suitability of the results for future studies	The design peak flows have been estimated for the purposes of the A12 road improvements project only with the target location being the A12 crossing. Future studies should assess the appropriateness of these peak flow estimates / hydrographs for the purposes of their investigation.
Give any other comments on the study	

7.4 Checks

Are the results consistent, for example at confluences?	N/A
What do the results imply regarding the return periods / frequency of floods during the period of record?	N/A ungauged catchment.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The 1% AEP (100-year return period) growth factor for the western tributary to the downstream model extent is 3.020. The 1% AEP (100-year return period) growth factor for the middle/eastern tributaries at the A12 crossing is 3.003. The two pooling group analysis results in almost similar growth curve with the 1% AEP growth factor close to 3.0. This is considered within the expected range.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the 0.1% AEP / 1% AEP for the western tributary catchment is 1.813. The ratio of the 0.1% AEP / 1% AEP for the middle/eastern tributaries is 1.808.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No other studies were available for this minor watercourse.
Are the results compatible with the longer-term flood history?	N/A

Describe any other checks on the results	The peak flow estimates will be checked within the model at the A12 crossing (the target location).

7.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
Western Tributary	0.800	1.08	1.27	1.46	1.52	1.74	2.02	2.91	2.46	3.34	4.08
Middle Tributary	0.275	0.383	0.456	0.529	0.554	0.635	0.734	1.06	0.876	1.17	1.45
Eastern Tributary	0.183	0.255	0.304	0.354	0.370	0.425	0.491	0.709	0.587	0.788	0.975

7.6 Uncertainty bounds

7.6.1 The Flood Estimation Guidelines 197_08 (2020) notes that it is more difficult to quantify uncertainty in design flows estimated from the ReFH rainfall-runoff model than for the FEH Statistical method which it reports approximate uncertainty bounds for. It is therefore highlighted that the final flow estimates are open to uncertainty, but uncertainty bounds have not been provided for this assessment.

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hydrographs are required for the hydraulic modelling of MWC21. See: MWC21 ReFH2.3 Hydrographs

Acronyms

Abbreviation	Term
AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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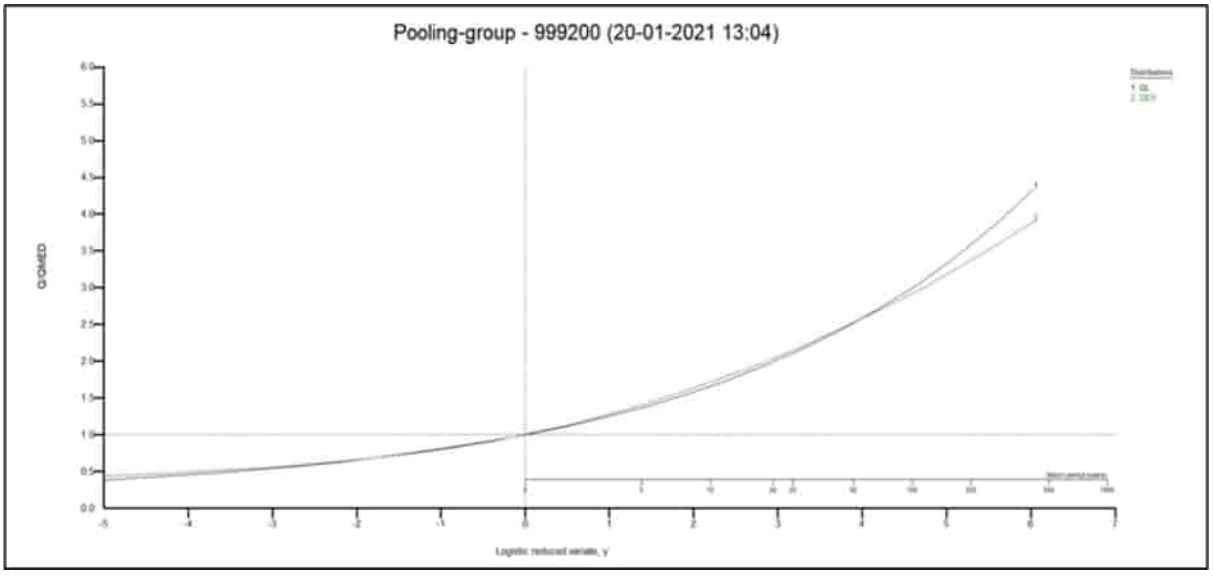
Annex A

A.1 Pooling Group Details – Middle / Eastern Tributaries

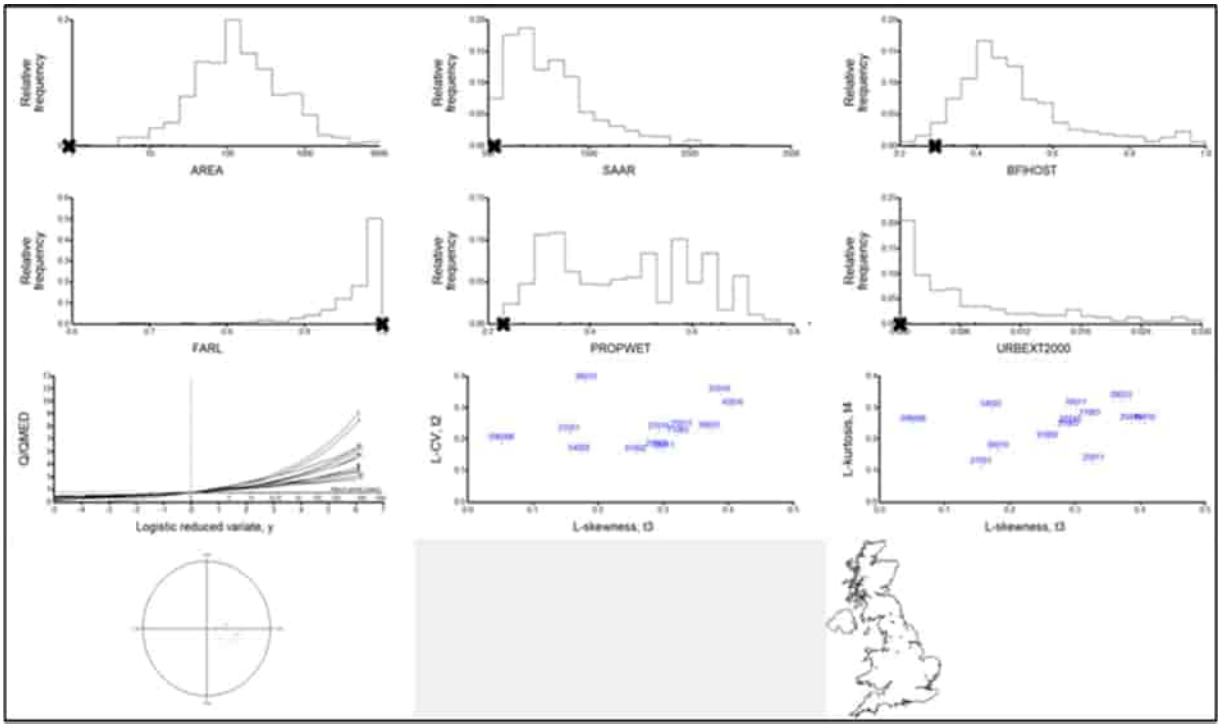
A.1.1 Final Pooling Group Details

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.523	42	1.84	0.163	0.301	0.579
27051 (Crimple @ Burn Bridge)	3.278	47	4.524	0.218	0.156	1.394
45816 (Haddeo @ Upton)	3.285	26	3.456	0.3	0.406	0.802
28033 (Dove @ Hollinsclough)	3.569	44	4.177	0.228	0.371	0.844
25019 (Leven @ Easby)	4.067	41	5.09	0.342	0.386	1.182
91802 (Allt Leachdach @ Intake)	4.107	34	6.35	0.153	0.257	0.718
25011 (Langdon Beck @ Langdon)	4.207	33	15.647	0.232	0.328	1.6
71003 (Croasdale Beck @ Croasdale Flume)	4.225	37	10.9	0.212	0.323	0.174
25003 (Trout Beck @ Moor House)	4.266	46	15.142	0.168	0.29	0.361
54022 (Severn @ Plynlimon Flume)	4.338	38	14.988	0.156	0.171	0.982
27010 (Hodge Beck @ Bransdale Weir)	4.455	41	9.42	0.224	0.293	0.028
206006 (Annalong @ Recorder)	4.468	48	15.33	0.189	0.052	1.925
36010 (Bumpstead Brook @ Broad Green)	4.793	52	7.395	0.382	0.181	2.412
Total		529				
Weighted means		529		0.226	0.268	

A.2 Final Growth Curve – Middle / Eastern Tributaries



A.3 Diagnostic Plots - Middle / Eastern Tributaries

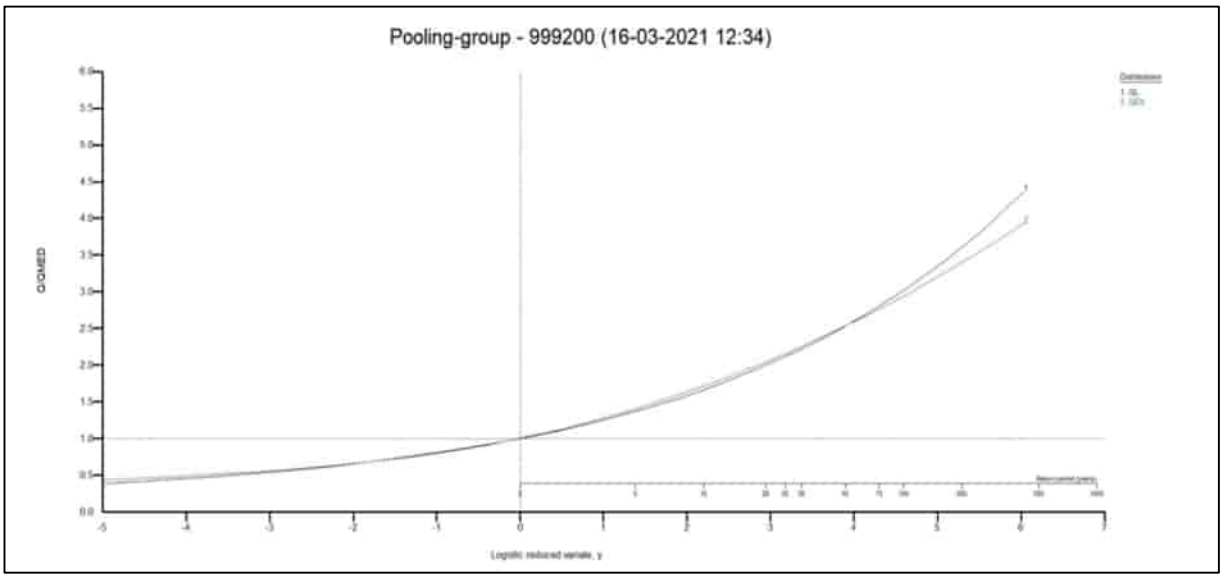


A.4 Pooling Group Details – Western Tributary

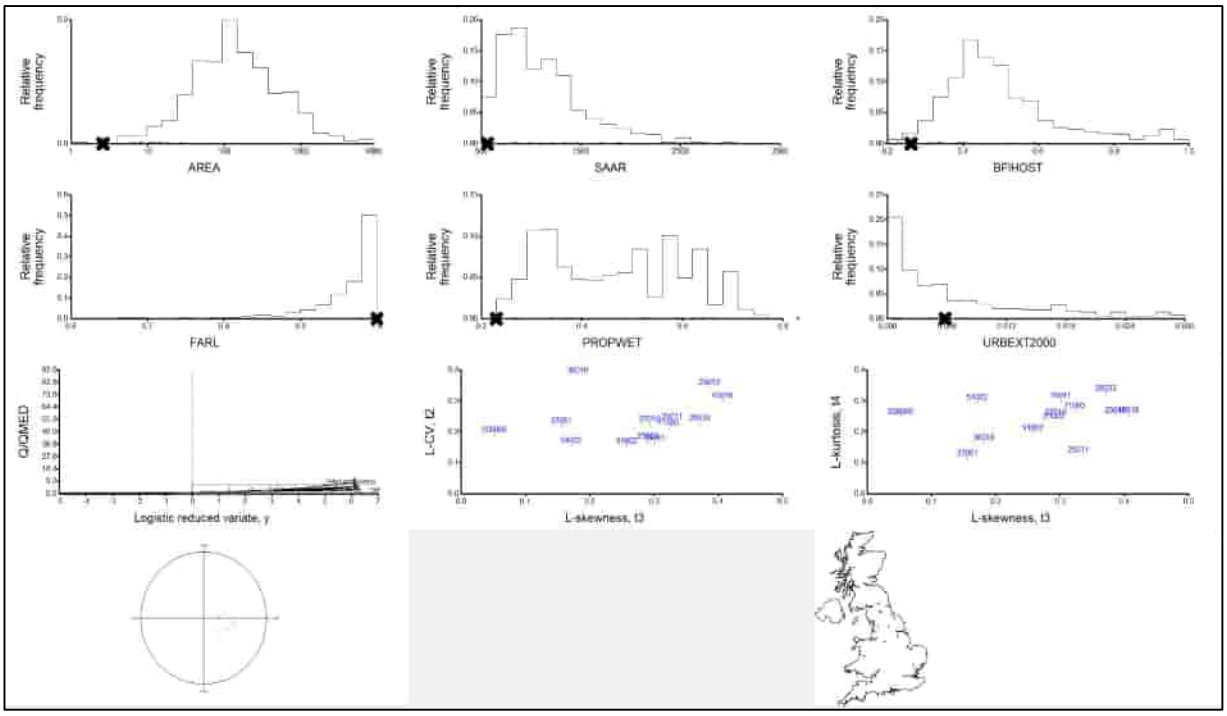
A.1.1 Final Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.462	42	1.84	0.163	0.301	0.579
27051 (Crimple @ Burn Bridge)	1.817	47	4.524	0.218	0.156	1.394
45816 (Haddeo @ Upton)	2.018	26	3.456	0.3	0.406	0.802
28033 (Dove @ Hollinsclough)	2.313	44	4.177	0.228	0.371	0.844
25019 (Leven @ Easby)	2.574	41	5.09	0.342	0.386	1.182
25011 (Langdon Beck @ Langdon)	2.894	33	15.647	0.232	0.328	1.6
27010 (Hodge Beck @ Bransdale Weir)	2.985	41	9.42	0.224	0.293	0.028
71003 (Croasdale Beck @ Croasdale Flume)	3.049	37	10.9	0.212	0.323	0.174
25003 (Trout Beck @ Moor House)	3.104	46	15.142	0.168	0.29	0.361
91802 (Allt Leachdach @ Intake)	3.196	34	6.35	0.153	0.257	0.718
206006 (Annalong @ Recorder)	3.2	48	15.33	0.189	0.052	1.925
36010 (Bumpstead Brook @ Broad Green)	3.29	52	7.395	0.382	0.181	2.412
54022 (Severn @ Plynlimon Flume)	3.321	38	14.988	0.156	0.171	0.982
Total		529				
Weighted means				0.228	0.268	

A.5 Final Growth Curve – Western Tributary



A.6 Diagnostic Plots – Western Tributary



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 26 MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

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0 Executive Summary

0.1.1 This report describes the hydraulic modelling and the results obtained for the existing and proposed A12 highway crossing at Minor Watercourse 26 (MWC26), NGR grid reference TL912233. The modelling and hydrology analyses have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 2-dimensional (2D) type using TUFLOW software (BMT 2022) and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 40% climate change (CC). No flow reconciliation was required to be carried out for the design hydrology. No data was available for calibration, but the model was verified using the Environment Agency (EA) flood maps. The model was used to simulate baseline and with-scheme scenarios for the three design events mentioned. The with-scheme scenario involved a new mainline to be built to the east of the existing A12 carriageway, which connects to it via a new proposed roundabout. The implementation of a new set of culverts to convey flows of the MWC26 channels across the proposed mainline and junction was required, and the discontinuance of two existing culverts. Sensitivity analysis of roughness and inflows was carried out to further validate the results.

0.1.3 The baseline (existing) modelling scenario shows the A12 carriageway remains dry for all flood events simulated. The proposed-scheme is flooded in a 1% (1 in 100) AEP +40%CC event. The proposed-scheme also has a minor adverse impact (+10-50mm) on downstream water levels. Therefore, mitigation works were required. The mitigation measures put in place prevent flooding of the proposed-scheme and have no adverse impacts to the downstream channel.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tey). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the proposed-scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA. This took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios, both upstream and downstream of the proposed-scheme, and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) are:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Minor Watercourses 7, 21, 21A, 23, 26 (this report)

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out to assess the baseline, with-scheme and mitigation scenarios for an unnamed tributary of the Roman River. The watercourse is designated as Minor Watercourse 26 based on the A12 drainage design numbering system for the main culvert crossings of the highway. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 The A12 crossing of the Minor Watercourse 26 (MWC26) will be upgraded with the addition of a new offline road alignment situated approximately 120m to the southeast of existing carriageway, including a slip road as

well as a new junction. The proposed-scheme will be embanked across the Minor Watercourse 26 valley and will require three culvert structures for the watercourse, plus minor re-alignment of the watercourse local to the new junction. Full details of the with-scheme arrangements are given in Section 7.

1.1.6 This report should be read in conjunction with the FRA [R10060\APP\6.6] (National Highways, 2022).

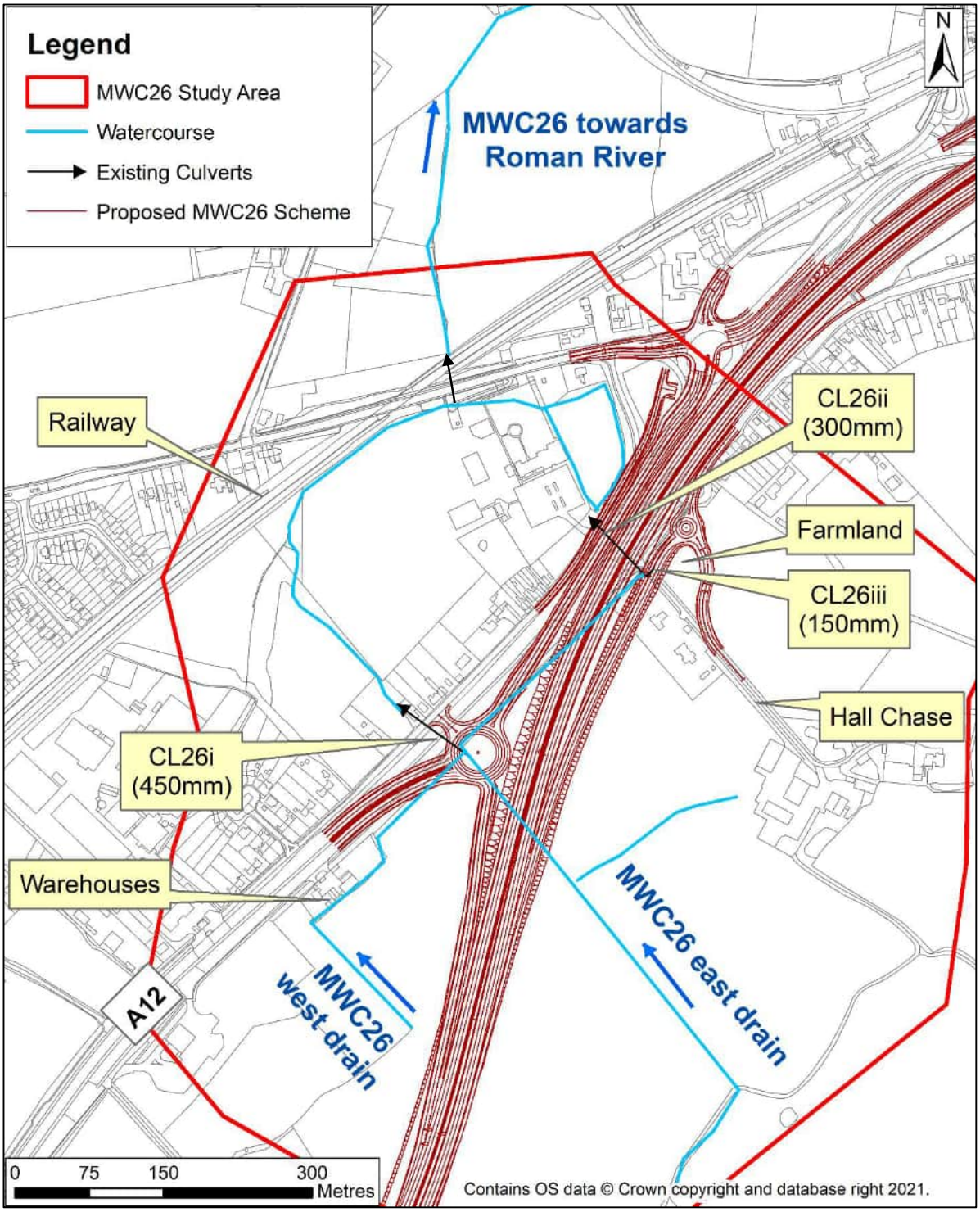
1.2 Study area

1.2.1 MWC26 is situated immediately southwest of Marks Tey, Essex. The watercourse is composed of two upstream reaches, referred to as east drain and west drain, originating approximately 600m upstream (south) of the A12 and flowing in a north-westerly direction (see Plate 1.1). The west drain flows past some warehouse buildings on its left bank before converging with the east tributary immediately upstream of the A12, where a 450mm culvert (CL26i) conveys part of the combined flows northerly under the highway and where a 250m long toe drain diverts the rest of the flows north-easterly towards Hall Chase alongside the highway. At Hall Chase, diverted flows from the toe drain split between a 300mm culvert (CL26ii) that crosses the A12 northerly and a 150mm culvert (CL26iii) that passes underneath Hall Chase to connect the toe drain to some low farmland just east of it.

1.2.2 Downstream of the A12, both flows carried across the A12 run northerly until meeting at the railway culvert before flowing northeast, where MWC26 discharges into the Roman River approximately 1.4km downstream of the A12 crossing.

1.2.3 The model extent covers an area of 0.66km^2 and includes a total reach length of approximately 1800m.

Plate 1.1 MWC26 Study Area



2 Input Data

2.1.1 The data used to construct the hydraulic model for MWC26 is summarised in Table 2.1.

Table 2.1 Data used to build the hydraulic model.

Data	Description	Source
LiDAR	1m resolution composite Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) 2020 data. Used to inform the hydraulic model with ground level information.	Department for Environment, Food & Rural Affairs (DEFRA), 2020
Ordnance Survey (OS) Mastermap	Land use data used to specify roughness values across the 2D floodplain.	National Highways
As-built survey	Survey of key hydraulic structures in model.	MKSurveys, December 2019
Outline Design Drawings	Drawings of the proposed-scheme design.	National Highways, 2021
Flood Zone Mapping	EA Risk of Flooding from Surface Water (RoFSW) flood maps. Used for model verification.	Environment Agency
Topographic Survey	Survey data of channel bed levels. Used to carve in the MWC26 channels.	Costain, 2017-2021
Site Visit Photos	Photos taken on a site visit of existing culverts. Used for modelled culvert dimensions.	MKSurveys & Flowline, 2021

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of Minor Watercourse 26 (MWC26) for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event is also required including an allowance for climate change (plus CC). For full details of the flow derivation methods and background hydrology refer to Appendix A Minor Watercourse 26 Hydrology Annex (HE551497-JAC-EWE-SCHW-RP-LE-0090).

3.2 Hydrological Considerations

3.2.1 MWC26 is located in South East England to the west of Colchester. MWC26 crosses the A12 at approximate co-ordinate 591200 223300 and discharges to the Roman River approximately 1.4km downstream of the A12 road crossing. MWC26 has a small catchment area of approximately 1.5km^2 to the downstream model extent (just upstream of the Roman River confluence). The catchment is generally classified as essentially rural with land use predominantly agricultural but there are some more urbanised parts of the catchment associated with the Marks Tey urban area. The middle section of the catchment (just downstream of the A12 crossing) is classified as moderately urbanised.

3.2.2 The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 411d (Hanslope), 582d (Hornbeam 3) and 573b (Wix) in the lower reaches of the catchment. The HOST class of the soil groups were assessed and soil type 573b (Wix) was found to contain HOST class 25. There are noted issues within the Flood Estimation Guidelines with BFIHOST values for soil which contains HOST 23 and 25 which has been corrected for BFIHOST19. The geology of the catchment was assessed using the Geology of Britain Viewer and found to be composed of predominantly London Clay formations consisting of clay, silt and sand.

3.3 Peak Flow Derivation

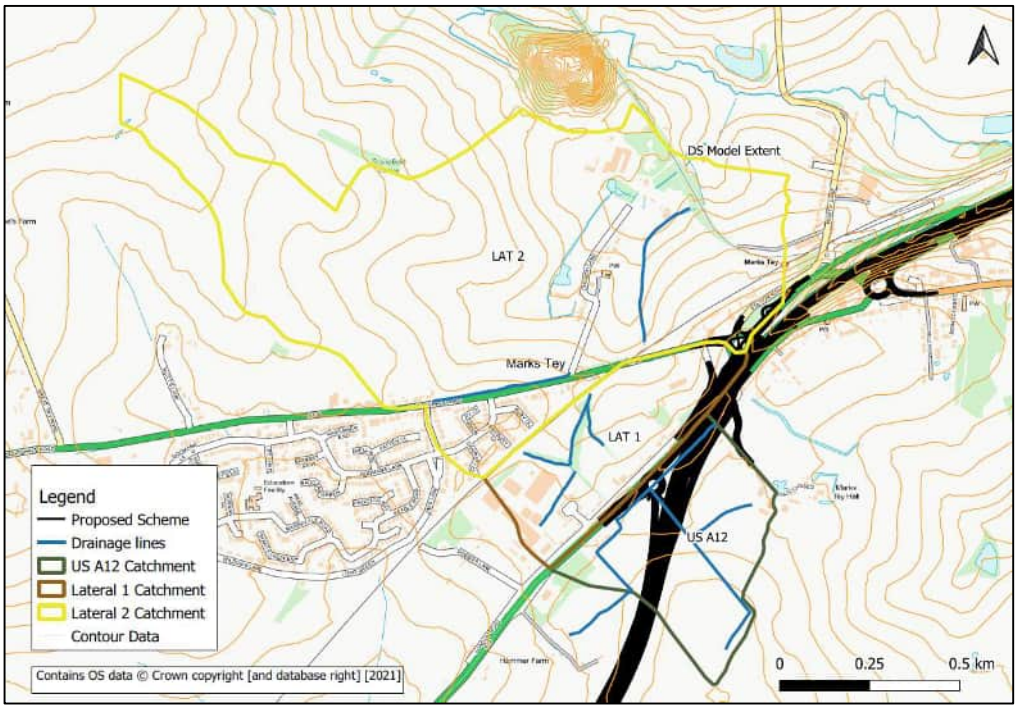
3.3.1 MWC26 is a small ungauged catchment and therefore flow estimation is open to great uncertainty. Both ReFH2.3 and FEH Statistical methods were therefore applied to estimate design peak flows for this watercourse. ReFH2.3 resulted in higher design peak flow estimates for MWC26 than the FEH Statistical method. Given ReFH2.3 uses BFIHOST19 which considers the known issues where the soils contain HOST Class 25 and also it uses FEH13 rainfall data, the ReFH2.3 method was selected for final peak flows estimates.

3.4 Hydraulic Model Inflows

3.4.1 For hydraulic modelling purposes, the MWC26 catchment was split into three sub-catchments. Flow estimates were derived for the upper catchment to the A12 crossing (US A12) which is the target location. Lateral inflow hydrographs were also derived for the residual catchment areas downstream of the road Lateral 1 (Lat 1) between the A12 road crossing and the railway line and Lateral 2 (Lat 2) between the railway line and the downstream model extent. The inflow for the residual Lateral 1 was applied as a lateral inflow within the hydraulic model. When the hydrology was produced, the downstream model extent was assumed to be at the location indicated in Plate 3.1. During the model build process, it was decided to reduce the extent of the model, to that shown in Plate 4.1. This meant that the Lateral 2 hydrology was not applied to the model. It is included in this section for completeness.

3.4.2 A schematisation of the MWC26 catchment is presented below.

Plate 3.1 Schematisation of the MWC26 catchment



3.4.3 The design peak flow estimates for the upper catchment to the A12 crossing (US A12) and for the residual catchments (Lateral 1 and Lateral 2) for the individual catchment critical storm duration are presented in Table 3.1.

Table 3.1 MWC26 peak flow estimates (m^3/s) at the target location US A12 and for the residual catchments (Lateral 1 and Lateral 2) for the individual catchment critical storm duration

Inflow	Peak Flow (m^3/s)										
	AEP 50% (1 in 2)	AEP 20% (1 in 5)	AEP 10% (1 in 10)	AEP 5% (1 in 20)	AEP 3.33% (1 in 30)	AEP 2% (1 in 75)	AEP 1.33% (1 in 75)	AEP 1% (1 in 100)	AEP 1% +CC 40% uplift (1 in 100)	AEP 0.5% (1 in 200)	AEP 0.1% (1 in 1000)
US A12	0.06	0.09	0.10	0.12	0.13	0.14	0.15	0.16	0.24	0.20	0.33
Lateral 1	0.07	0.09	0.11	0.13	0.14	0.15	0.17	0.18	0.25	0.21	0.35
Lateral 2	0.23	0.30	0.35	0.40	0.44	0.48	0.53	0.56	0.81	0.68	1.13

3.5 Model Runs and Critical Storm Duration

3.5.1 Introduction

To assess the critical storm duration for MWC26 two model runs were required. Run 1 was undertaken using the critical storm duration (CSD), areal reduction factor (ARF) and seasonal correction factor (SCF) for the catchment up to the initial downstream model extent and Run 2 was undertaken using the CSD, ARF and SCF for the catchment up to the target location (the A12 crossing). As described in Section 3.4.1, the adopted model extent was truncated to that shown in Plate 4.1, and so it is the Run 2 flows that are relevant to the final model. The flow values for both runs are provided here for completeness. Further details on the hydrology of the model runs and associated peak flow estimates are provided below.

3.5.2 Run 1

For Run 1 all inflows were derived using the CSD (9.0hrs), ARF (0.98) and SCF (0.655), relevant to the catchment to the downstream model extent. This run resulted in the following peak flow estimates for the modelled events.

Table 3.2 Run 1 Peak Flow Estimates (Storm Duration = 9hrs)

Inflows	Peak flows (m^3/s)		
	5% (1 in 20) AEP	1% (1 in 100) AEP	1% (1 in 100) AEP +40% CC uplift
US A12	0.12	0.16	0.24
Lateral 1	0.13	0.18	0.25
Lateral 2	0.40	0.56	0.81
DS Model Extent	0.53	0.74	1.07

3.5.3 Run 2

For Run 2 all inflows were derived using the CSD (5.5hrs), ARF (0.988) and SCF (0.644) to the target location: the A12 road crossing (US A12). This run resulted in the following peak flow estimates for the modelled events.

Table 3.3 Run 2 Peak Flow Estimates

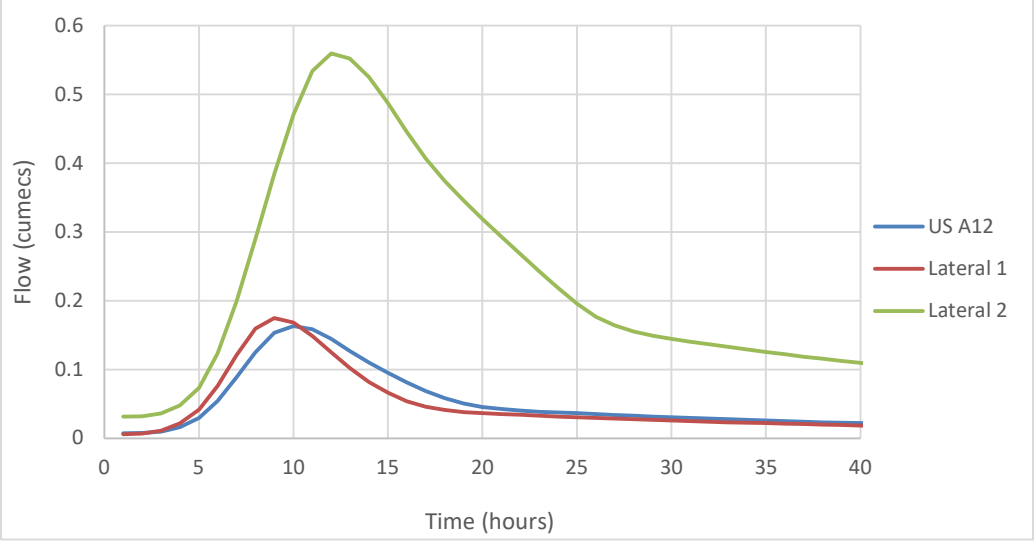
Inflows	Peak flows (m^3/s)		
	5% (1 in 20) AEP	1% (1 in 100) AEP	1% (1 in 100) AEP +40% CC uplift
US A12	0.12	0.16	0.24
Lateral 1	0.13	0.18	0.26
Lateral 2	0.34	0.54	0.77

3.6 Inflow Hydrograph Shape

3.6.1 As the MWC26 catchment is ungauged it was assessed that it was appropriate to derive hydrographs for input into the hydraulic model using ReFH2.3. For Run 1 the ReFH2.3 hydrograph was derived at the A12 crossing (the target location) and for the residual catchments downstream of the A12 (Lateral 1 and Lateral 2) based on a 9.0hr event storm duration, which is equivalent to the storm duration for the catchment up to the downstream model extent. For Run 2 the ReFH2.3 hydrograph was derived at the A12 crossing (the target location) and for the residual catchments downstream of the A12 (Lat 1 and Lat 2) based on a 5.5hr event storm duration, which is equivalent to the storm duration for the catchment up to the target location (US A12).

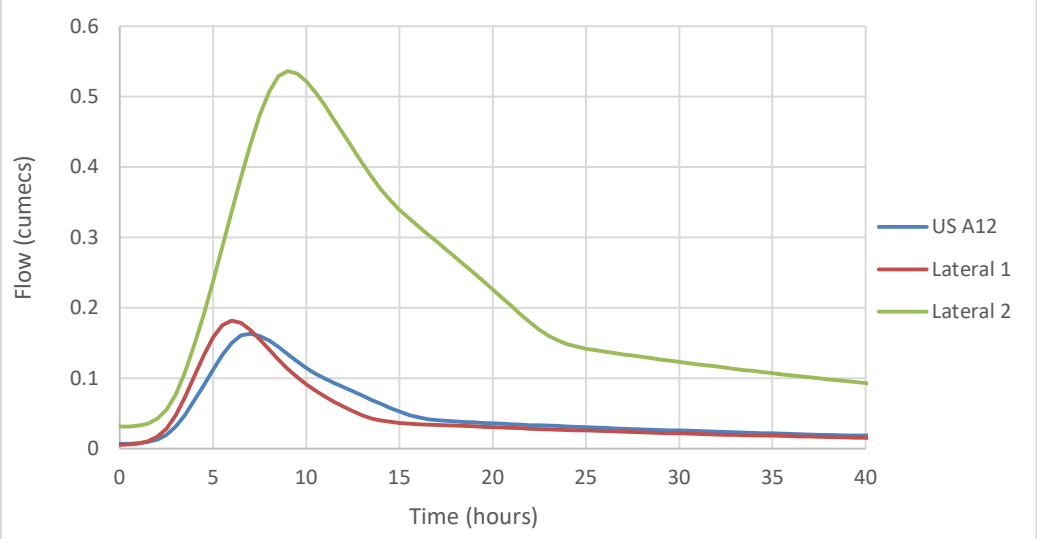
3.6.2 The Run 1 1% (1 in 100) AEP event hydrographs for the upstream catchment to the A12 crossing (US A12) and for the residual catchments (Lateral 1 and Lateral 2) are presented in Plate 3.2.

Plate 3.2 Run 1 - 1% (1 in 100) AEP Hydrographs



3.6.3 The Run 2 1% (1 in 100) AEP event hydrographs for the upstream catchment to the A12 crossing (US A12) and for the residual catchments (Lateral 1 and Lateral 2) are presented in Plate 3.3.

Plate 3.3 Run 2 – 1% (1 in 100) AEP Hydrographs



3.7 Climate Change

3.7.1 A climate change allowance for peak rainfall intensity was obtained from the latest government guidance at the time of assessment (Environment Agency, 2021). The peak rainfall intensity allowance is appropriate for small catchments (less than 5km^2). An uplift of 40% (the upper end, 2080s scenario) was applied. For Run 1 this resulted in a peak flow at the A12 crossing (US A12) of 0.237m^3/s. For Run 2 the peak flow following the 40% uplift at the A12 crossing (US A12) is 0.236m^3/s.

3.8 Flow Reconciliation

3.8.1 For Run 1 flow reconciliation at the downstream model extent could be undertaken by adjusting the residual catchment flows only (Lateral 1 / Lateral 2). As described previously, this was not required as Run 1 hydrology was not utilised in the model. As the inflows to the A12 crossing were calculated using ReFH2.3, flow reconciliation was not considered necessary at the A12 crossing for Run 2.

3.9 Summary

3.9.1 Design peak flows and hydrographs for MWC26 were derived using ReFH2.3, using the FEH catchment descriptors available from the FEH Web Service (with manual calculation of FEH catchment descriptors where appropriate) and adopting climate change allowance as per the latest EA guidance. Design inflow hydrographs were derived for two hydraulic model runs, namely Run 1 – for the longer critical storm duration to the originally envisaged downstream model extent and Run 2 – for the

critical storm duration to the target location, i.e., the A12 crossing. The hydrology that was adopted in the model, was that from Run 2, based on the finalised model extent.

4 Baseline Model Build

4.1 Methodology

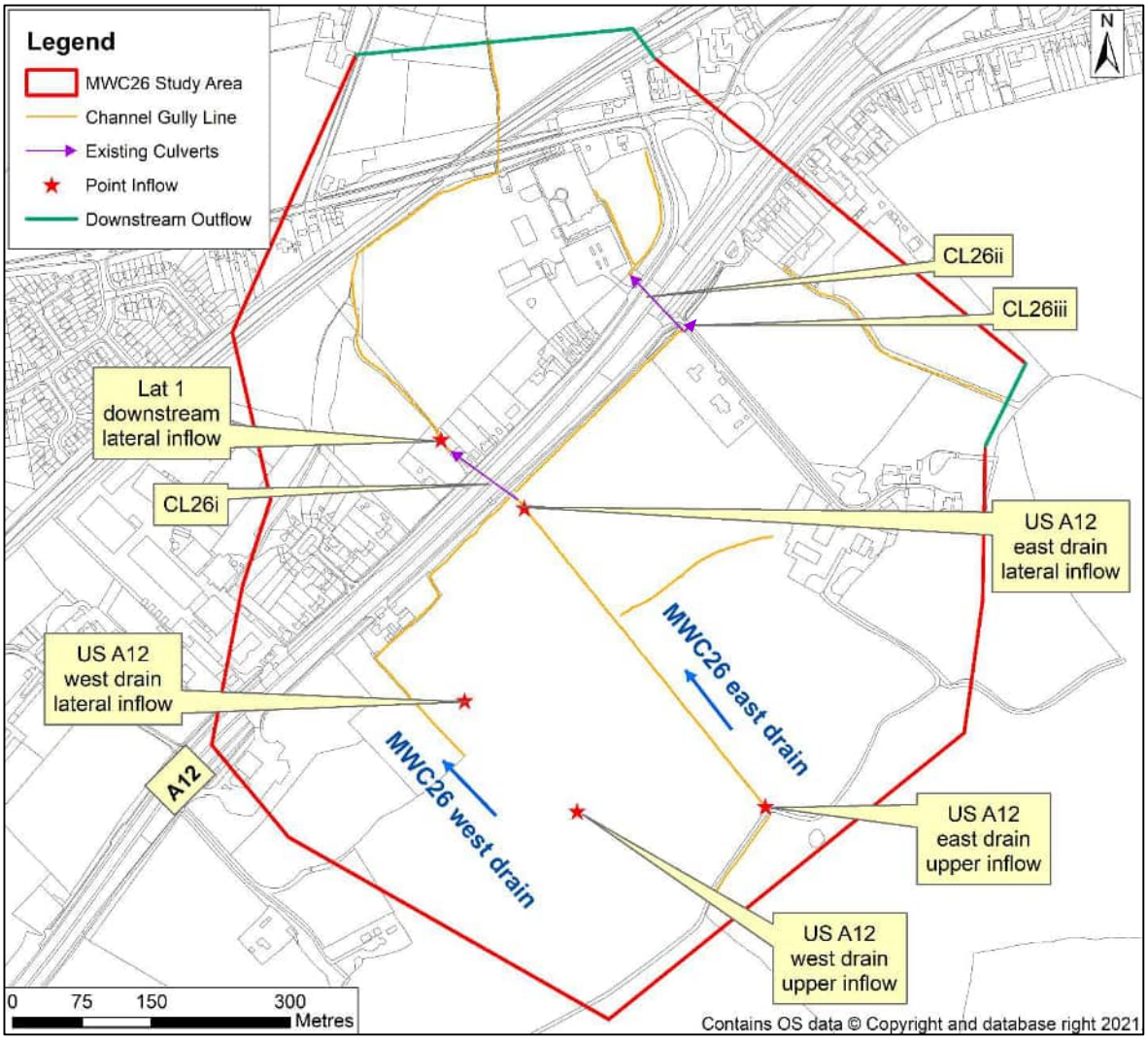
4.1.1 The MWC26 hydraulic model uses a two-dimensional (2D) schematisation, built in TUFLOW version 2020-10-AA (BMT, 2020). 1D 'ESTRY' elements were used to represent the culverts within the TUFLOW model and linked to the 2D domain via SX links. The 2D representation of the channel was considered appropriate as the highway culvert controls pass-forward flows and upstream water levels. Therefore, detailed representation of the channel capacity is not required.

4.1.2 The model was built from scratch and used to simulate the baseline, with-scheme and mitigation scenarios. Three design events were simulated for each scenario: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) AEP +40%CC events.

4.2 Model Extent

4.2.1 The 2D model domain covers an area of 0.7km^2. The model extent has been drawn in a way to capture the floodplains of each tributary and the downstream watercourse. Representation of the model extent is shown in Plate 4.1, and model configuration is explained in detail in Section 4.3. It should be noted that inflows have been placed at the location of the natural flow path of the terrain.

Plate 4.1 MWC26 TUFLOW Model Setup



4.3 Model Resolution and Topography

4.3.1 1m horizontal resolution LiDAR DTM data from 2020 was available for the model extent and was downloaded from the DEFRA website to inform the hydraulic model with ground level information.

4.3.2 A 1m grid cell resolution was deemed appropriate for the model; as this is sufficient to represent the open channel, the floodplain topography and the active floodplain details whilst avoiding a long simulation run time. The model grid has been oriented to be aligned with key flow paths.

4.3.3 The open channel sections were represented in the model using 1.5m wide gully lines to define channel width, whereby the river bed levels have been reinforced using nominal point data taken from a channel survey (see Section 2). Bridge structures upstream and downstream of the A12 have been ignored, as investigation showed that these features would not

affect the model results within the area of interest. This includes the railway culvert, located 320m downstream of the eastern A12 crossing.

4.3.4 The watercourse crosses the A12 via two separate inlets. The east and west drains converge and enter a 450mm pipe that discharges at the southwest corner of the playing fields downstream of the A12 crossing. A toe drain at the 450mm pipe inlet also conveys flows east alongside the A12 towards Hall Chase, where a 300mm culvert provides a second crossing of the carriageway. A 150mm culvert allows water to cross Hall Chase from west to east and discharges it onto farmland east of Hall Chase where it ponds within a local depression. To summarise, MWC26 tributary culverts consist of:

- A 450mm culvert (**CL26i**), at the confluence of the two upstream tributaries, which discharges immediately downstream of the A12.
- A 300mm culvert (**CL26ii**), at the downstream end of the toe drain running along the upstream side of the A12, which discharges immediately downstream of the A12.
- A 150mm culvert (**CL26iii**), at the downstream end of the toe drain running along the upstream side of the A12, which discharges immediately east of Hall Chase.

4.3.5 MWC26 runs north for approximately 1.4km before finally discharging into the Roman River. It was not considered necessary to model the watercourse to the Roman River and an appropriate downstream boundary was chosen at the location shown in Plate 4.1 (Downstream Outflow). Culverts were schematised in the model using embedded 1D 'ESTRY' elements with a Manning's 'n' roughness value of 0.013, which is a mid-value for either metal or concrete culvert types (Chow, 1959). Culvert losses are applied based on standard recommended values; 0.5 and 1 for the inlet and outlet loss, respectively. CL26i has a bend in the pipe, and so the hydraulic losses were modified to account for this.

4.4 Hydraulic Friction

4.4.1 OS MasterMap data was used to identify land use type and inform the TUFLOW model with different hydraulic roughness (Manning's 'n' coefficient) values. Hydraulic roughness coefficients are applied over each grid cell of the 2D domain depending on the land use taken from the MasterMap data, as shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow, 1959). Depth-varying roughness has been used to stabilise the model at shallow depths. Roughness values adopted are shown in Table 4.1. For depths between 50mm and 300mm, Manning's 'n' value is interpolated between n1 and n2. Below 50mm, n1 is applied, and above 300mm, n2 is applied. The default interpolate method uses a curved fit so that the n values transition gradually.

4.4.2 As mentioned in Section 4.3.5 above, the culverts were modelled assuming a roughness value of 0.013, taken from Chow (1959) as a mid-

value for either concrete or metal culverts that are straight and free of debris.

Table 4.1 MWC26 hydraulic roughness

Land Use	Depth varying Manning's '' roughness	
	n1	n2
Manmade Surface	0.05	0.025
General Surface (Step)	0.05	0.025
Property Gardens	0.1	0.05
Roads/Tracks/Paths	0.05	0.025
Manmade Structures	0.05	0.025
Slopes	0.1	0.05
Buildings	1.0	
Natural Land	0.1	

4.5 Boundary Conditions

4.5.1 Two types of boundary conditions have been implemented in the model and can be seen in Plate 4.1.

- QT boundaries were applied to the 2D domain at the upstream ends of the two tributaries (upper inflows) and immediately downstream of the proposed-scheme location within both drains (upper lateral inflows). The sum of those inflows corresponds to the contributing area of the two tributaries to the north of A12 (US A12). A lateral inflow was applied downstream of the existing A12 crossing at the outlet of CL26i to represent the residual catchment located between A12 and the railway line (Lat 1). The residual catchment standing between the railway line and Roman River (Lat 2) has not been represented as it stands too far from the scheme. The catchment hydrology analysis is explained in Section 3.
- Water level vs flow (HQ) boundaries were implemented to the north and east of the model extent to allow channel flows out of the model and prevent 'glass-walling' of water. The boundary applies a normal depth assumption with slope determined by inspection of the LiDAR data.

5 Modelled Events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 40% uplift in accordance with the latest EA guidance as detailed in Section 3.

5.1.2 As discussed in Section 3, two model runs were undertaken as part of deriving the hydrological inflows – Run 1 and Run 2. Run 2 relates to the adopted model extent and was utilised in the analysis going forward.

5.1.3 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients and hydrological inflows (see Section 6.4).

Table 5.1 Modelled events for MWC26

Scenario	5% AEP (1 in 20)	1% AEP (1 in 100)	1% AEP +40%CC (1 in 100)
Baseline	✓	✓	✓
Design	✓	✓	✓
Mitigation	✓	✓	✓
Roughness Sensitivity		✓	
Hydrological Inflow Sensitivity		✓	

6 Model Proving

6.1 Introduction

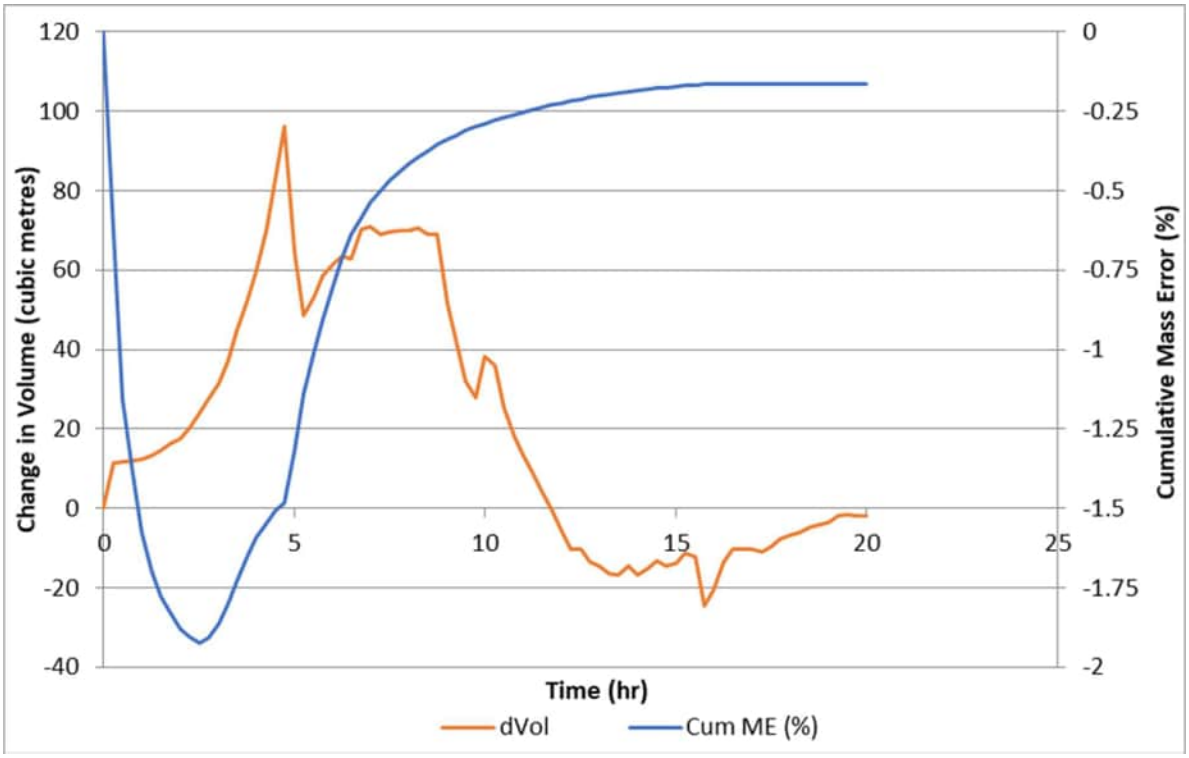
6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model Performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model computational performance was achieved. The cumulative mass error diagnostics output from the model have been checked for each model simulation. Plate 6.1 shows the cumulative mass error (Cum ME) and the change in volume (dVol) for the baseline 1% (1 in 100) AEP event simulation. This plot is typical for all the events and scenarios simulated. The tolerance range recommended by the software manual is +/-1% mass balance error.

6.2.2 The cumulative mass error tolerance is exceeded at the onset of the floodplain wetting but stabilises to less than 1% error well before the peak of the inflow hydrographs (Plate 3.2) and the corresponding dVol.

Plate 6.1 Model proving Cumulative ME and dVol plot for 1% (1 in 100) AEP event



6.3 Comparison with EA Flood Maps

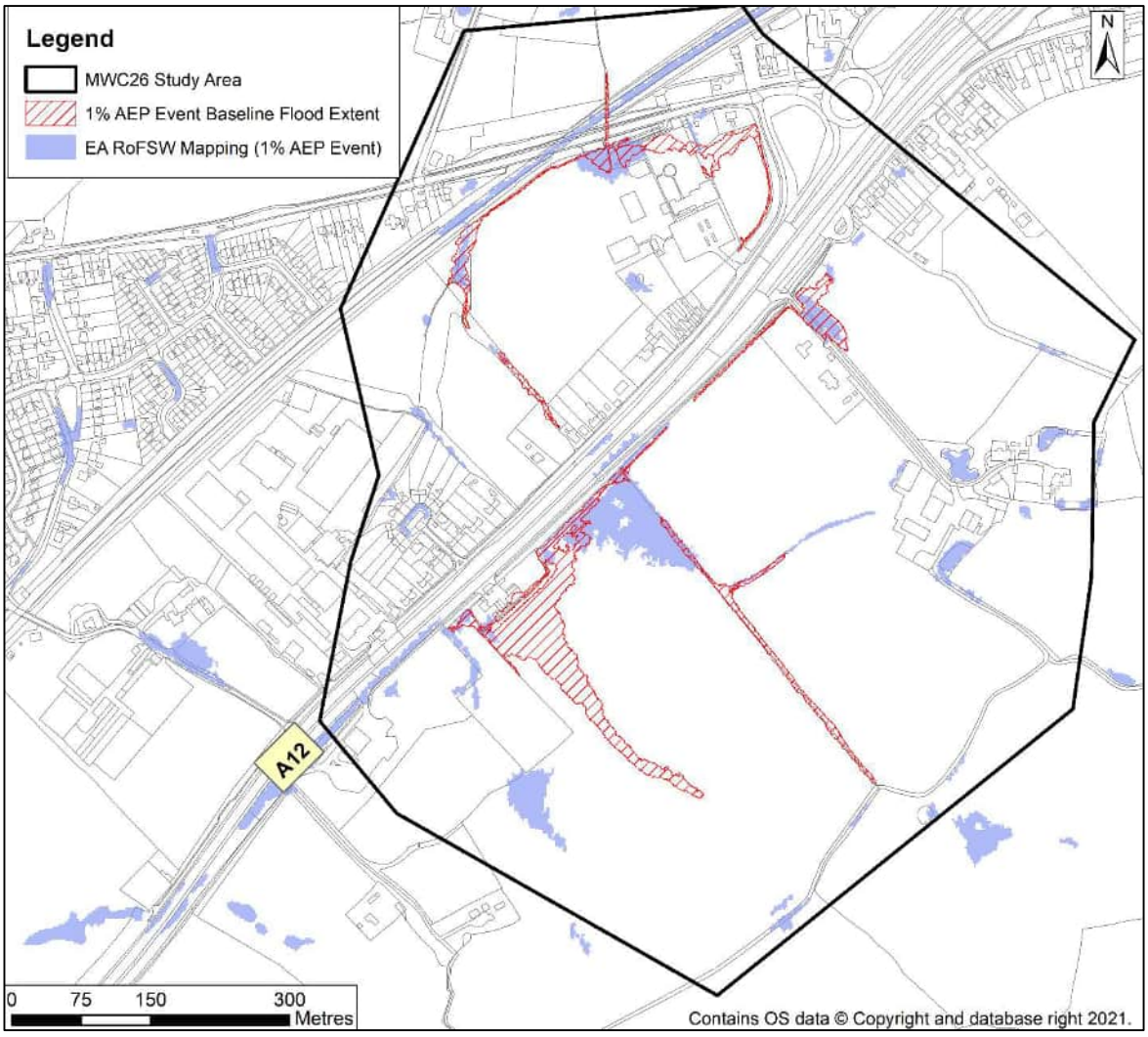
6.3.1 MWC26 has not been included in the Environment Agency (EA) published Flood Zone mapping of fluvial flood risk. Therefore, a direct comparison between model results and EA flood maps is not possible. However, the EA Risk of Flooding from Surface Water (RoFSW) mapping can be used to identify areas of pooling.

6.3.2 Plate 6.2 shows the modelled 1% (1 in 100) AEP event flood extent and the EA RoFSW mapping extents. Flood extents are similar around Hall Chase and immediately upstream of the railway line.

6.3.3 The EA RoFSW map shows a large area of ponding against the A12 embankment upstream of culvert CL26i that is not predicted in the model. This is because the existing culvert under the A12 here has not been accounted for in the modelling methodology employed to produce the EA maps.

6.3.4 Another noticeable discrepancy between the two flood extents is in the field upstream of the A12 between the east and west channels. At this location the EA RoFSW flood map shows no risk of flooding, whereas the model predicts widespread flooding across the floodplain. As the inflows are directly applied into modelled watercourses within the MWC26 model, flows are transferred to locations that are not necessarily picked up by the RoFSW flood map, as the modelling methodology employed by the latter does not represent watercourses. Considering this, full verification of the model cannot be based on the comparison of results with EA RoFSW maps. The analysis is therefore reliant on the use of industry standard modelling and hydrology tools, as is common for small watercourse studies.

Plate 6.2 EA Risk of Flooding from Surface Water and 1% (1 in 100) AEP Extent



6.4 Sensitivity Analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness and hydrological inflows. All sensitivity tests were performed using the 1% (1 in 100) AEP event.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Plates B.1 and B.2 in Appendix B.1. shows the impact of changing the model roughness on predicted peak water levels. The results show that the floodplain upstream of the A12 embankment is moderately sensitive to adjustment of model roughness, with a water level response of +60mm when roughness is increased.

Decreasing roughness results in a -180mm change in in-channel water level immediately upstream of the A12. Downstream of the A12, the watercourse is showing almost no sensitivity to changes in roughness.

Hydrological Inflow Sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Plates B.3 and B.4 provided in Appendix B.2 show the impact of changing the model inflows on predicted peak water levels. The results show that adjusting inflows by +20% results in flooding of the left floodplain of the east drain by +115mm. A -20% adjustment results in a change of in-channel water levels by -195mm immediately upstream of the A12. At the downstream end of the model extent, adjustments to inflows result in a +380mm and -325mm change to in-channel water levels, when inflows are increased and decreased, respectively.

7 Design Modelling

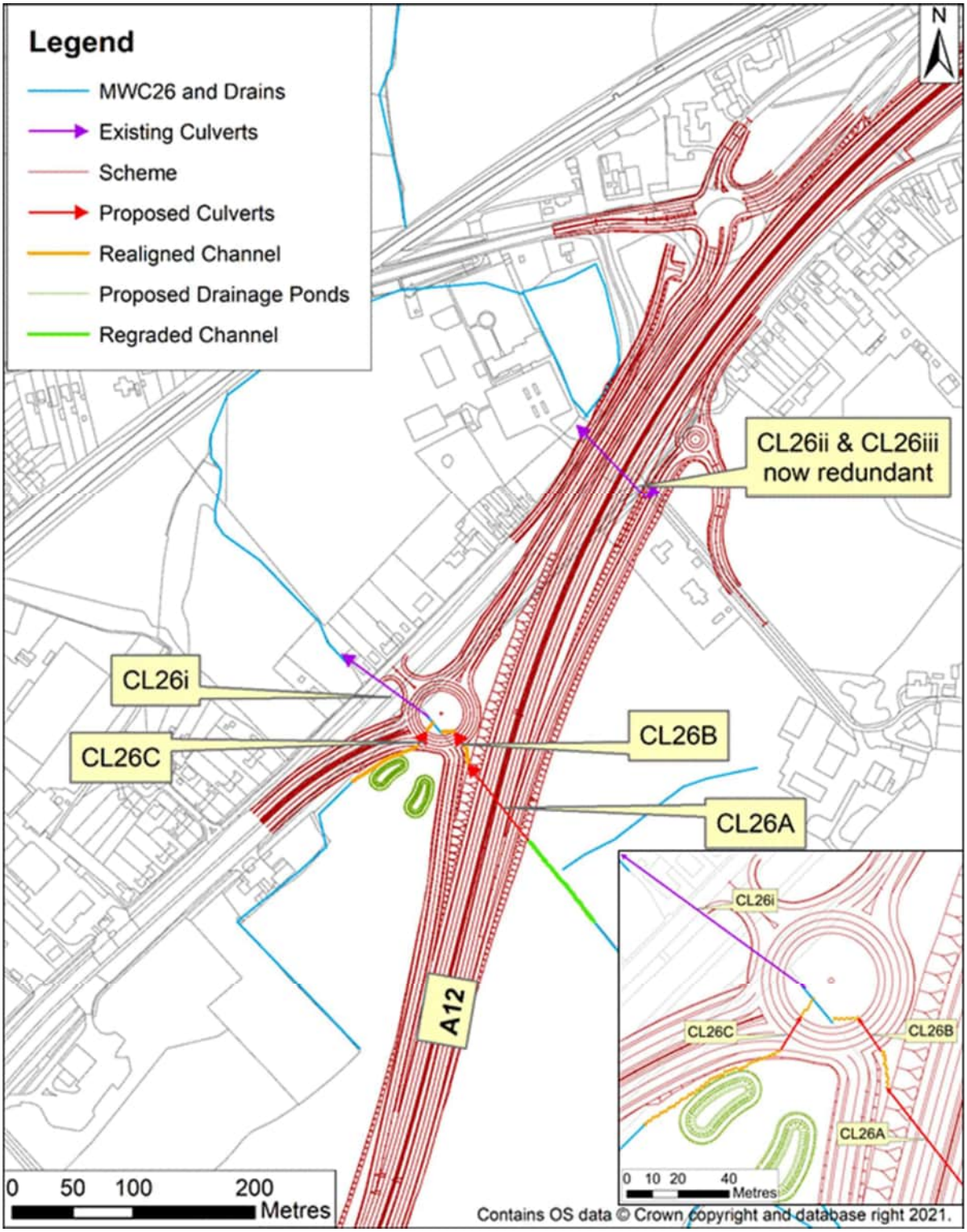
7.1 Design Model Build

7.1.1 The baseline model was updated to include the proposed A12 scheme. Changes to the model were as follows and as illustrated in Plate 7.1:

- The proposed A12 road design was included in the model as a modification to the baseline model topography using an ascii DTM surface overlain.
- The proposed mainline cuts across the flow path of the eastern drain. A new proposed 450mm circular concrete culvert (CL26A) conveys the eastern drain flow under the proposed mainline, discharging into a realigned 1.5m-wide channel. Downstream of the new mainline crossing, the drain is realigned towards an additional proposed 450mm concrete circular culvert (CL26B) that outfalls into the centre of the new junction roundabout. Upstream of the new mainline, the eastern drain was simply regraded to produce a smooth gradient between the open and culverted sections.
- The upper part of the western tributary was redirected immediately upstream of the new A12 carriageway via a 1.5m-wide open channel running along the toe of the realigned highway embankment and discharging into the regraded eastern drain immediately upstream of CL26A culvert.
- The lower part of the western tributary remains unchanged whilst running along the toe of the existing A12 embankment until being realigned in order to cross the new junction roundabout via a 450mm circular concrete culvert (CL26C) and finally discharge into the realigned eastern drain in the centre of the roundabout. From there, the realigned channel redirects both flows towards the existing culvert CL26i.
- All three existing culverts were left unchanged in the model. It should be noted however that CL26ii and CL26iii were made redundant as they lie underneath the proposed Junction 25.
- Manning's roughness layer was updated to reflect the additional road surfaces resulting from the A12 proposed-scheme. Manning's 'n' values were updated as per Table 4.1.

7.1.2 As part of the proposed-scheme, highway drainage attenuation ponds will occupy the west drain floodplain in between the existing A12 and new A12 crossing, immediately west of Junction 25 roundabout. As such, those have been included within the MWC26 design model, in the form of a two pond and bund structures represented using TUFLOW Z shape polygon features: the bunds have their crest set 300mm above existing ground level, the final design level will be determined in future design stages.

Plate 7.1 Design Model Updates



8 Model Results

8.1 Baseline Results

8.1.1 Plate 8.1 shows the maximum flood extent for all modelled events in the baseline scenario. The flood extents for each of the modelled events can be found in Appendix C.

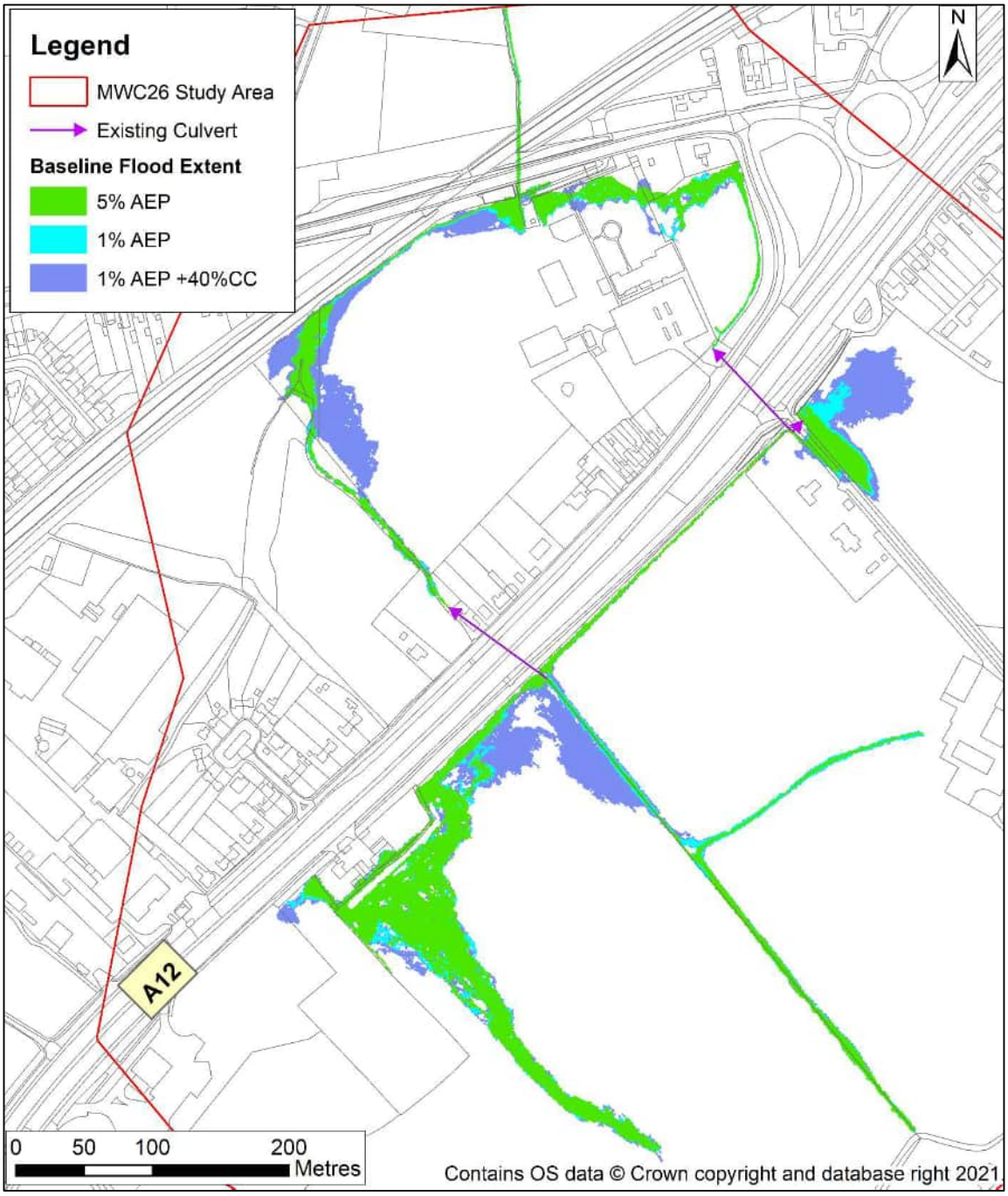
8.1.2 The area immediately upstream of A12 is flooded for all events simulated, with almost identical flood extents in the 5% (1 in 20) and 1% (1 in 100) AEP events and without impounding behind the A12. This is due to the upstream field being extremely flat, with large drains acting to contain the flow in-bank, with the exception of Hall Chase area where impounding is observed for all events. This natural floodplain gets occupied for the 1% (1 in 100) AEP +40%CC event with impounding observed against the A12 in the vicinity of CL26i.

8.1.3 For all return periods, 71-79% of flows upstream of the A12 are conveyed through CL26i, while 9-16% flows east along the toe drain towards Hall Chase. Of this, 26-33% crosses the A12 via CL26ii, while 6-8% flows through CL26iii to the farmland east of Hall Chase. The remainder pools on the west side of Hall Chase before overtopping the road and spilling into the farmland, inundating the land to depths of nearly 900mm during a 1% (1 in 100) AEP +40%CC event. All properties on Hall Chase remain dry for all storm events.

8.1.4 The most western drain upstream of the A12 crossing is unable to contain flows for all storm events, causing flooding to 5 properties. During a 1% (1 in 100) AEP +40%CC event, these properties flood to depths of up to 230mm.

8.1.5 There is no flooding to the A12 highway. Downstream of the A12 crossing, for all modelled events there is some flooding to the floodplains, but all flow remains in-channel downstream of the railway line.

Plate 8.1 Baseline Flood Extents



8.2 With-Scheme Model Results

8.2.1 Plate 8.3 shows the changes to the water level due to the proposed A12 scheme for the 1% (1 in 100) AEP +40%CC event. The maximum flood level differences between baseline and with-scheme scenarios are categorised into levels of impacts due to the proposed-scheme. The impact of the proposed-scheme for all other flood events can be found in Appendix C.

8.2.2 In all events the A12 carriageway remains unaffected by flooding, whilst its associated slip roads and roundabout are severely impacted by flooding for the 1% (1 in 100) +40%CC event. The baseline flooding has been displaced south towards the middle of the roundabout and west around the mitigation ponds as seen in Plate 8.2. Whilst the upstream flood extent remains unchanged in the with-scheme scenario for all events, flood depths local to the proposed-scheme and downstream of it are modified across most of the study area. Flood levels generally increase upstream of the existing A12 due to the reduction of the flood storage due to the proposed-scheme, slip roads and associated Junction roundabout.

8.2.3 There is now no flooding near Hall Chase because the new crossing occupies the ditch, so flows can no longer go through it. The properties immediately upstream of the existing A12 to the west experience shallower flooding, to depths 10-50mm lower than in the baseline model as the upper flows of the western drain are redirected due to the proposed-scheme.

8.2.4 For the 1% (1 in 100) AEP and 1% (1 in 100) AEP +40%CC events, the existing culvert CL26i and proposed culverts CL26A, CL26B and CL26C are all surcharged. For the 5% (1 in 20) AEP event, only the existing culvert (CL26i) is surcharged.

8.2.5 During a 1% (1 in 100) AEP +40%CC event, the roundabout floods to depths of 50-70mm. A section of the A12 mainline to the south also experiences minor flooding below depths of 5mm.

8.2.6 In all events, the extent of flooding has been reduced due to the loss of storage caused by the proposed-scheme. As a consequence, maximum water levels are increased upstream of CL26i and more flows go through it. This is further exacerbated by the lack of drainage at Hall Chase now that CL26ii and CL26iii are disabled, which allows greater volumes of water to pond there. The increased conveyance through CL26i results in an increase in peak flow to the downstream channel, most noticeably in a 5% (1 in 20) AEP event.

8.2.7 The proposed-scheme has a detrimental effect on maximum water levels downstream of the A12 carriageway, most noticeably within the toe drain alongside the railway line, where a 25mm increase in maximum water levels is experienced in a 1% (1 in 100) AEP +40%CC event The increase in downstream water levels due to greater pass forward flow, as well as

flooding to the proposed-scheme itself, mean mitigation works are deemed necessary.

Plate 8.2 With-Scheme Flood Extents

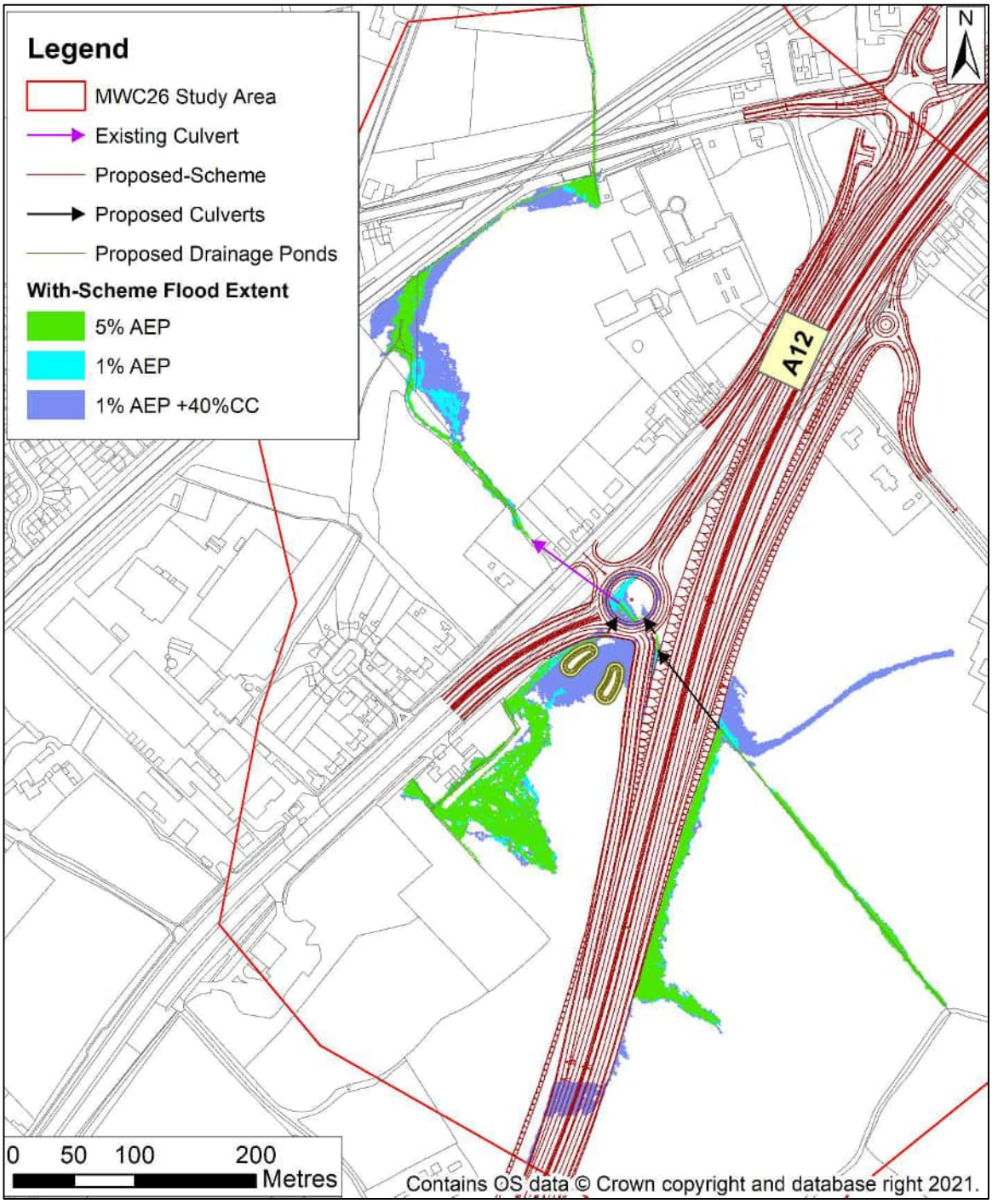
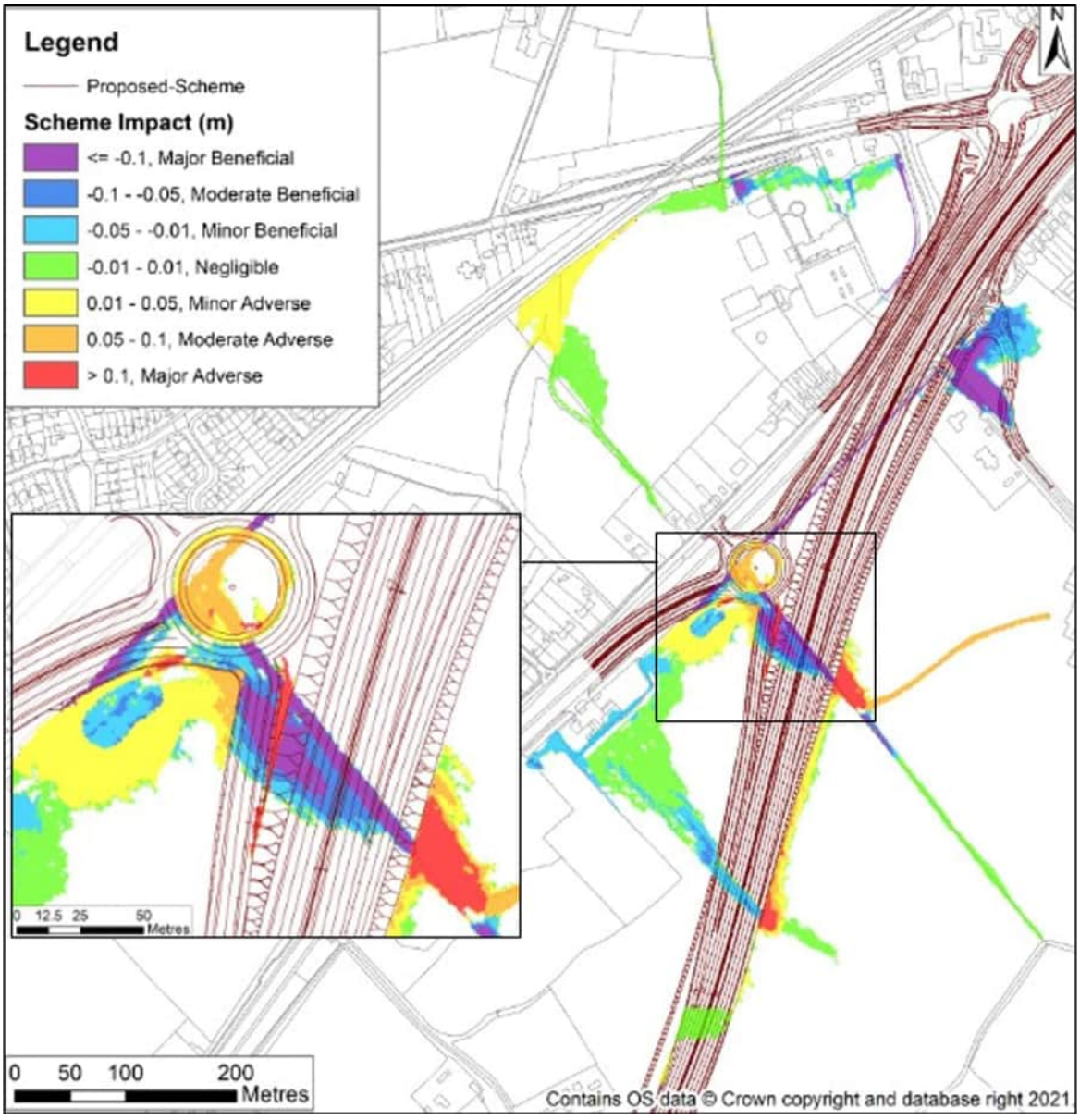


Plate 8.3 Water Level Difference (With-scheme – Baseline) for 1% (1 in 100) AEP +40%CC



9 Mitigation Modelling

9.1 Mitigation Model Build

9.1.1 The with-scheme model was updated to include mitigation measures to address the impacts resulting from the proposed-scheme.

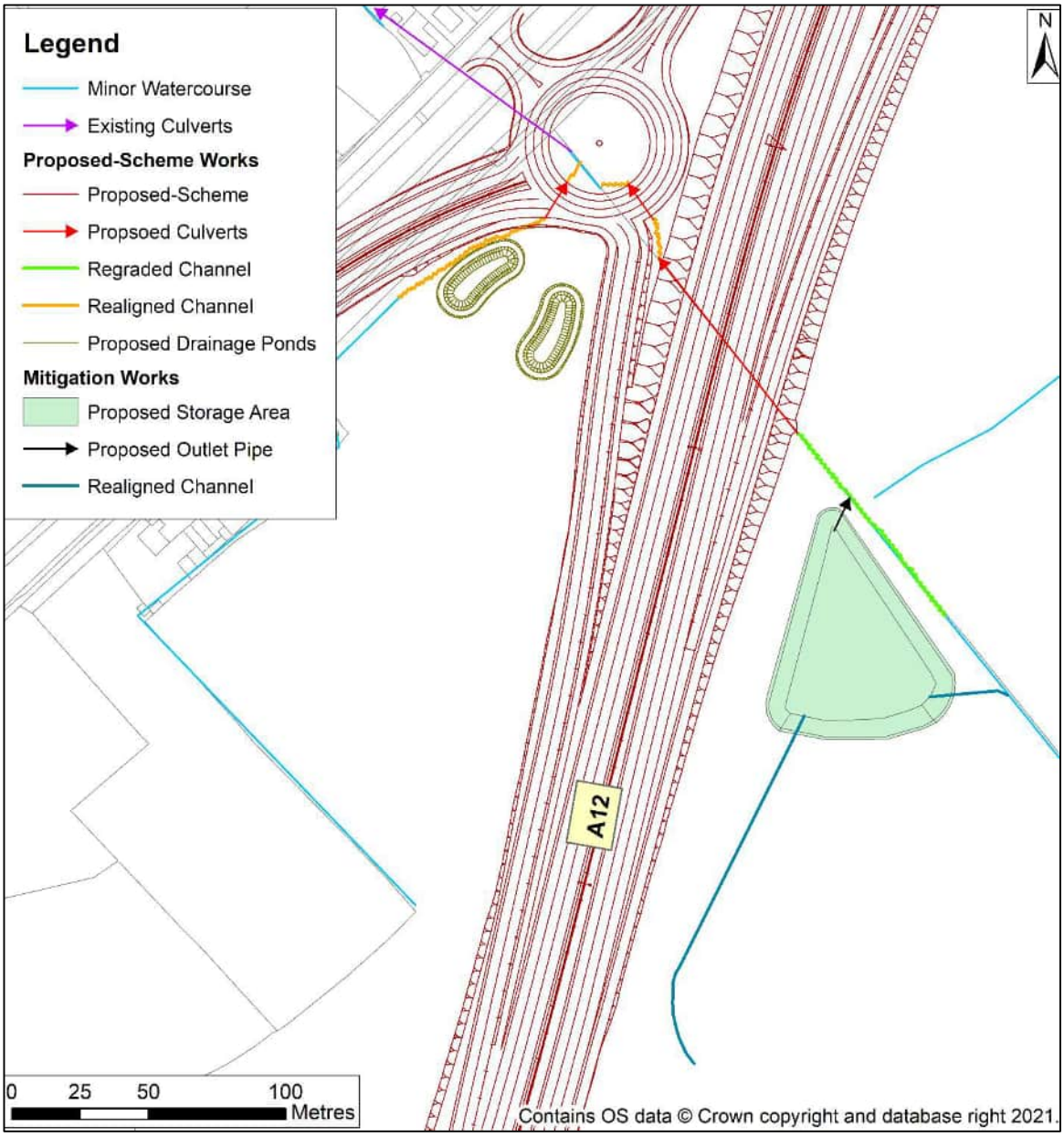
9.1.2 Various mitigation options were considered and trialled. This resulted in a preferred option that was found to provide flood protection to the A12 Highway up to a 1% (1 in 100) AEP +40%CC standard of protection. The preferred option also ensures that there are no adverse impacts downstream of the proposed-scheme.

9.1.3 Changes to the model were as follows and as illustrated in Plate 9.1:

- An online flood storage area was represented in the model approximately 30m upstream of the proposed culvert CL26A. This required the realignment of the upper western drain and the implementation of a flow split on the eastern drain to discharge part of the channel flow into the storage area. An outlet pipe of 300mm diameter was also included to control flows at the outfall.

- The model topography was therefore modified to represent the newly excavated channels required to divert flow into the storage area from both the east and west drains.

Plate 9.1 MWC26 Mitigation Setup



9.2 Mitigation Model Results

9.2.1 Plate 9.3 shows the changes to the water level due to the mitigation works for the 1% (1 in 100) AEP +40%CC event. The maximum flood level differences between baseline and with-mitigation are categorised into levels of impact. The impact of the proposed-scheme for all other flood events can be found in Appendix D.

9.2.2 The proposed-scheme remains dry for all AEP events as demonstrated by Plate 9.2.

9.2.3 The realigned channel directing the west drain towards the storage area prevents ponding against the proposed A12 embankment in all events.

9.2.4 Despite the fact a proportion of the east drain flows are now conveyed into the storage area, there still remains considerable ponding against the proposed-scheme at the inlet of CL26A, to depths of up to 200mm in a 1% (1 in 100) AEP +40%CC event. The adverse impacts of the mitigation scenario on maximum flood depths compared to the baseline are constrained to within the channel, with no detrimental effect on the floodplains.

9.2.5 The peak flow through the existing culvert CL26i is delayed and lower than in the baseline and with-scheme scenarios. This is because water is being held in the storage area upstream of the A12.

9.2.6 Downstream of the proposed-scheme, backing up of water against the railway embankment is heavily reduced compared to the baseline, with a reduction in maximum water level of up to 1m for the for the 1% (1 in 100) AEP +40%CC event.

Plate 9.2 Mitigation Flood Extents

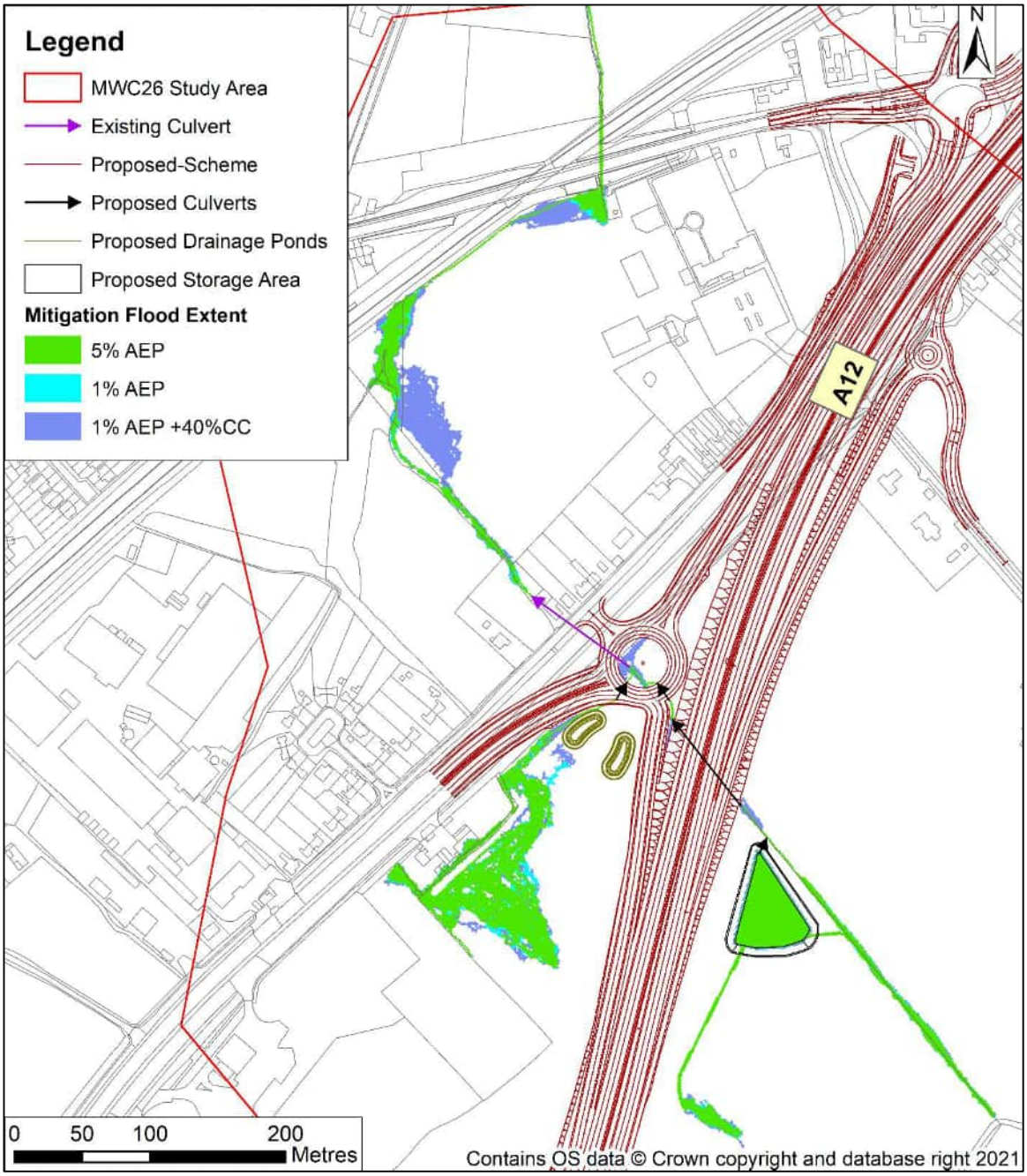
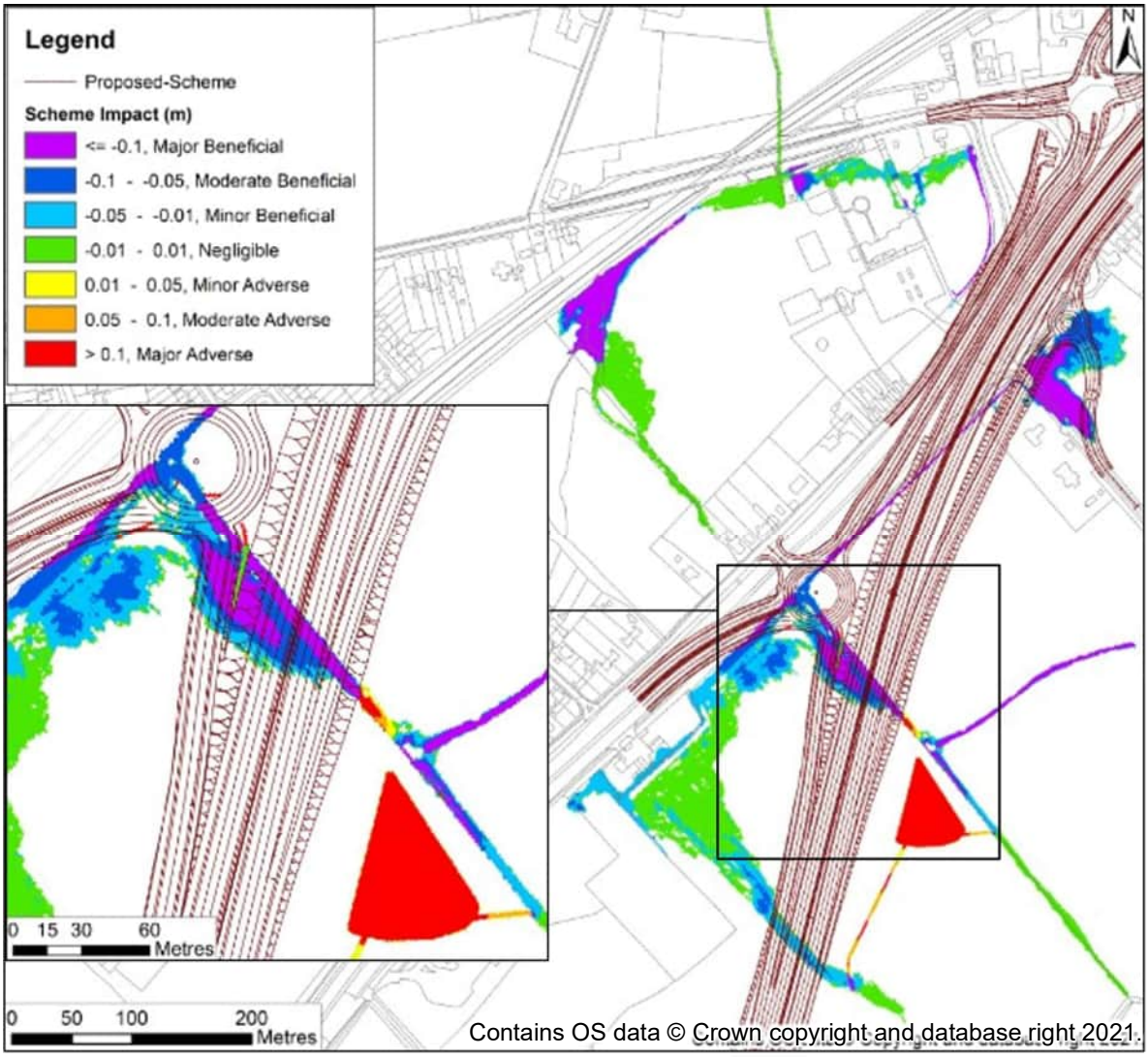


Plate 9.3 Water Level Difference (Mitigation – Baseline) for 1% (1 in 100) AEP +40%CC



10 Model Assumptions and Limitations

10.1 Introduction

10.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological, surveyed, and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

10.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed-scheme and are therefore, appropriate for the flood risk assessment.

10.2 Hydrology

10.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- As MWC26 is a small ungauged catchment there is inevitable uncertainty associated with flow estimation for small, ungauged catchments.
- ReFH2.3 is most appropriate for the target catchment as it uses BFIHOST19 and more recent (FEH13) rainfall data.
- FEH Statistical is not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 25 soils.

10.3 Hydraulic Modelling

10.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for MWC26 are as follows:

- Culvert and model roughness values have been assigned across the model using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the TUFLOW Manual. The dimensions for structures have been based on detailed survey measurements.
- The LiDAR data is assumed to appropriately represent the floodplain.
- A 1m grid has been used. This is deemed a sufficient level of detail to represent floodplain topography and flooding mechanisms predicted by the model in the vicinity of the proposed-scheme.

- No survey drawings of the Culverts CL26ii and CL26iii were received, and arrangement is based on the site visit photos provided. It should be noted that the exact dimensions and location will not impact results as CL26i pipe capacity is the key hydraulic controller of the A12 crossing.
- No calibration was carried out as the catchment is ungauged.
- The mitigation solution has been designed using TUFLOW-ESTRY software, and it demonstrates a successful solution to provide flood protection to the A12 Highway up to a 1% (1 in 100) AEP +40%CC standard of protection, and it also ensures that there are no adverse impacts downstream of the scheme. Detailed design may need to consider more in-depth hydraulic analysis to provide the required flow split on the eastern drain.

11 Conclusion

11.1.1 This report has detailed the modelling carried out to assess the baseline, with-scheme and with-mitigation flood risk for Minor Watercourse 26 and its tributaries with respect to the proposed highway scheme.

11.1.2 The results of the baseline modelling have shown that the existing culvert CL26i constricts flows moving south to north. However, the A12 carriageway is unaffected by flooding. Hall Chase floodplain naturally retains some important volumes transferred easterly via the toe drain, with CL26ii only able to transfer approximately 30% of this northward. Properties to the west of the catchment are flooded for all storm events.

11.1.3 The introduction of the proposed-scheme causes increased pass-forward flow to the downstream channel due to a combination of additional volumes no longer conveyed towards Hall Chase and reduced floodplain storage caused by the proposed-scheme. Furthermore, the proposed-scheme is inundated during a 1% (1 in 100) AEP +40%CC event. Therefore, mitigation works are deemed necessary.

11.1.4 To this effect, a storage pond, channel realignment and flow split were represented in the model upstream of the proposed A12 crossing, with water from both east and west drains directed into it. These mitigation measures are effective in preventing flooding to the proposed-scheme and produce no negative impacts downstream.

Acronyms

Abbreviation	Term
AEP	Annual exceedance probability
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate change
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SPRHOST	Standard percentage runoff derived using the HOST soil classification

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Appendix A – Minor Watercourse 26 Hydrology

See document HE551497-JAC-EWE-SCHW-RP-LE-0090

Appendix B – Sensitivity Testing

Plate B.1 Water level difference (Manning's 'n' Roughness changed by 20%) for the 1% AEP event

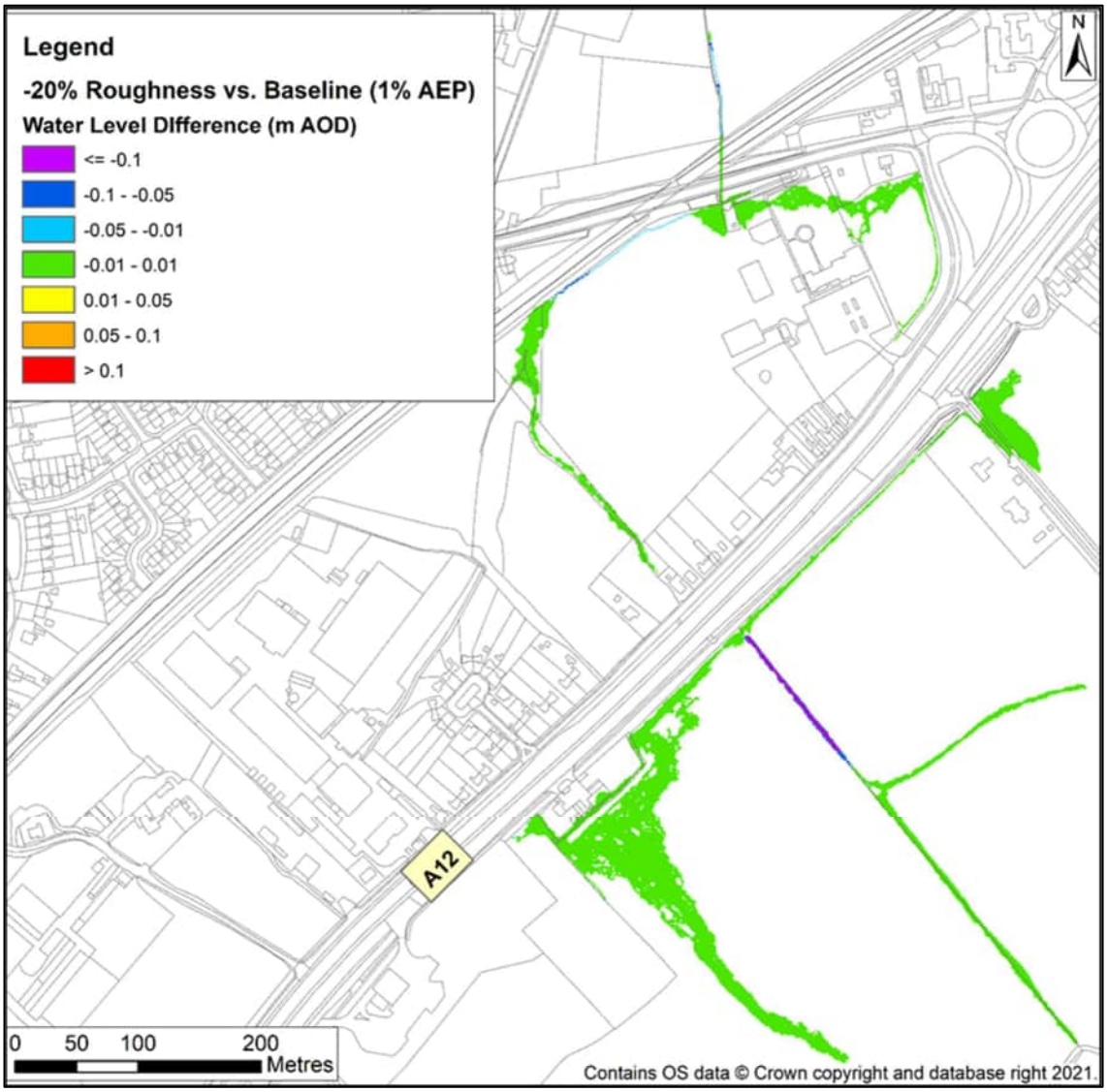


Plate B.2 Water level difference (Manning's 'n' Roughness changed by +20%) for the 1% AEP event

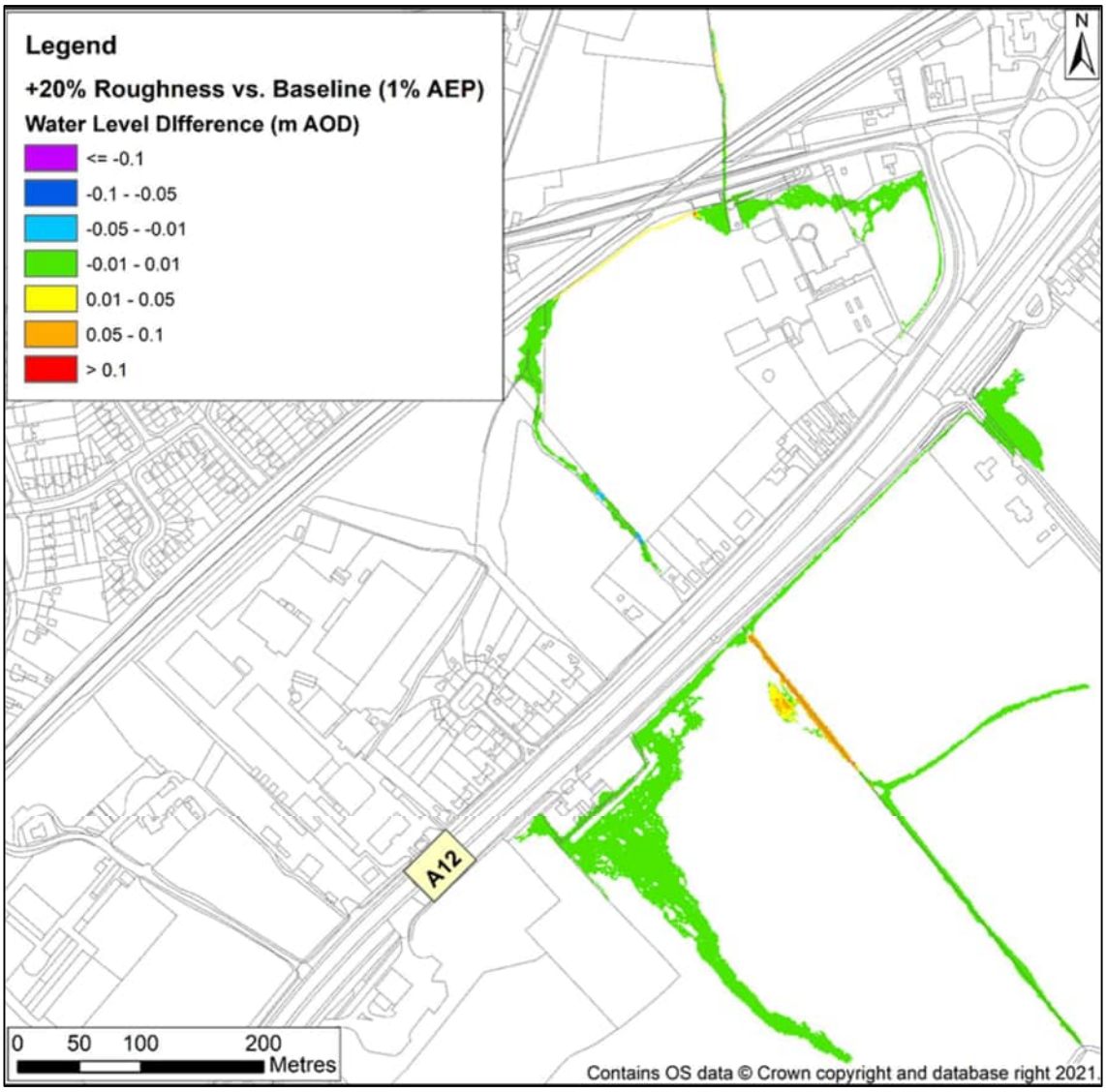


Plate B.3 Water level difference (Hydrological inflows into model changed by -20%) for the 1% AEP event

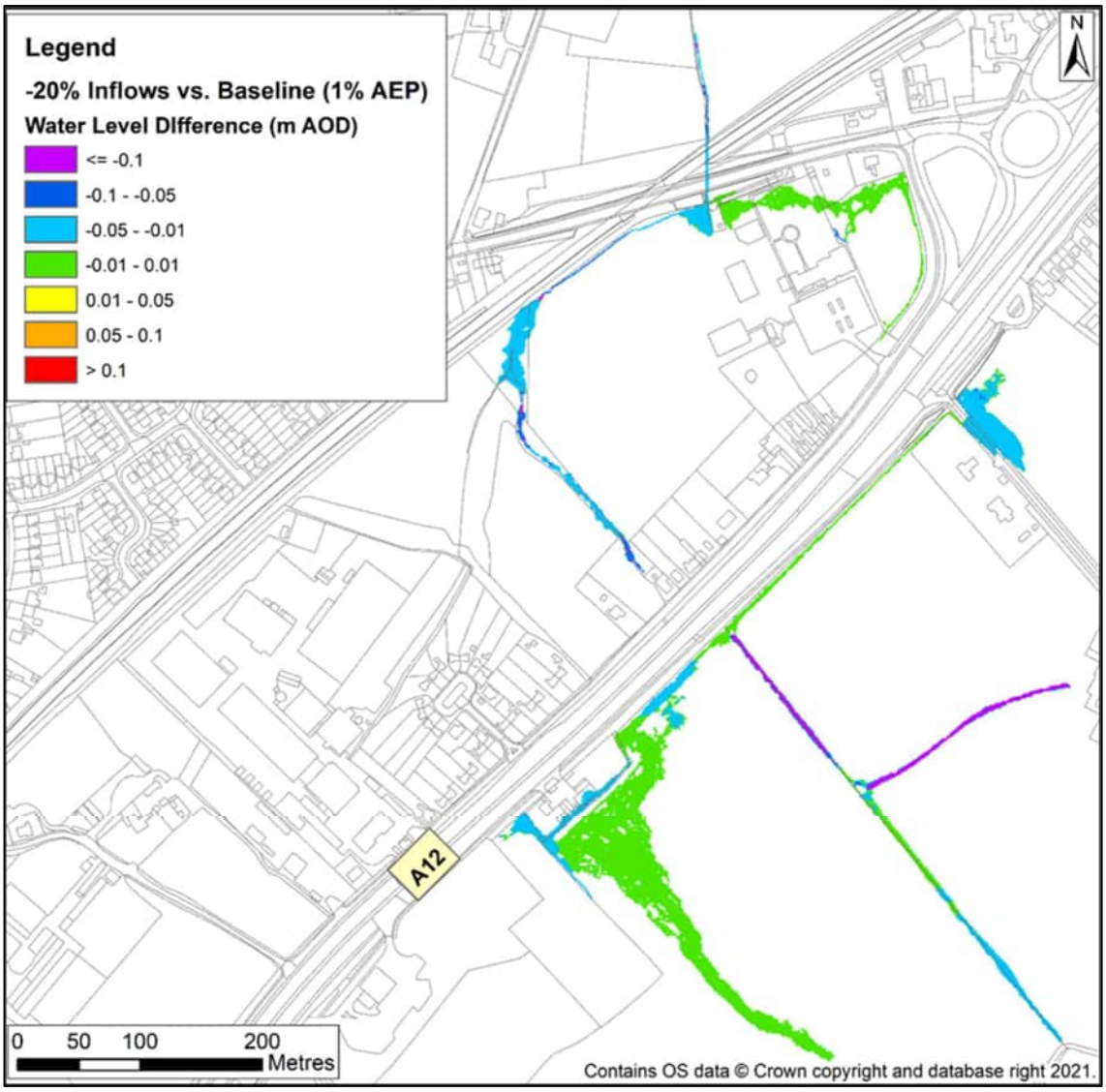
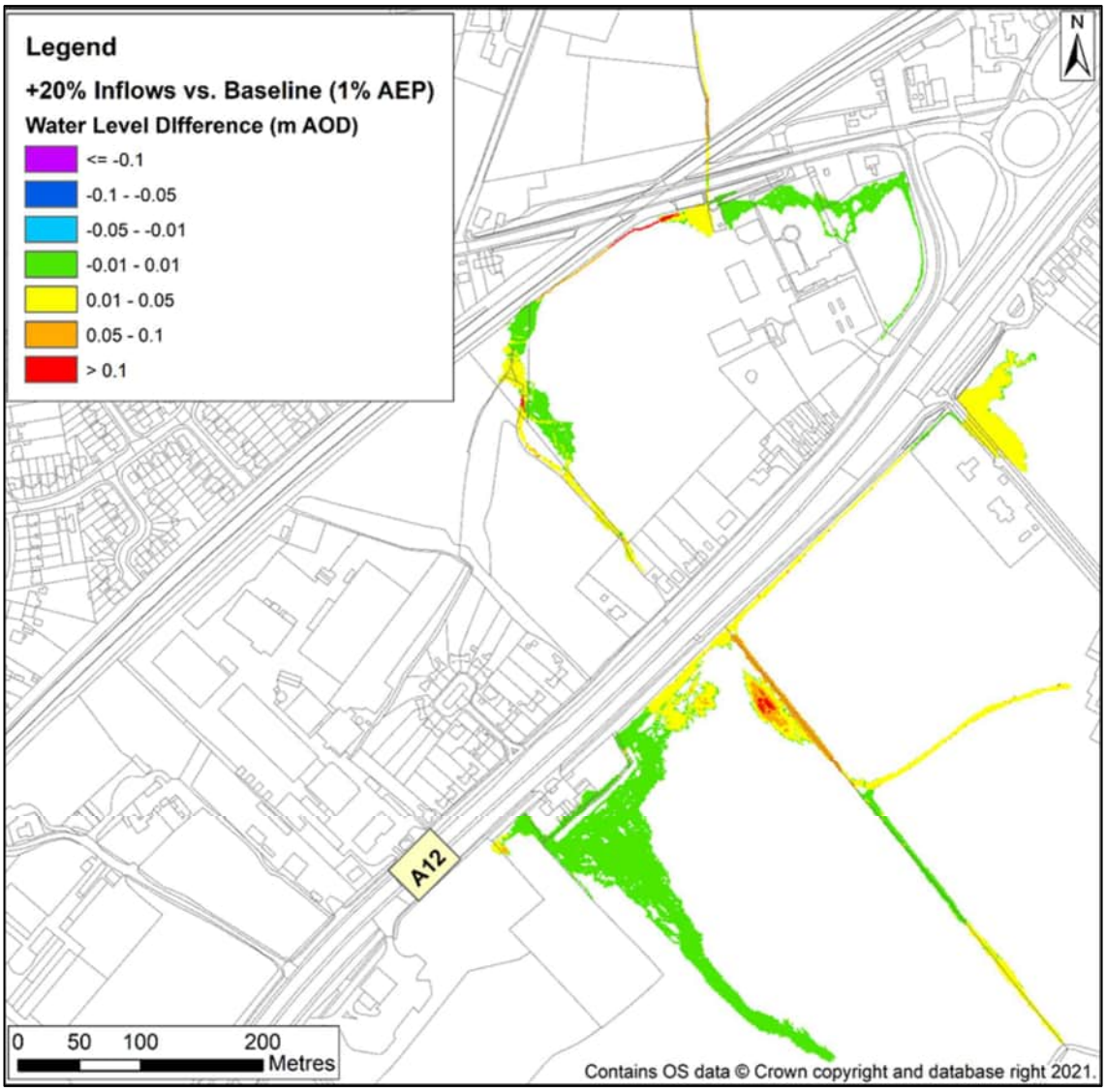


Plate B.4 Water level difference (Hydrological Inflows into model changed by +20%) for the 1% AEP event



Appendix C – With-Scheme Flood Mapping

Plate C.1 5% AEP maximum flood extent for baseline and with-scheme scenarios

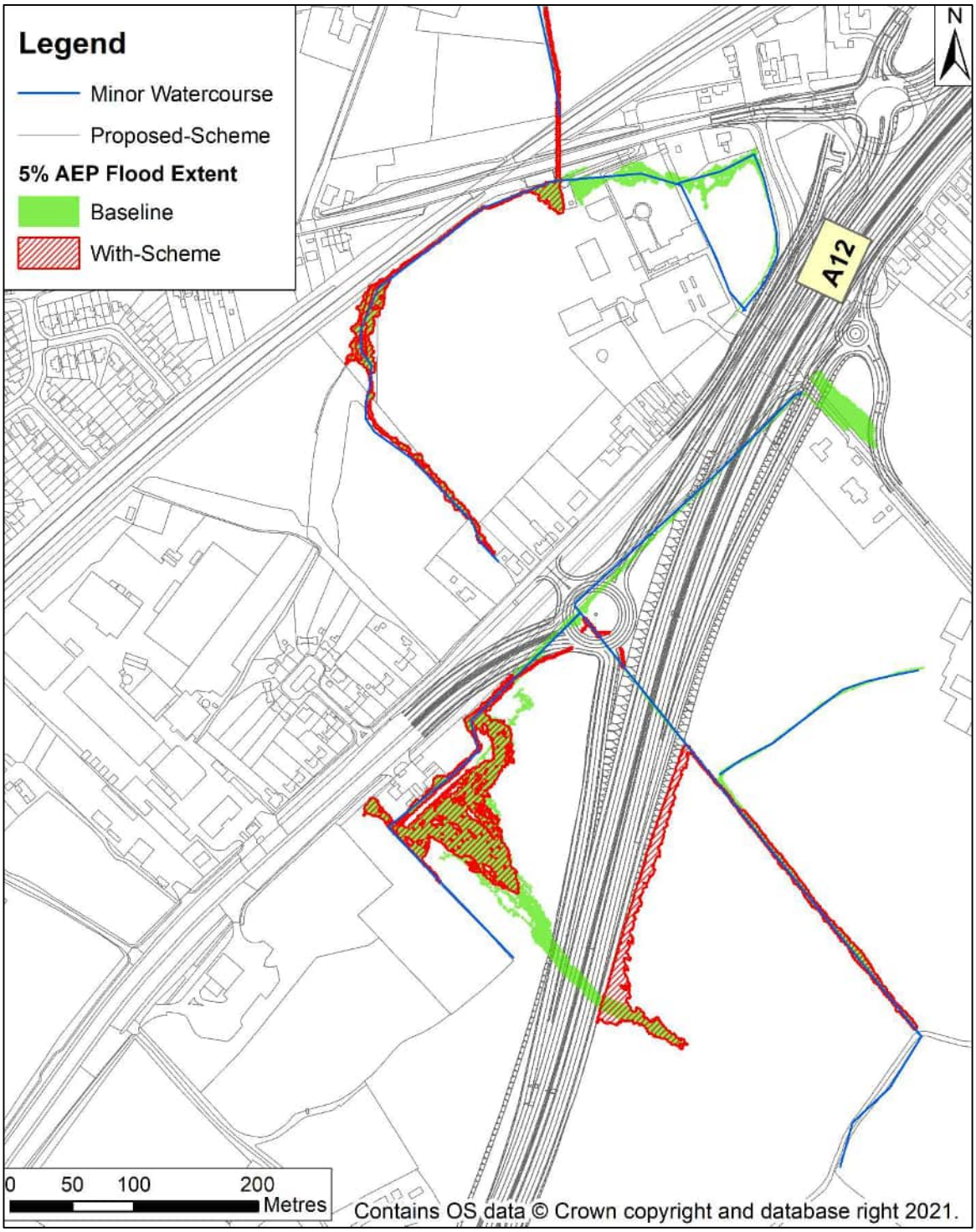


Plate C.2 1% AEP maximum flood extent for baseline and with-scheme scenarios

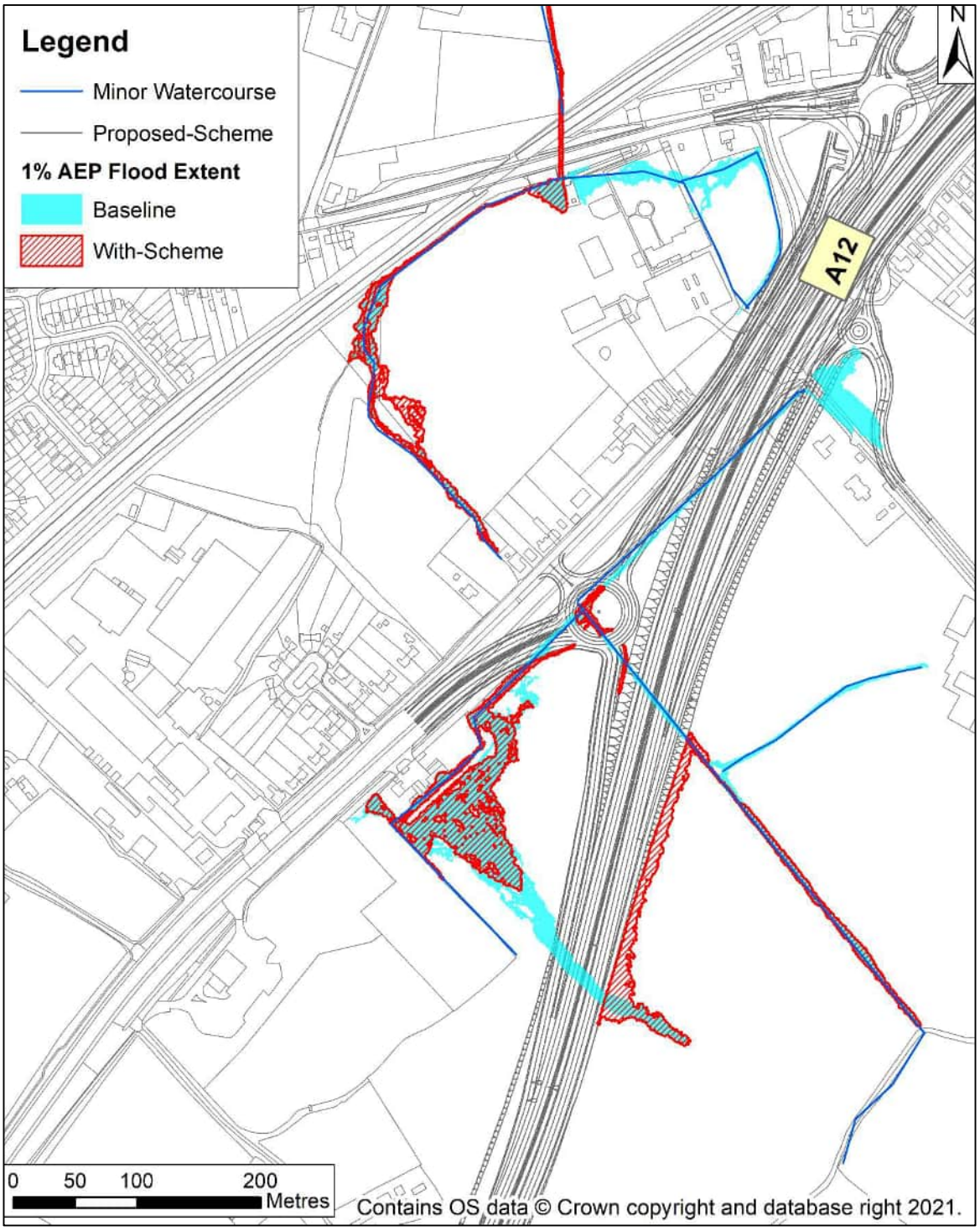


Plate C.3 1% AEP +40%CC maximum flood extent for baseline and with-scheme scenarios

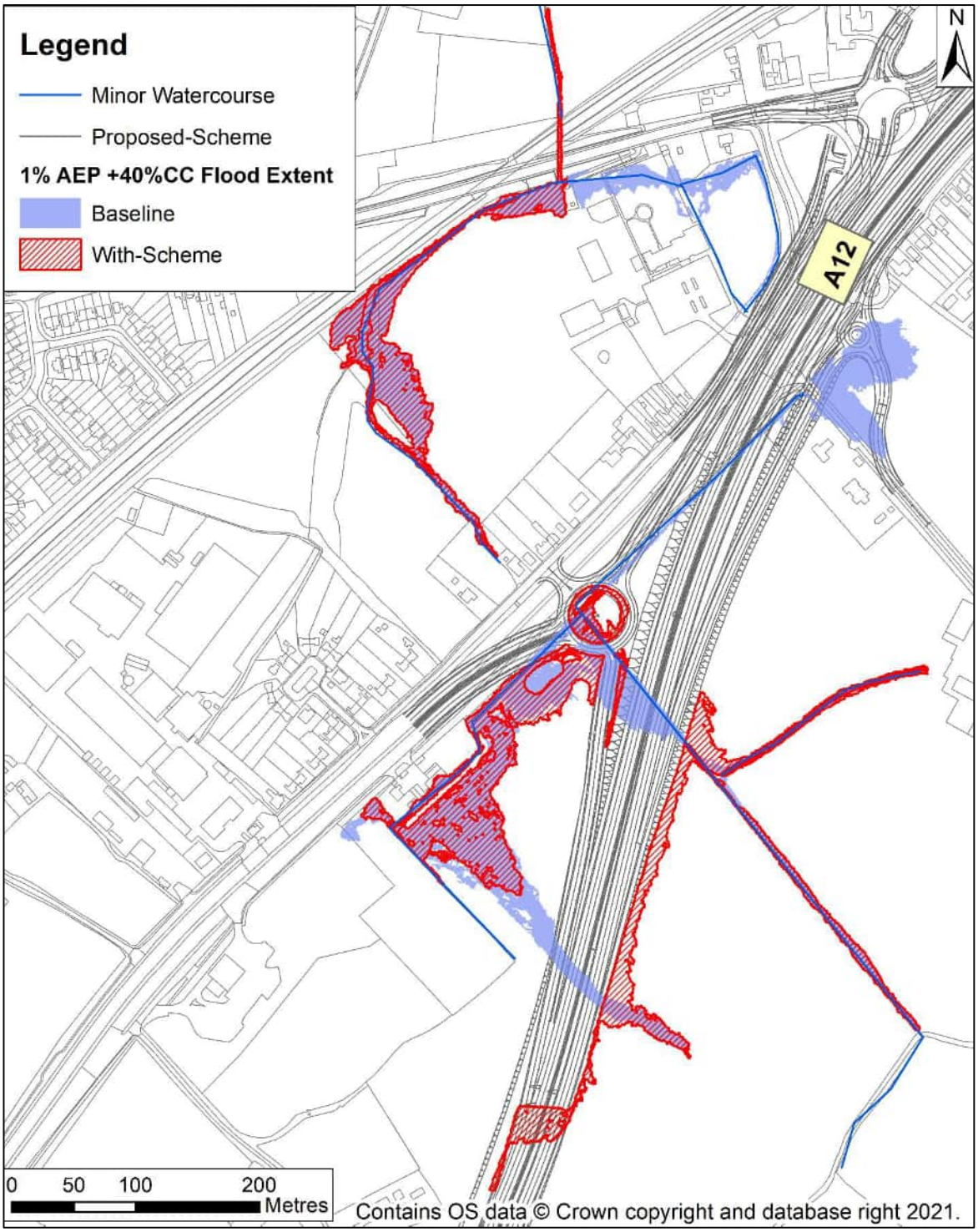


Plate C.4 Water level difference (with-scheme – baseline) for the 5% AEP event.

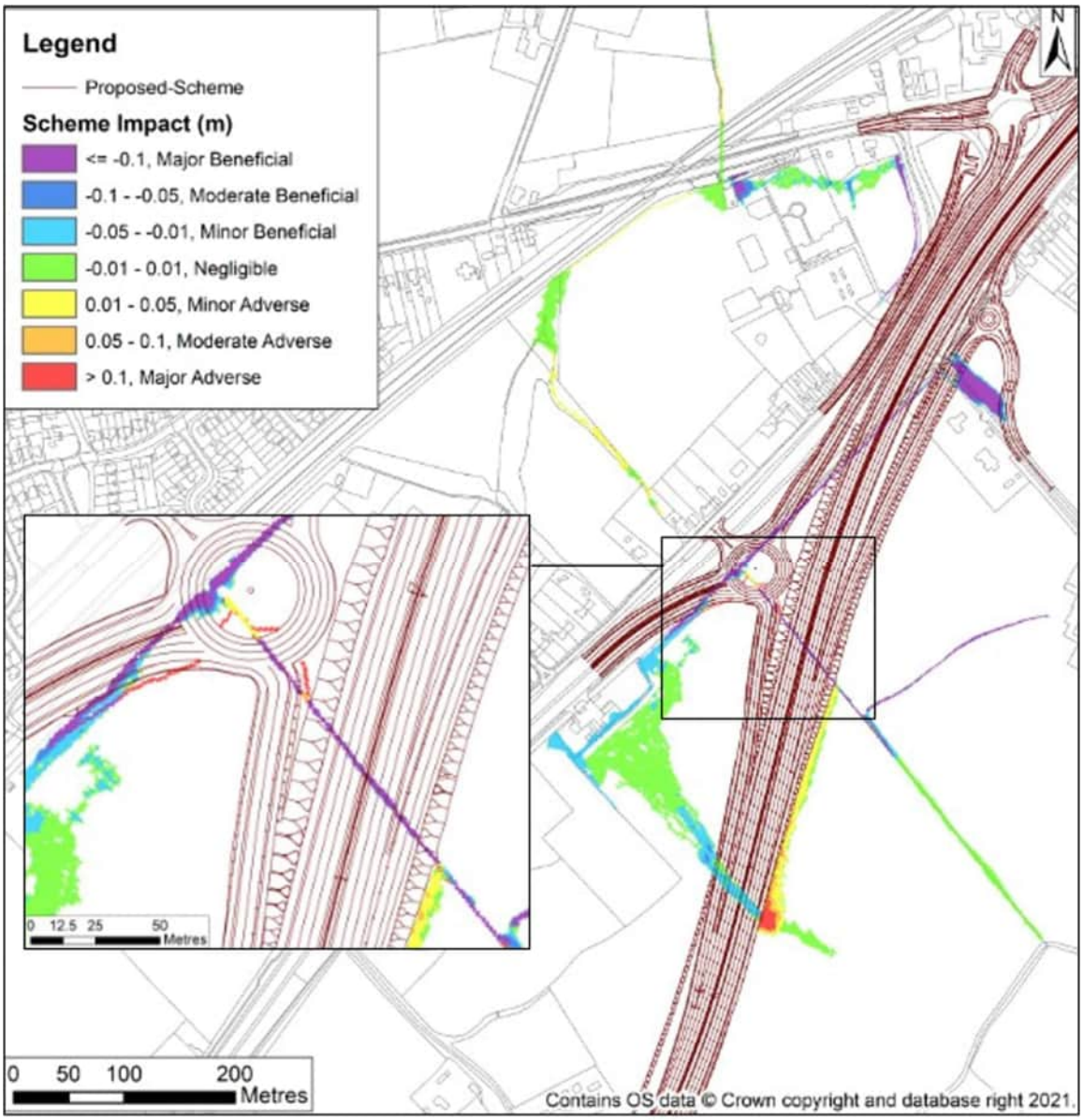


Plate C.5 Water level difference (with-scheme – baseline) for the 1% AEP event.

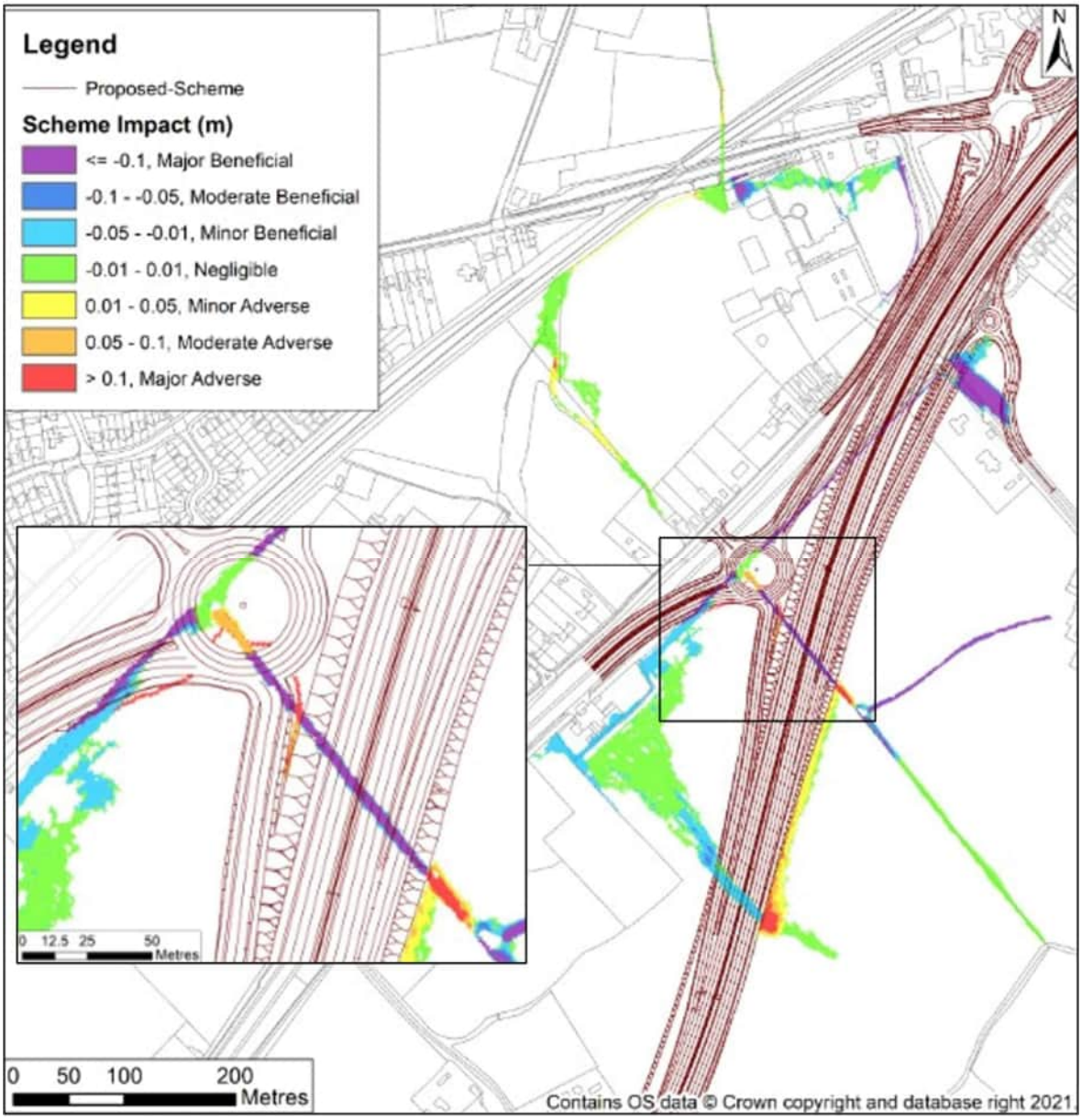
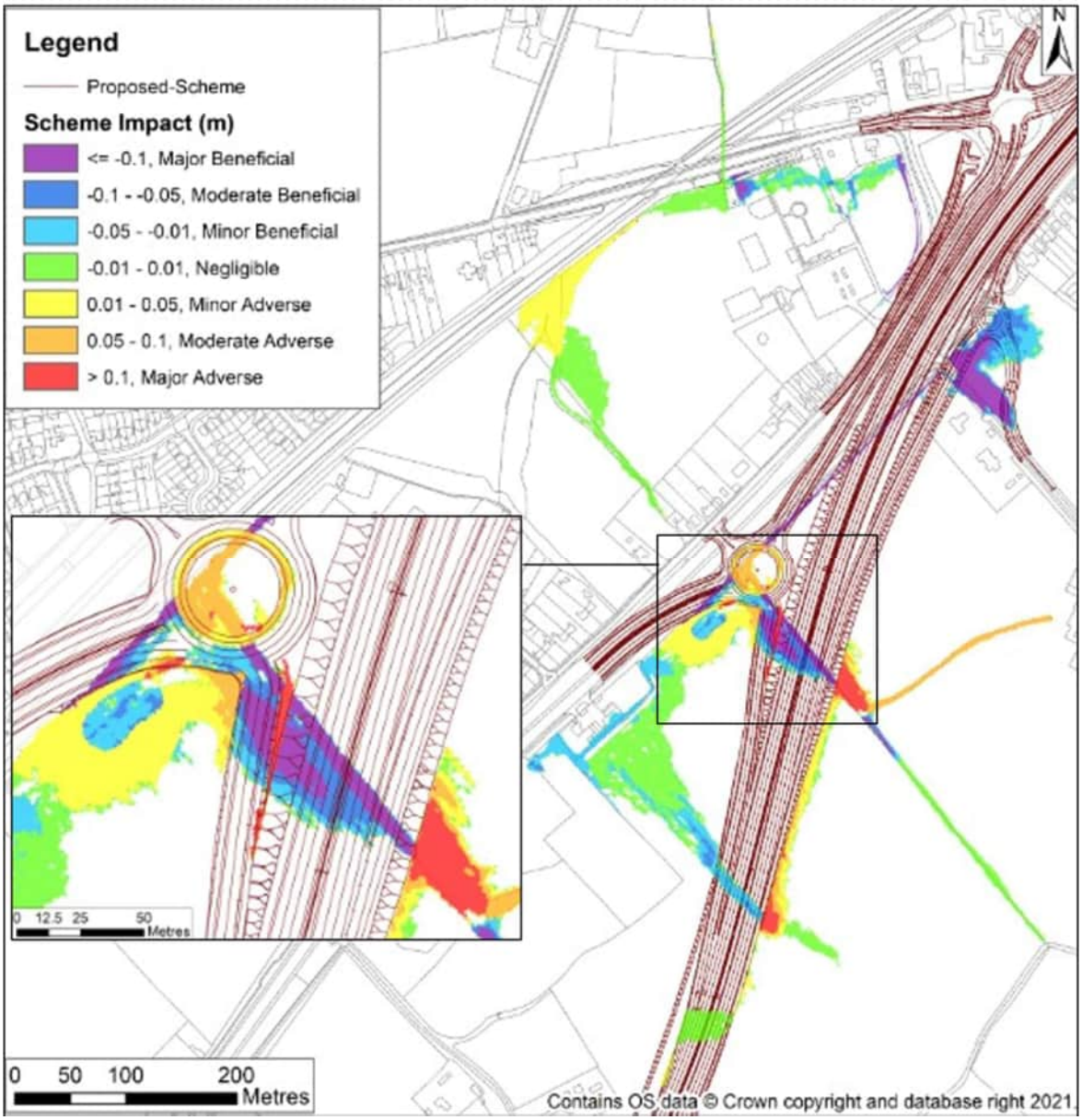


Plate C.6 Water level difference (with-scheme – baseline) for the 1% AEP +40%CC event.



Appendix D – Mitigation Flood Mapping

Plate D.1 5% AEP maximum flood extent for baseline and mitigation scenarios

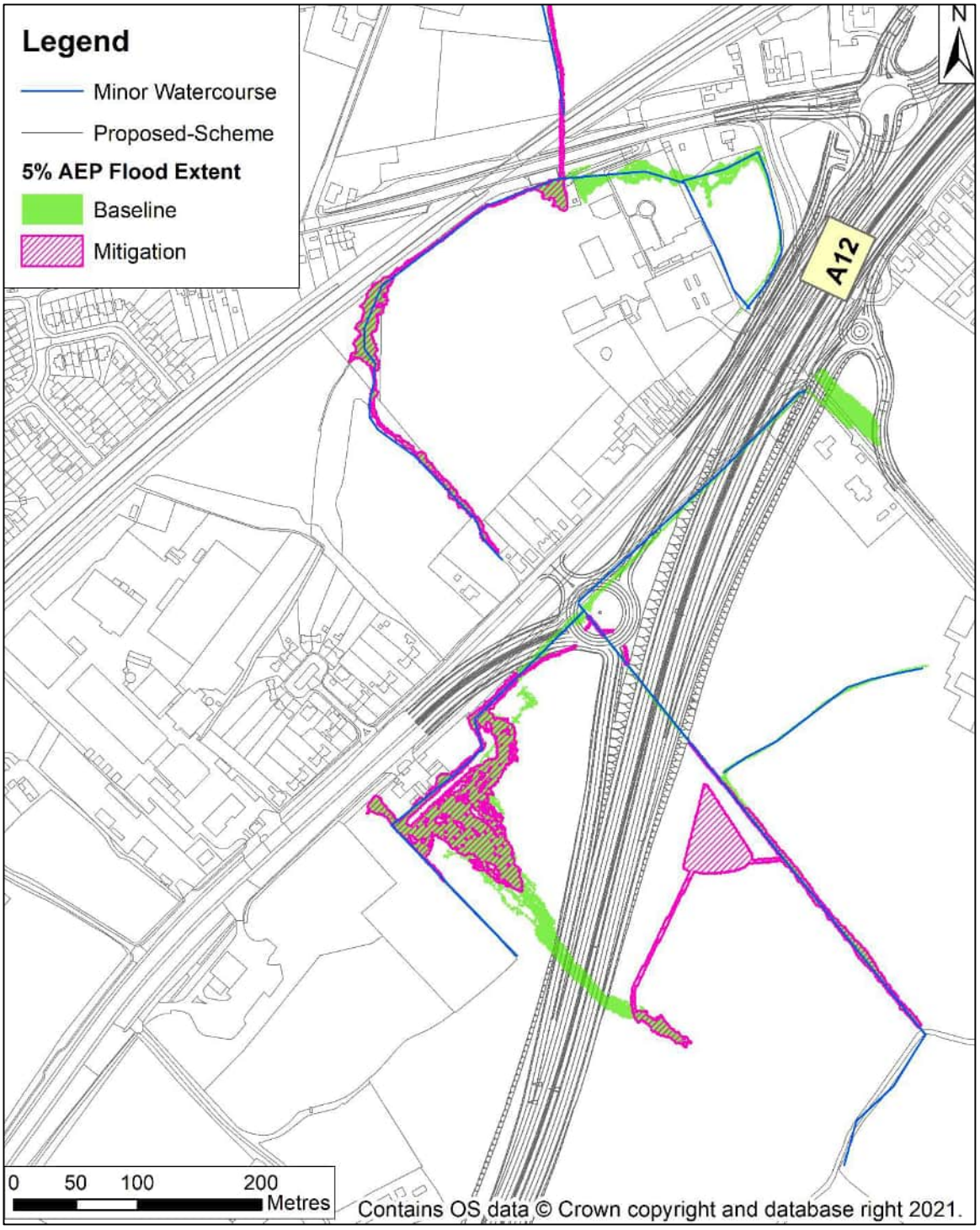


Plate D.2 1% AEP maximum flood extent for baseline and mitigation scenarios

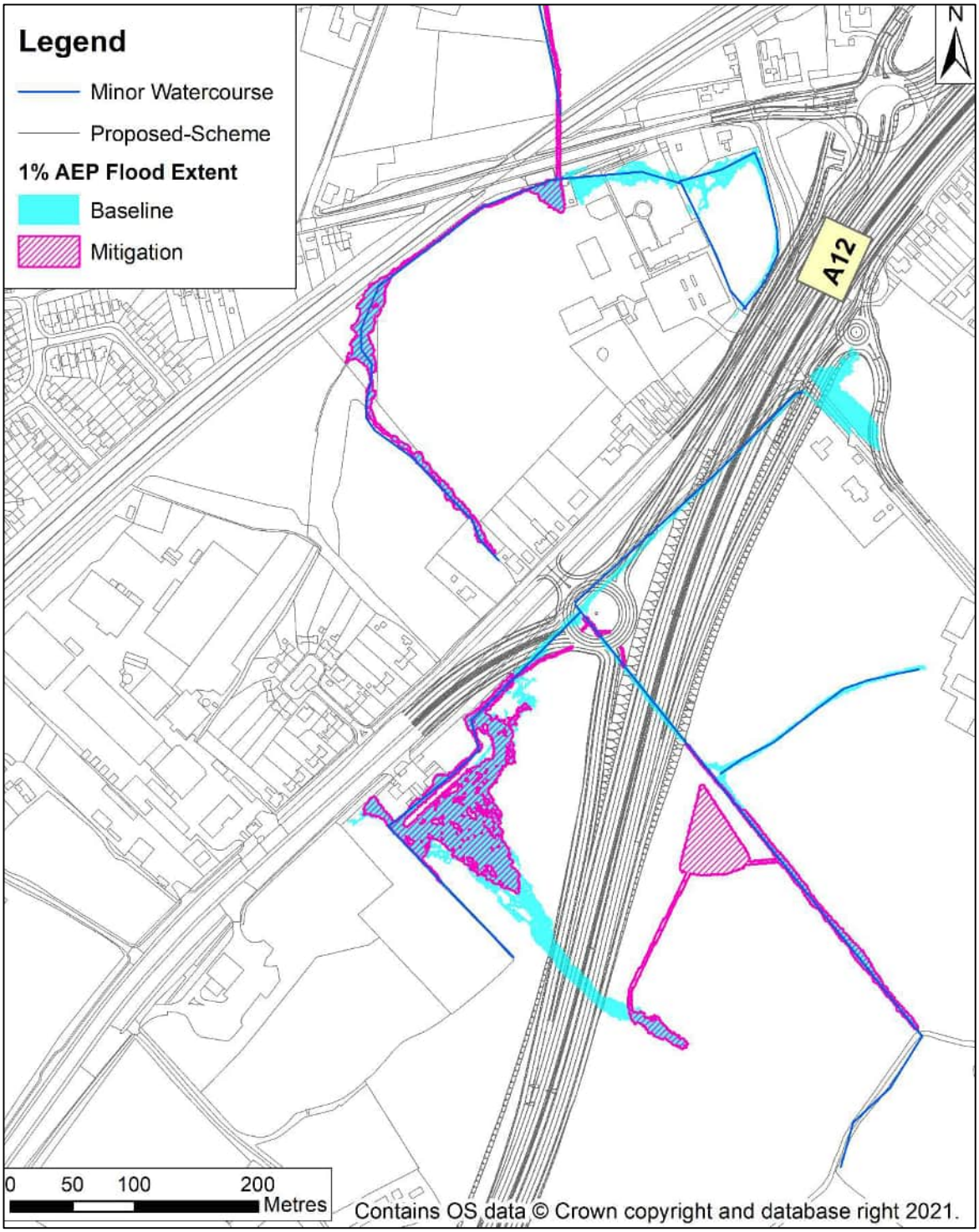


Plate D.3 1% AEP +40%CC maximum flood extent for baseline and mitigation scenarios

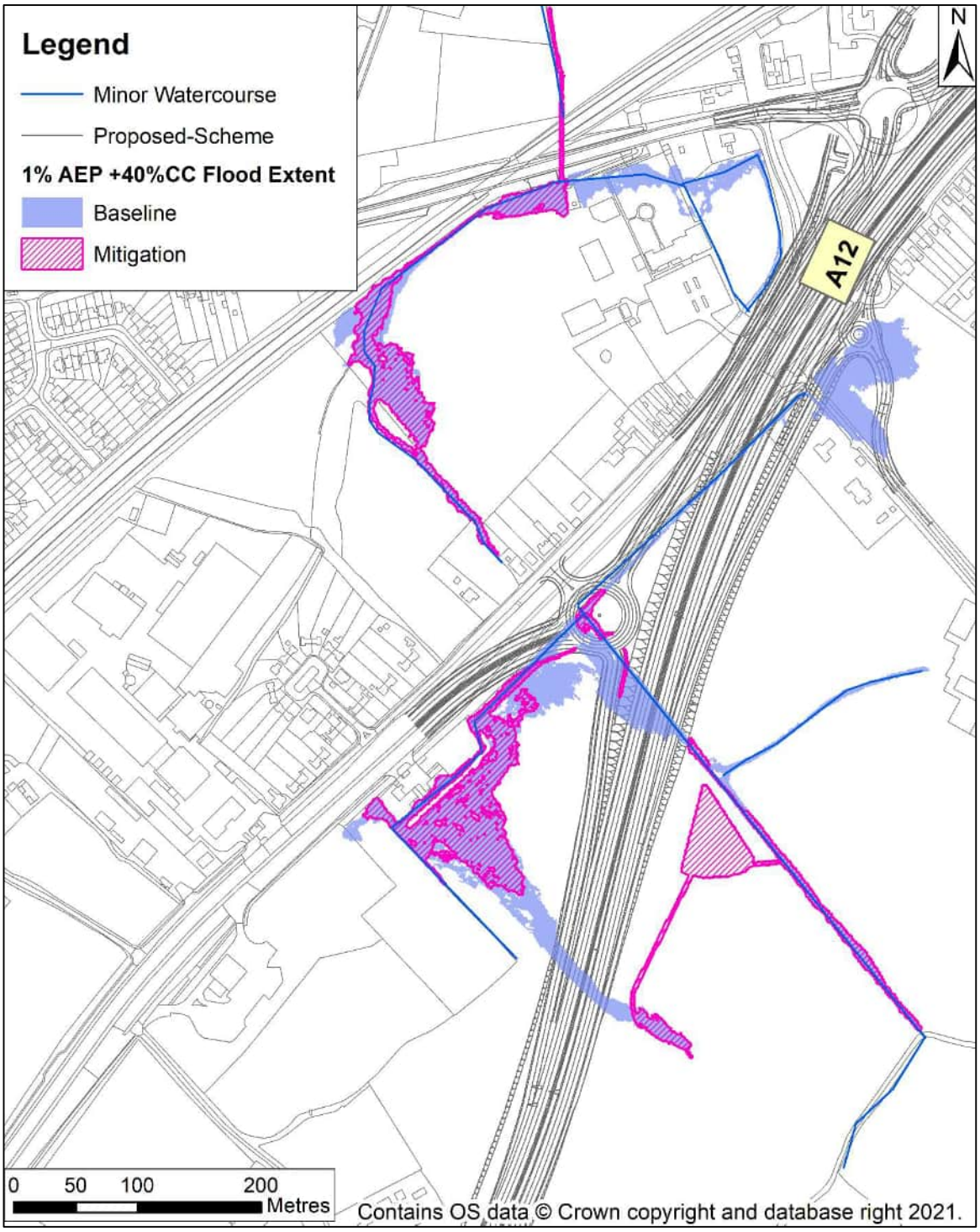


Plate D.4 Water level difference (mitigation - baseline) for the 5% AEP event

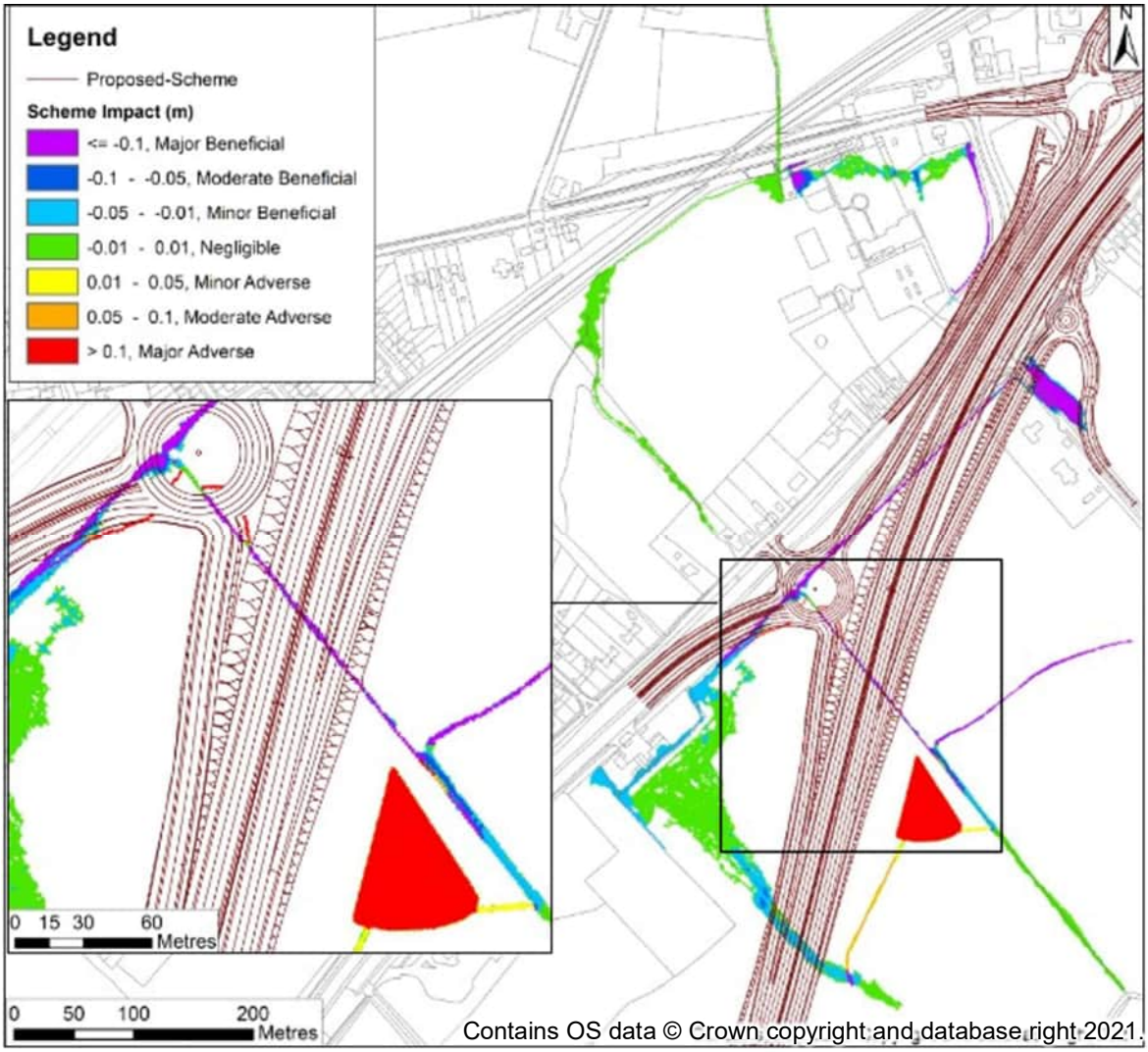


Plate D.5 Water level difference (mitigation - baseline) for the 1% AEP event

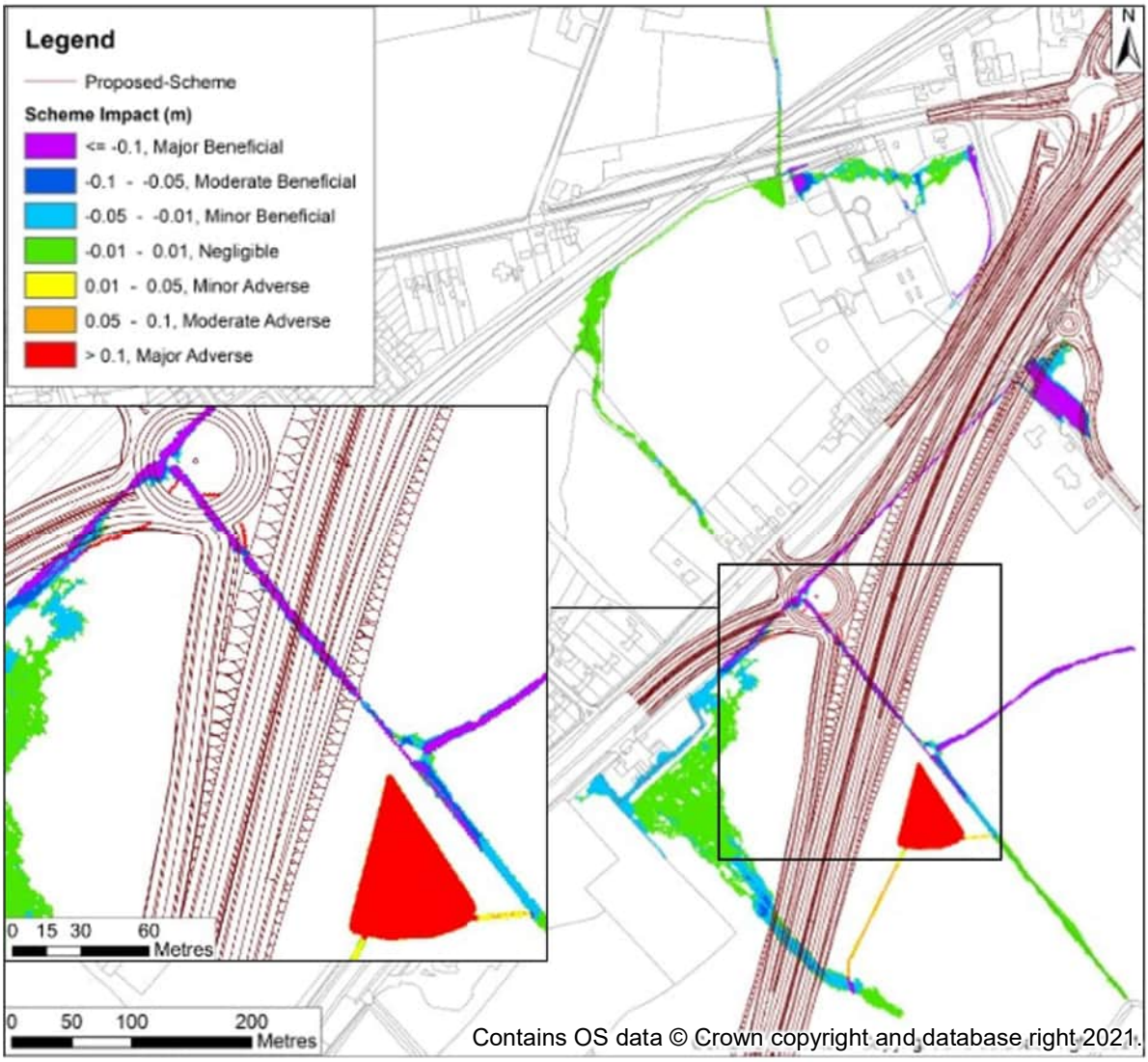
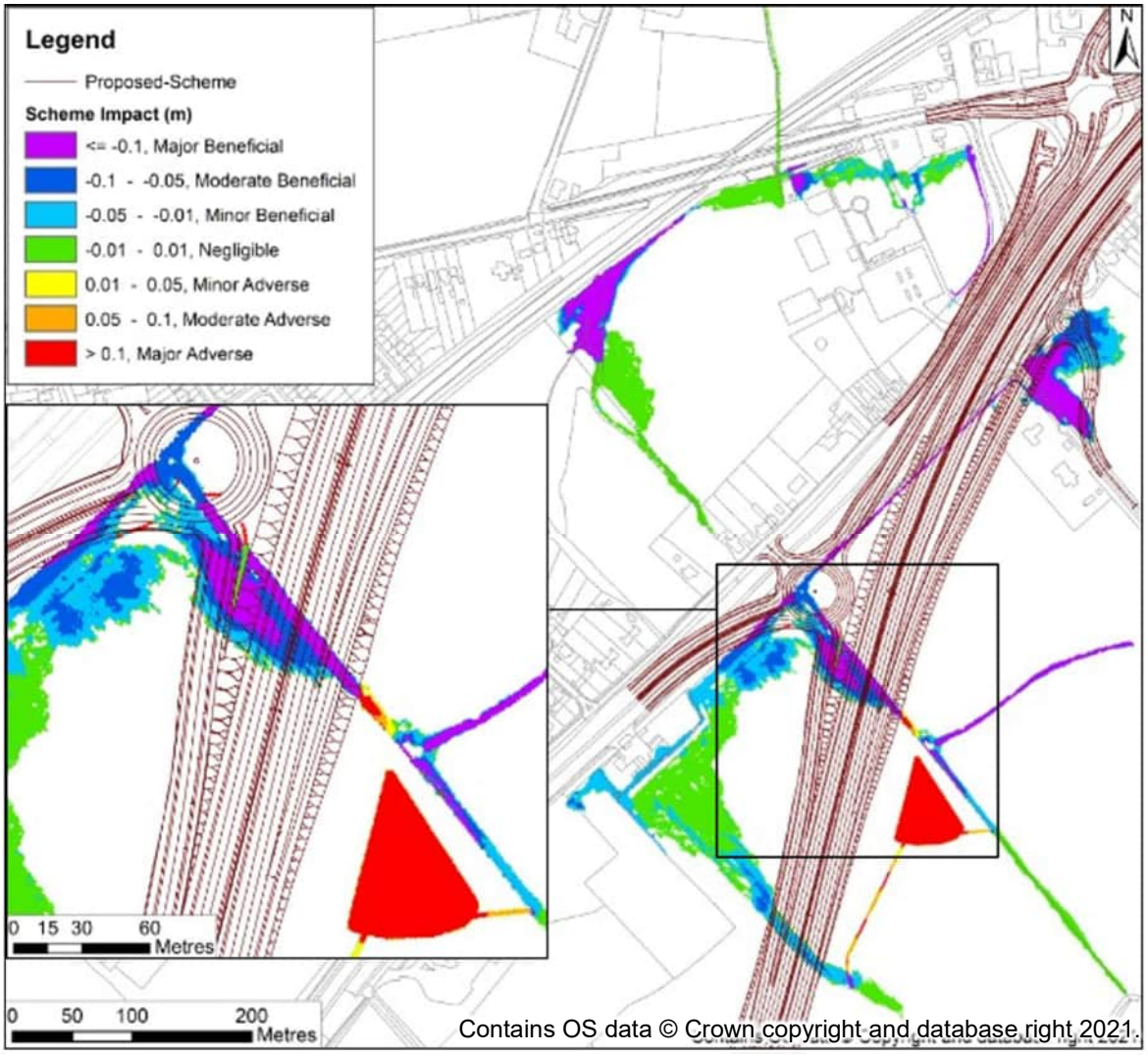


Plate D.6 Water level difference (mitigation - baseline) for the 1% AEP +40%CC event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 26 HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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1 Executive Summary

This document has been prepared to support the A12 Minor Watercourse 26 with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation for the Minor Watercourse 26 (MWC26) catchment which crosses the A12. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future.

3 Summary of assessment

3.1 Summary

3.1.1 A summary of the key information contained within the detailed hydrological assessment of MWC26 is provided in the table below.

Catchment location	MWC26 is located in South East England to the west of Colchester. MWC26 crosses the A12 at approximate co-ordinates 591200 223300 and discharges to the Roman River approximately 1.4km downstream of the A12 road crossing.
Purpose of study and scope	The A12 is proposed to be upgraded and therefore peak flows and inflow hydrographs are required for the numerical hydraulic modelling of this watercourse. Flows have been derived for the following Annual Exceedance Probability (AEP) events: 5%, 1% and 1% plus climate change (plus cc), equivalent to the 20-year, 100-year and 100-year plus cc return periods events respectively.
Key catchment features	The minor watercourse has generally a rural catchment with land use predominantly agricultural but there are some more urbanised parts of the catchment associated with the Marks Tey urban area. HOST Class 25 soils are present within some parts of the catchment.
Flooding mechanisms	Peak flows / volumes.
Gauged / ungauged	Ungauged.
Final choice of method	ReFH2.3 (flow from FEH Statistical method compared)
Key limitations / uncertainties in results	Small ungauged watercourse. Catchment contains HOST Class 25 soils and therefore BFIHOST values may be open to uncertainty.

3.2 Note of flood frequencies

3.2.1 The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period. Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval. The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Table 3.1 Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

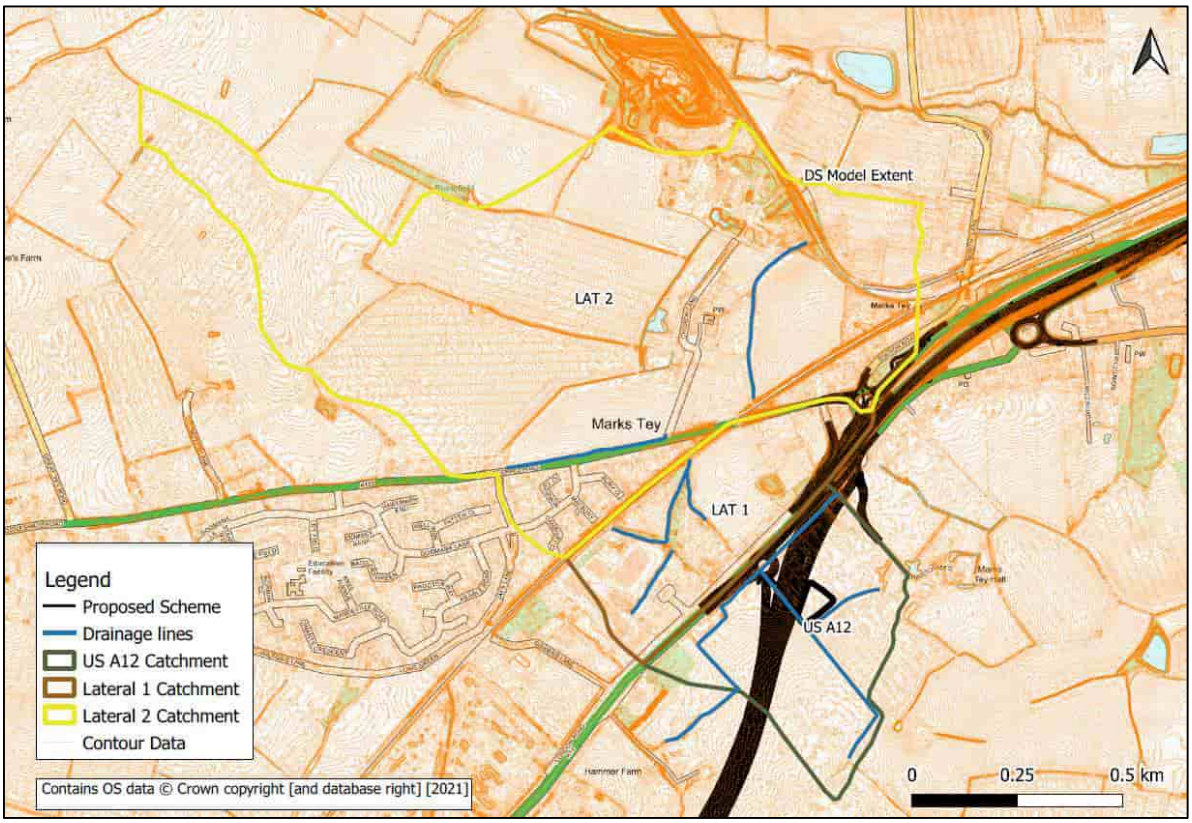
4 Method Statement

4.1 Requirements for flood estimates

Overview	The proposed A12 upgrade route crosses Minor Watercourse 26 (MWC26) and therefore peak flows and hydrographs are required for the hydraulic modelling of this watercourse. Design peak flows and hydrographs are required for the following AEP events: 5%, 1% and 1% plus climate change (plus cc). This is equivalent to the 20-year, 100-year and 100-year plus cc return periods). A climate change uplift factor of plus 40% will be applied for the 1% AEP (100-year return period) event. This is based on current Environment Agency guidance[1] (peak rainfall intensity allowance, upper end, 2080s scenario). MWC26 is a small and ungauged watercourse. The lumped catchment flows were derived at two locations, namely, at the A12 crossing and at the downstream modelling extent. However, for deriving inflow hydrographs required for the numerical hydraulic modelling the overall catchment has been divided into three sub-catchments, namely, one point inflow from the catchment up to the A12 road crossing, one lateral flow from the residual catchment between the A12 crossing and the railway line and one lateral flow from the residual catchment downstream of the railway to the downstream model extent.
Project scope	This is a simple hydrological study for one minor watercourse with a catchment area to the model extent of approximately 1.5km^2. Therefore, ReFH model parameter calibration for this ungauged catchment is outwith the scope of this project.

[1] Environment Agency (2016). Flood risk assessments: climate change allowances (updated 2021).

4.2 The catchment



Description	As shown in the above figure, Minor Watercourse 26 crosses the A12 to the east of Marks Tey. MWC26 then flows through some urban areas of Marks Tey, crossing the railway line and then flows north east before discharges to the Roman River downstream of this location. The catchment is classified as generally essentially rural but there are more urbanised areas of the catchment associated with Marks Tey. Land use in the US A12 and Lateral 2 (Lat 2) catchments is predominantly agricultural. The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of soil types 411d (Hanslope), 582d (Hornbeam 3) and 573b (Wix) in the lower reaches of the catchment. The HOST class of the soil groups were assessed and soil type 573b (Wix) was found to contain HOST class 25. There are noted issues within the Flood Estimation Guidelines[2] with BFIHOST values for soil

[2] Flood Estimation Guidelines, Technical guidance 197_08 (Environment Agency, June 2020)

	which contains HOST 23 and 25 which is considered to have been corrected for BFIHOST19. The geology of the catchment was assessed using the Geology of Britain Viewer[3] and found to be composed of predominantly London Clay formations consisting of clay, silt and sand.

4.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released 24th September 2020. This contains provisional data up to water year 2019-20.

[3] geology from the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

4.4 Gauging stations flow and level

Watercourse	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
Roman River	Roman River @ Bounstead Bridge		37021	52.6	Essex profile weir with crest trapping.	1965 to 1991
River Blackwater	Blackwater @ Appleford Bridge		37010	247.3	Double throated trapezoidal flume and high flow rated spillway.	1962 to present
River Ter	Ter @ Crabbs Bridge		37003	77.8	Theoretically rated	1932 to present

4.5 Data available at each flow gauging station in Table 4.4

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED ?	OK for pooling ?	Data quality check needed ?	Other comments on station and flow data quality
Roman River	1965 to 1991	N	Yes	No	No	Structure suffers from severe drowning. Groundwater influenced regime.
Blackwater @ Appleford Bridge	1962 to present	N	Yes	Yes	No	Groundwater influenced regime. Intermittently affected by the Ely / Ouse transfer scheme.
Ter @ Crabbs Bridge	1963 to present	N	Yes	No	No	Recently removed from suitable for pooling category. This was noted as due to many AMAX events being out of bank and this is not accounted for in rating.

4.6 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings	N/A			Not required for this assessment.
Historical flood data	N/A			Not Available
Flow or river level data for events	N/A			Not available for this assessment.
Rainfall data for events	N/A			Not required for this assessment.
Potential evaporation data	N/A			Not required for this assessment.
Results from previous	N/A			Not available for this

studies				assessment.
Other data or information	N/A			Not available for this assessment.

4.7 Hydrological understanding of catchment

Conceptual model	Hydraulic modelling of MWC26 is required to assess potential impacts of upgrading the A12. The target location is the A12 crossing of MWC26. Flooding is likely to be caused by peak flows/volumes at the A12 crossing location.
Unusual catchment features	The catchment is a small predominantly rural clay catchment located in South East England. There are noted issues with BFIHOST values for the soil (which contains HOST 25 soils) within parts of the catchment. The noted issues with BFIHOST are considered to have been corrected for BFIHOST19.

4.8 Initial choice of approach

Is FEH appropriate?	Yes, FEH methods are appropriate but care needs to be taken due to BFIHOST issue.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	ReFH2.3 and FEH Statistical methods have been applied to derive the peak flow estimates and hydrographs for this catchment. The hydrographs have been derived using ReFH2.3 method. Peak flow estimates have been derived for the catchment to the A12 crossing and at the downstream modelling extent. Two sets of lateral flows have been derived for the residual catchment areas between the A12 road and the railway line (Lateral 1) and between the railway line and the downstream model extent (Lateral 2).

Software to be used (with version numbers)	FEH Web Service[4] / WINFAP 4[5] / ReFH2.3

4.9 Locations where flood estimates required

4.9.1 The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

[4] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

[5] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

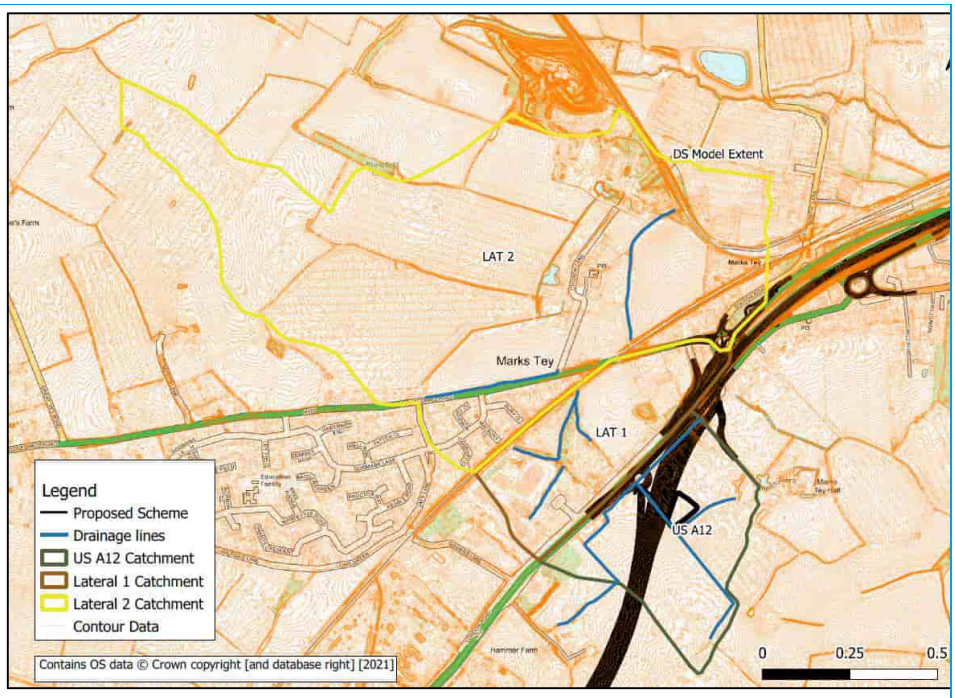
4.10 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Water-course	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
US A12	L	MWC4	Catchment upstream of the A12	591251	223287	N/A	0.22
Lat 1	S	MWC4	Catchment between the A12 and the railway line.	591092	223572	N/A	0.20
Lat 2	S	MWC4	Catchment between the railway line and the downstream model extent.	591356	224209	N/A	1.03
DS Model Extent	L	MWC4	Catchment to downstream model extent.	591356	224209	N/A	1.46

Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required.

Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced.

The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.



4.11 Important catchment descriptors at each subject site (incorporating any changes made)

4.11.1 The FEH Web Service does not pick up MWC26 catchment correctly. For deriving some of the FEH catchment descriptors such as PROPWET / SAAR / FPEXT the MWC26 catchment was picked up using the FEH Web service at 591250 223900. Other key FEH catchments descriptors such as DPLBAR / DPSBAR and URBEXT were calculated for the target locations using standard FEH methodologies.

Site code	FARL	PROPWET	BFIHOST	BFIHOST 19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
US A12	1	0.23	0.453	0.416	0.44	6.22	570	0.007	0.244
Lat 1	1	0.23	0.453	0.416	0.42	10.55	570	0.107	0.244
Lat 2	1	0.23	0.453	0.416	1.02	6.74	570	0.020	0.244
DS Model Extent	1	0.23	0.453	0.416	1.23	6.93	570	0.030	0.244

4.12 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	The FEH Web Service catchment boundary was assessed and found to be inaccurate. The catchment areas for this watercourse were therefore assessed using 0.1m contour data and LIDAR data.
Record how other catchment descriptors were checked and describe any changes.	Key catchment descriptors such as FARL and soils were checked using mapping data and key catchment descriptors were calculated using standard methods (e.g. area, DPSBAR, DPLBAR, URBEXT).
Source of URBEXT	URBEXT1990 and URBEXT2000 calculated using URBAN50K method.
Method for updating of URBEXT	URBEXT calculated using the topographic map and FEH method.

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	The FEH Statistical method was used as a comparison against ReFH2.3 peak flow estimates. The FEH Statistical method uses BFIHOST which has known issues in South East England on clay catchments with HOST Class 25 soils within the catchment.

5.2 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjustment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, (A/B)[a]	If more than one donor			
						Weight	Weighted ave. adjustment		
US A12	0.037	CDs	-	-	-	-	-	1.008	0.037
DS Model Extent	0.227	CDs	-	-	-	-	-	1.034	0.235

Are the values of QMED spatially consistent?	Yes
Method used for urban adjustment for subject and donor sites	

Parameters used for WINFAP v4 urban adjustment if applicable

Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010) in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016).

5.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Donor gauges were assessed for the purposes of QMED adjustment; however, no donor was selected. The Roman River @ Bounstead Bridge gauge (37021) was assessed as it is the closest gauge, however, the gauge was rejected as it is on a much larger catchment to the target and it is noted as having a groundwater influence regime. Runoff was also noted as influenced by groundwater abstraction and /or recharge and runoff impacted by both effluent returns and industrial and / or agricultural abstraction. This gauge was therefore assessed as not suitable as a donor for this small ungauged catchment. Both the River Blackwater @ Appleford Bridge and River Ter @ Crabbs Bridge gauges were also assessed as inappropriate donors and would have resulted in a decrease in QMED. Given no gauges were assessed as suitable donors QMED was not adjusted.

5.4 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)

5.5 Derivation of pooling groups

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
Final Pooling Group	Donor	No	Removed: 27073 / 26016 (high BFIHOST), 47022 / 106002 (low FARL) and 49005 (high discordance / short record /relatively high BFIHOST) Added: 27010 / 49003	L-CV: 0.215 L-skew: 0.270

Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).

5.6 Derivation of flood growth curves at subject sites

Site code	Method	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
Final Pooling Group	P	Final Pooling Group	GL: distribution gives an acceptable fit.	Urban Adjustment	Location: 1.000 Scale: 0.202 Shape: -0.279	2.886

Notes
Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis
Urban adjustments are all carried out using the method of Kjeldsen (2010).
Growth curves were derived using the procedures from Science Report SC050050 (2008).

5.7 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	25	50	100	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events									
	50	20	10	5	4	2	1	0.5	0.2	0.1
US A12	0.037	0.050	0.060	0.071	0.075	0.090	0.107	0.128	0.162	0.194
DS Model Extent	0.235	0.315	0.379	0.452	0.478	0.569	0.678	0.810	1.028	1.234

6 Revitalised flood hydrograph 2 (ReFH2) method

6.1 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 has been applied to derive both peak flow estimates and hydrographs for the target watercourse at key locations for input into a hydraulic model.

6.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp}	BL (hours)	BR
US A12	ReFH2.3	3.58	-	356.6	-	29.8	1.8
Lat 1	ReFH2.3	2.94	-	356.6	-	29.49	1.82
Lat 2	ReFH2.3	5.64	-	356.6	-	35.79	1.78
DS Model Extent	ReFH2.3	6.22	-	356.6	-	37.28	1.78
Brief description of any flood event analysis carried out							
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

6.3 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
US A12	Rural	Winter	5.5
DS Model Extent	Rural	Winter	9.0

6.4 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
Lat 1	Winter	4.5		Default
Lat 2	Winter	9.0		Default

6.5 Flood estimates from the ReFH2 method for individual critical storm duration

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + 40% CC	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC	0.5	0.2	0.1
US A12	0.063	0.086	0.101	0.117	0.122	0.140	0.163	0.236	0.197	0.269	0.331
Lat 1	0.067	0.092	0.109	0.126	0.132	0.151	0.175	0.253	0.211	0.285	0.352
Lat 2	0.225	0.300	0.351	0.403	0.420	0.482	0.561	0.810	0.683	0.928	1.13
DS A12	0.299	0.397	0.464	0.533	0.556	0.637	0.741	1.07	0.901	1.22	1.49

7 Discussion and summary of results

7.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak			
	Return period 20-years / 5% AEP		Return period 100-years / 1% AEP	
	FEH Statistical	ReFH2.3	FEH Statistical	ReFH2.3
US A12	0.071	0.117	0.107	0.163
DS A12	0.452	0.533	0.678	0.741

7.2 Final choice of method

Choice of method and reasons	The final choice of method is ReFH2.3 as it incorporates updates to BFIHOST19 that take into consideration the HOST Class 23 and 25 soils and also the FEH13 rainfall data. This is considered to be appropriate to adopt for this study.
How will the flows be applied to a hydraulic model?	The ReFH2.3 design hydrographs will be applied in the hydraulic model. The US A12 hydrographs will be applied at the start of the hydraulic model. The residual ReFH2.3 hydrographs will be applied as lateral flows within the hydraulic model. To assess the critical storm duration for MWC26 two model runs will be required. Run 1 will be undertaken using the critical storm duration (CSD), areal reduction factor (ARF) and seasonal correction factor (SCF) of the downstream model extent and Run 2 will be undertaken using the CSD, ARF and SCF for the target location the A12 crossing.

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are as follows: • ReFH2.3 is most appropriate for the target catchment as it uses BFIHOST19 and FEH13 rainfall data. • FEH Statistical is not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 25 soils. • Water is assumed not to back up from the Roman River during simulated flood events.
Discuss any particular limitations	None.
Provide information on the uncertainty in the design	The ReFH2.3 method has been used to derive design peak flows and hydrographs. This method is appropriate

peak flow estimates and the methodology used	but there is inherent uncertainty regarding using default model parameters. The standard approach in this situation is to use default model parameters. If the catchment was gauged the ReFH parameters may be adjusted to reduce the uncertainty.
Comment on the suitability of the results for future studies	The design peak flows have been estimated for the purposes of the A12 road improvements project only with the target location being the A12 crossing. Future studies should assess the appropriateness of these peak flow estimates / hydrographs for the purposes of their investigation.
Give any other comments on the study	

7.4 Checks

Are the results consistent, for example at confluences?	Flows are consistent.
What do the results imply regarding the return periods / frequency of floods during the period of record?	N/A ungauged catchment.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The 1% AEP growth factor at the A12 crossing is 2.582 using the ReFH2.3 method. The 1% AEP growth factor at the A12 crossing is 2.886 using the FEH Statistical method. These values are within the expected range.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the 0.1% AEP / 1% AEP is 2.029 using the ReFH2.3 method, the corresponding FEH Statistical estimates ratio is 1.820.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No other studies were available for this minor watercourse.
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	The peak flow estimates will be checked within the model at the downstream model extent for Run1.

7.5 Final results

7.5.1 In order to assess the critical storm duration for MWC26 two model runs will be required. Run 1 will be undertaken using the critical storm duration (CSD), areal reduction factor (ARF) and seasonal correction factor (SCF) of the downstream model extent for all inflows / residual flows and Run 2 will be undertaken using the CSD, ARF and SCF for the target location the A12 crossing.

Table 7.1 Run 1 Flood Peak Estimates using D/S Model Extent CSD (9.0hrs), ARF (0.98) and SCF (0.655)

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + 40% CC uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + 40% CC uplift	0.5	0.2	0.1
US A12	0.064	0.086	0.101	0.116	0.122	0.140	0.163	0.237	0.199	0.272	0.333
Lat 1	0.068	0.092	0.108	0.125	0.130	0.150	0.175	0.253	0.213	0.291	0.355
Lat 2	0.225	0.299	0.350	0.402	0.419	0.480	0.559	0.808	0.681	0.925	1.13
DS Model Extent*	0.299	0.397	0.464	0.533	0.556	0.637	0.741	1.07	0.901	1.22	1.49

*DS Model Extent peak flow estimates provided for Run 1 (i.e. CSD of 9.0 hours used to derive all inflows) only for flow reconciliation purposes.

Table 7.2 Run 2* Flood Peak Estimates using Target Location (US A12) CSD (5.5hrs), ARF (0.988) and SCF (0.644)

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + 40% CC uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + 40% CC uplift	0.5	0.2	0.1
US A12	0.063	0.086	0.101	0.117	0.122	0.140	0.163	0.236	0.197	0.269	0.331
Lat 1	0.070	0.096	0.113	0.130	0.136	0.157	0.182	0.263	0.220	0.299	0.368
Lat 2	0.212	0.286	0.336	0.387	0.404	0.463	0.536	0.771	0.647	0.876	1.08

*Flow derived using CSD of 5.5 hours for all sub-catchments.

7.6 Uncertainty bounds

7.6.1 The Flood Estimation Guidelines 197_08 (2020) notes that it is more difficult to quantify uncertainty in design flows estimated from the ReFH rainfall-runoff model than for the FEH Statistical method which it reports approximate uncertainty bounds for. It is therefore highlighted that the final flow estimates are open to uncertainty, but uncertainty bounds have not been provided for this assessment.

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hydrographs are required for the hydraulic modelling of MWC26. See: MWC26 ReFH2.3 Hydrographs 1

Annex A

A.1 Pooling Group Details

A.1.1 Initial Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
76011 (Coal Burn @ Coalburn)	2.474	42	1.84	0.163	0.301	0.621	2.474	1.63	1096	0.074	1	0
27073 (Brompton Beck @ Snainton Ings)	3.235	39	0.812	0.215	0.035	1.295	3.235	8.06	721	0.237	1	0.008
27051 (Crimple @ Burn Bridge)	4.198	47	4.524	0.218	0.156	0.217	4.198	8.17	855	0.013	1	0.006
45816 (Haddeo @ Upton)	4.198	26	3.456	0.3	0.406	0.947	4.198	6.81	1210	0.011	1	0.005
28033 (Dove @ Hollinsclough)	4.447	44	4.177	0.228	0.371	0.624	4.447	7.92	1346	0.007	1	0
26016 (Gypsey Race @ Kirby Grindalythe)	4.815	22	0.1	0.321	0.266	1.206	4.815	15.85	757	0.03	1	0
25019 (Leven @ Easby)	4.842	41	5.09	0.342	0.386	1.588	4.842	15.09	830	0.019	1	0.004
91802 (Allt Leachdach @ Intake)	4.892	34	6.35	0.153	0.257	0.674	4.892	6.54	2554	0.003	0.992	0
25003 (Trout Beck @ Moor House)	4.905	46	15.142	0.168	0.29	0.53	4.905	11.4	1905	0.041	1	0
47022 (Tory Brook @ Newnham Park)	4.951	25	6.176	0.257	0.191	0.56	4.951	13.43	1403	0.023	0.942	0.014
71003 (Croasdale Beck @ Croasdale Flume)	4.964	37	10.9	0.212	0.323	0.274	4.964	10.71	1882	0.016	1	0

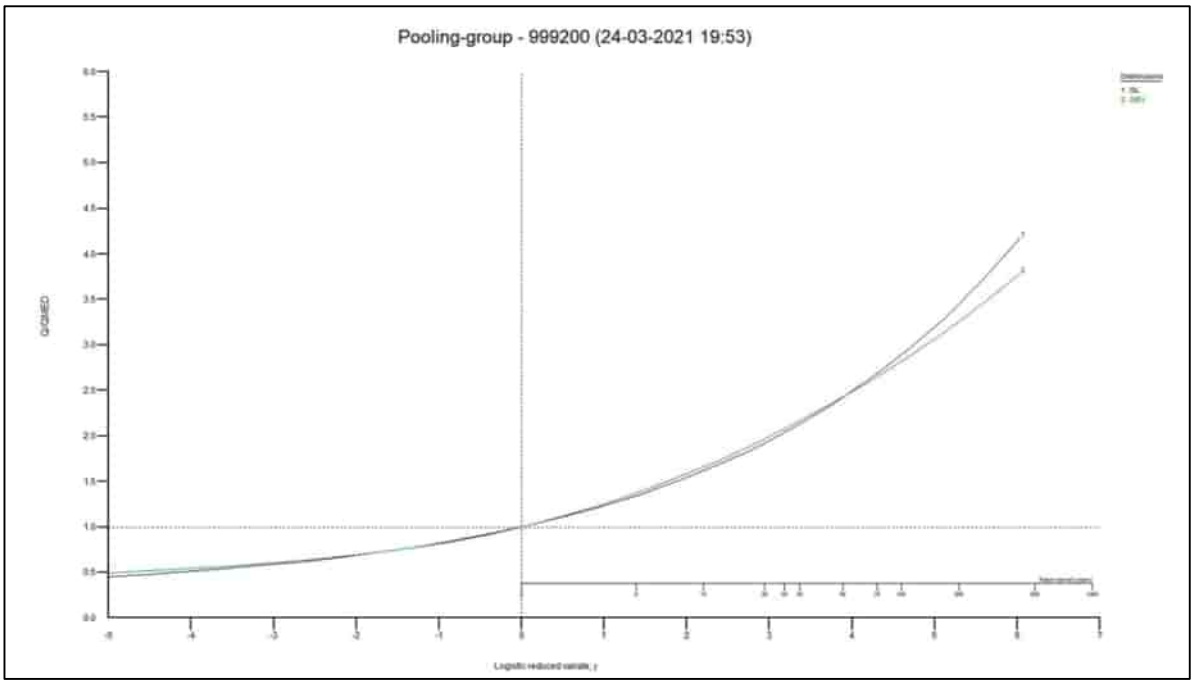
25011 (Langdon Beck @ Langdon)	4.974	33	15.647	0.232	0.328	0.703	4.974	12.79	1463	0.012	1	0.001
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	4.981	9	5.777	0.271	0.151	2.877	4.981	16.08	1044	0.023	0.991	0.006
106002 (Laxdale @ Laxdale)	5.018	12	17.449	0.103	0.132	1.16	5.018	10.64	1993	0.024	0.888	0
54022 (Severn @ Plynlimon Flume)	5.074	38	14.988	0.156	0.171	0.716	5.074	8.75	2481	0.01	1	0
206006 (Annalong @ Recorder)	5.16	48	15.33	0.189	0.052	2.008	5.16	14.44	1704	0.023	0.981	0
Total		543										
Weighted means				0.22	0.243							

A.1.2 Final Pooling Group

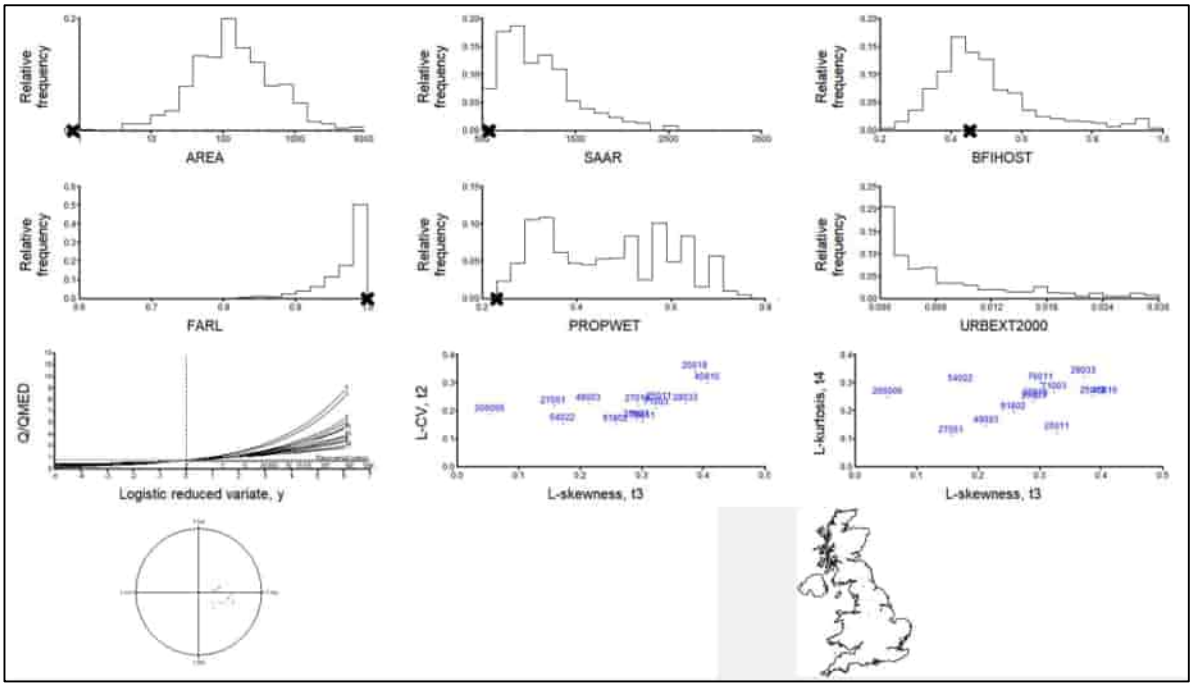
Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	Distance SDM	AREA	SAAR	FPEXT	FARL	URBEXT 2000
76011 (Coal Burn @ Coalburn)	2.474	42	1.84	0.163	0.301	0.833	2.474	1.63	1096	0.074	1	0
27051 (Crimple @ Burn Bridge)	4.198	47	4.524	0.218	0.156	1.359	4.198	8.17	855	0.013	1	0.006
45816 (Haddeo @ Upton)	4.198	26	3.456	0.3	0.406	0.968	4.198	6.81	1210	0.011	1	0.005
28033 (Dove @ Hollinsclough)	4.447	44	4.177	0.228	0.371	0.833	4.447	7.92	1346	0.007	1	0
25019 (Leven @ Easby)	4.842	41	5.09	0.342	0.386	1.996	4.842	15.09	830	0.019	1	0.004
91802 (Allt Leachdach @ Intake)	4.892	34	6.35	0.153	0.257	0.845	4.892	6.54	2554	0.003	0.992	0
25003 (Trout Beck @ Moor House)	4.905	46	15.142	0.168	0.29	0.529	4.905	11.4	1905	0.041	1	0
71003 (Croasdale Beck @ Croasdale Flume)	4.964	37	10.9	0.212	0.323	0.189	4.964	10.71	1882	0.016	1	0
25011 (Langdon Beck @ Langdon)	4.974	33	15.647	0.232	0.328	1.364	4.974	12.79	1463	0.012	1	0.001
54022 (Severn @ Plynlimon Flume)	5.074	38	14.988	0.156	0.171	0.984	5.074	8.75	2481	0.01	1	0
206006 (Annalong @ Recorder)	5.16	48	15.33	0.189	0.052	2.44	5.16	14.44	1704	0.023	0.981	0
27010 (Hodge Beck @ Bransdale Weir)	5.217	41	9.42	0.224	0.293	0.029	5.217	18.82	987	0.009	1	0.001

49003 (de Lank @ de Lank)	5.391	53	13.646	0.225	0.213	0.63	5.391	21.61	1628	0.064	0.998	0
Total		530										
Weighted means		1007		0.215	0.27							

A.2 Final Growth Curve



A.3 Diagnostic Plots



Acronyms

Abbreviation	Term
AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 23 MODELLING ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

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0 Executive summary

0.1.1 This report describes the hydraulic modelling and the results obtained, for the existing and proposed A12 highway crossing at Minor Watercourse 23 (MWC23), NGR grid reference TL878203. The modelling and hydrology analyses have been undertaken in line with the latest relevant industry standards, using the most up to date available data, including detailed topographic survey. The model is a 2-dimensional (2D) type using TUFLOW software (BMT 2022) and was built from scratch.

0.1.2 Three design events were simulated: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) Annual Exceedance Probability (AEP) plus 40% climate change (CC). No flow reconciliation was required to be carried out for the design hydrology. No data was available for calibration. The model was verified against the Environment Agency (EA) flood maps. The model was used to simulate baseline and with-scheme scenarios for the three design events mentioned. Sensitivity analysis of roughness and model flows were carried out to further validate the results.

0.1.3 The with-scheme scenario involved a new, offline highway alignment to be built to the south of the existing A12, as well as construction of a new junction and slip roads on the existing A12 to maintain local access routes. Channel realignment and new culverts were required to manage the fluvial drainage system under the proposed-scheme arrangement.

0.1.4 The baseline (existing) modelling scenario shows the A12 carriageway is free from flooding. The proposed-scheme impacts were deemed a significant change from the baseline scenario; therefore, mitigation works were required. The mitigation measures comprised a storage pond and ditch arrangement which facilitates flood attenuation and prevents any increase in flood depth or increase in pass forward flow from the scheme,.

0.1.5 The existing and proposed A12 remain free of flooding in all scenarios.

1 Introduction

1.1 Purpose of the report

1.1.1 The A12 Chelmsford to A120 Widening Scheme would improve the A12 between Junction 19 (Boreham) and Junction 25 (Marks Tay). The proposed-scheme includes widening of the A12 to three lanes throughout the length of the scheme plus associated works to junctions and side roads.

1.1.2 A Flood Risk Assessment (FRA) (National Highways, 2022) has been prepared as Appendix 14.5 to the Environmental Statement for the proposed-scheme to inform design development and support a Development Consent Order (DCO) application. Hydraulic modelling was required to support the FRA, which took the form of computational hydraulic modelling, including catchment hydrology. Modelled water levels for the baseline and with-scheme scenarios both upstream and downstream of the proposed-scheme and the associated flood extents were determined for a range of storm flood events.

1.1.3 The main river crossings for which hydraulic modelling was carried out to support the FRA (listed south to north) were:

- Boreham Brook
- River Ter
- River Brain and Lower Blackwater
- Rivenhall Brook
- Middle Blackwater
- Domsey Brook
- Roman River
- Minor Watercourses 7, 21, 21A, 23 (this report), 26

1.1.4 This report details the methodology and the results of the hydraulic modelling carried out to assess the baseline and with-scheme scenarios for an unnamed tributary of the River Domsey. The watercourse is designated as Minor Watercourse 23 (MWC23) based on the A12 drainage design numbering system for the main culvert crossings of the highway. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.1.5 For the A12 crossing of MWC23, a new offline highway will be built to the south of the existing A12, along with construction of a new junction and slip roads on the existing A12, a road bridge will be installed across the

new A12 highway in order to maintain local access routes. Modification to the fluvial drainage network require the realignment of the watercourse and the construction of culverts under the new roads. Full details of the with-scheme arrangements are given in Section 7.1.

1.1.6 This report should be read in conjunction with the FRA [R10060/APP/6.6] (National Highways 2022).

1.2 Study area

1.2.1 MWC23 is located in South East England, just outside Feering (see Plate 1.1), upstream branches of the watercourse crosses the current A12 at two locations. The watercourse flows generally in a south-easterly direction before it discharges into Domsey Brook, approximately 1.1km downstream of the existing A12 crossing. The watercourse features north and west upstream branches that converge around Prested Hall. The north tributary was not modelled explicitly as it is a smaller tributary. Instead, it was included as a point inflow in this model located downstream of the A12 crossing.

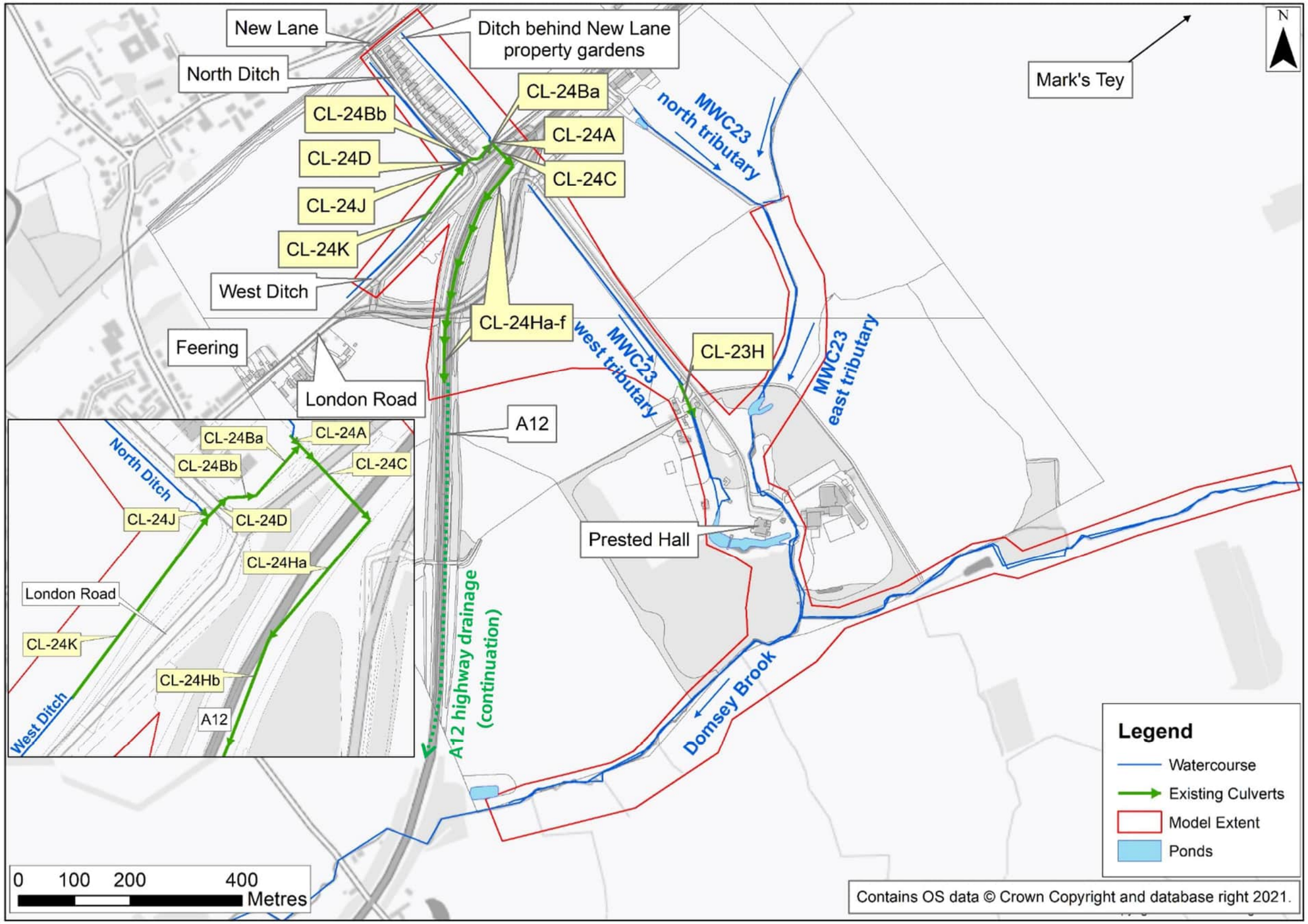
Historically the west tributary has been diverted into the A12 highway carrier drain, via a culvert network. There are numerous changes of pipe size and direction, so an explicit labelling system was utilised in the baseline model. On the north side of the A12 CL-24K conveys flow from the “West Ditch” (which is a toe drain of London Road) to a confluence with the “North Ditch” and CL-24J. CL-24J then carries the combined flow into CL-23D. Culvert CL-23D crosses under New lane and flows into two pipe sections in series which are designated CL-24Ba and CL-24Bb, this pipeline outfalls into CL-24C near its upstream end.

CL-24C conveys flows the combined flows from the north of the existing A12 into the existing A12 main carrier drain (labelled CL-23Ha-g). The continuation A12 drainage follows the alignment of the A12 for approximately 500m beyond the model extent before discharging into Domsey Brook.

The remaining downstream leg of the MWC23 West Tributary therefore drains just the area to the south of the existing A12, it converges with MWC23 East Tributary at Prested Hall before discharging into Domsey Brook. CL-23H is an access road culvert that conveys flow north to south under properties near Prested Hall.

1.2.2 The model extent covers an area of 0.33km^2 and includes a total channel reach length of approximately 1300m.

Plate 1.1 MWC23 study area



2 Input data

2.1.1 The data used to construct the hydraulic model for MWC23 is summarised in Table 2.1.

Table 2.1 MWC23 hydraulic model inputs

Data	Description	Source
LiDAR	1m resolution composite Digital Terrain Model (DTM) from LiDAR (Light Detection And Ranging) 2020 data. Used to inform the hydraulic model with ground level information.	Department for Environment, Food & Rural Affairs (DEFRA), 2020
Ordnance Survey (OS) Mastermap	Land use data used to specify roughness values across the 2D floodplain.	National Highways
A12 Drainage Network Data	Data regarding Culvert CL-24Ha-g	National Highways, 2021
As-built survey	Survey of culverts CL-24C	MKSurveys, 2019
Site Visit Data	Ground-truthing findings on CL-23H	Jacobs, 2021
Topographic Survey	Survey of the topography adjacent to the A12	Costain, 2017-2021
Outline Design Drawings	Drawings of the scheme design	National Highways, 2021
Flood Zone Mapping	EA Risk of Flooding Surface Water (RoFSW) flood maps. Used for model verification	Environment Agency

3 Hydrology

3.1 Introduction

3.1.1 Design peak flows and hydrographs are required for input into the numerical hydraulic model of MWC23 for the 5% (1 in 20) and 1% (1 in 100) AEP events. The 1% (1 in 100) AEP event is also required including an allowance for climate change (+CC). For full details of the flow derivation methods and background hydrology refer to Appendix A Minor Watercourse 23 Hydrology Annex (HE551497-JAC-EWE-SCHW-RP-LE-0092).

3.2 Hydrological considerations

3.2.1 MWC23 has a small catchment area of approximately 3.36km^2 up to the downstream model extent. The catchment is classified as essentially rural with a few dwellings present throughout the catchment and a series of local roads and the A12 cutting across the catchment. Land use is predominantly agricultural. There is also a solar farm to the west of Catchment 2 (see Plate 3.1).

3.2.2 MWC23 has two main branches namely, Catchment 1 (to the east) and Catchment 2 (to the west). These tributaries combine downstream of the A12 crossing, just upstream of the watercourse's confluence with the Domsey Brook. Catchment 3 and Catchment A also flow into MWC23 upstream of the Domsey Brook confluence (refer to Plate 3.1).

3.2.3 The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this assessment to be composed of predominantly soil type 411d (Hanslope) with a small area of 571s (Efford 1) soils to the western side of the catchment. The soils within the catchment were also assessed using Soilscapes (Landis 2021), which indicates that the soils are predominantly lime-rich loamy and clayey soils with impeded drainage with a small area to the west of the catchment being composed of freely draining slightly acid loamy soils.

3.2.4 According to the Geology of Britain Viewer (British Geological Survey 2021) the geology of the catchment is composed of London Clay Formations consisting of clay, silt and sand. The superficial deposits were noted as being composed of Lowestoft Formation (Diamicton).

3.3 Peak flow derivation

3.3.1 MWC23 is a small ungauged catchment and therefore flow estimation is open to great uncertainty. Both ReFH2.3 and FEH Statistical methods were therefore applied to estimate design peak flows for this watercourse. ReFH2.3 resulted in higher design peak flow estimates for MWC23 than the FEH Statistical method. Given ReFH2.3 uses updated soil and FEH13 rainfall data, the ReFH2.3 method was selected for final peak flows estimates.

3.4 Hydraulic model inflows

3.4.1 For the purposes of hydraulic modelling the overall MWC23 catchment was split into four catchments as following: Catchment 1 (eastern tributary), Catchment 2 (western tributary), Catchment 3, and Catchment A. The inflow within Catchment 2 was then further sub-divided (area scaled) within the hydraulic model so that inflows from the upper sub-catchments could be applied as point inflows at the start of the modelled reaches and the inflows from lower residual catchments could be applied as lateral inflows downstream of these point inflow locations. A schematisation of the MWC23 catchment is presented in Plate 3.1.

3.4.2 For Catchment 1, the total inflow was applied as a point inflow at the start of the model (Catch 1).

3.4.3 For Catchment 2, 31.9% of the total inflow was applied as a point inflow at the start of the model upstream of the railway (Catch 2 US RL), 12.9% was applied as a lateral inflow between the railway and the current A12 (Catch 2 Lat 1), 5.3% was applied as a lateral for the sub-catchment north west of New Lane (Catch 2 Lat 1a) and 5.3% was applied as a lateral for the sub-catchment south west of New Lane (Catch 2 Lat 1b), 5.4% was applied as a point inflow to represent the catchment area between the A12 and Catchment 2 Lateral 3 (Catch 2 Lat 2a) and 8.2% was applied as a point inflow to represent the catchment between Catchment 2 Lateral 2 and Catchment 3 (Catch 2 Lat 2b). Division of Catchment 2 Lateral 2 into 2a and 2b was carried out because of where the proposed-scheme would be located. Catchment 2 Lateral 3 was also divided into 2 sub-catchments - Catch 2 Lat 3a (6.5%) and Catch 2 Lat 3b (9.7%) for the same reason. It should be noted that the total inflows applied to Catchment 2 is only 85.2% (that is, 31.9+12.9+5.3+5.3+5.4+8.2+6.5+9.7) of the total flow. This is because, following the hydrological analysis, the road drainage team confirmed that an area in the upper catchment is currently redirected towards the Domsey Brook and therefore 14.8% of the total inflows is discharging out of the MWC23 catchment (see the excluded area in Plate 3.1). The Catchment 2 Lateral 3 area upstream of the A12 was therefore excluded from the modelled inflows.

3.4.4 For Catchment 3 and Catchment A the flows were applied directly within the model as point inflows.

3.4.5 The design peak flow estimates for the MWC23 catchments (Catchment 1, Catchment 2, Catchment 3 and Catchment A) for the individual catchment critical storm durations (CSDs), areal reduction factors (ARFs) and seasonal correction factors (SCFs) are presented in Table 3.1.

Plate 3.1 Schematisation of the MWC23 catchment

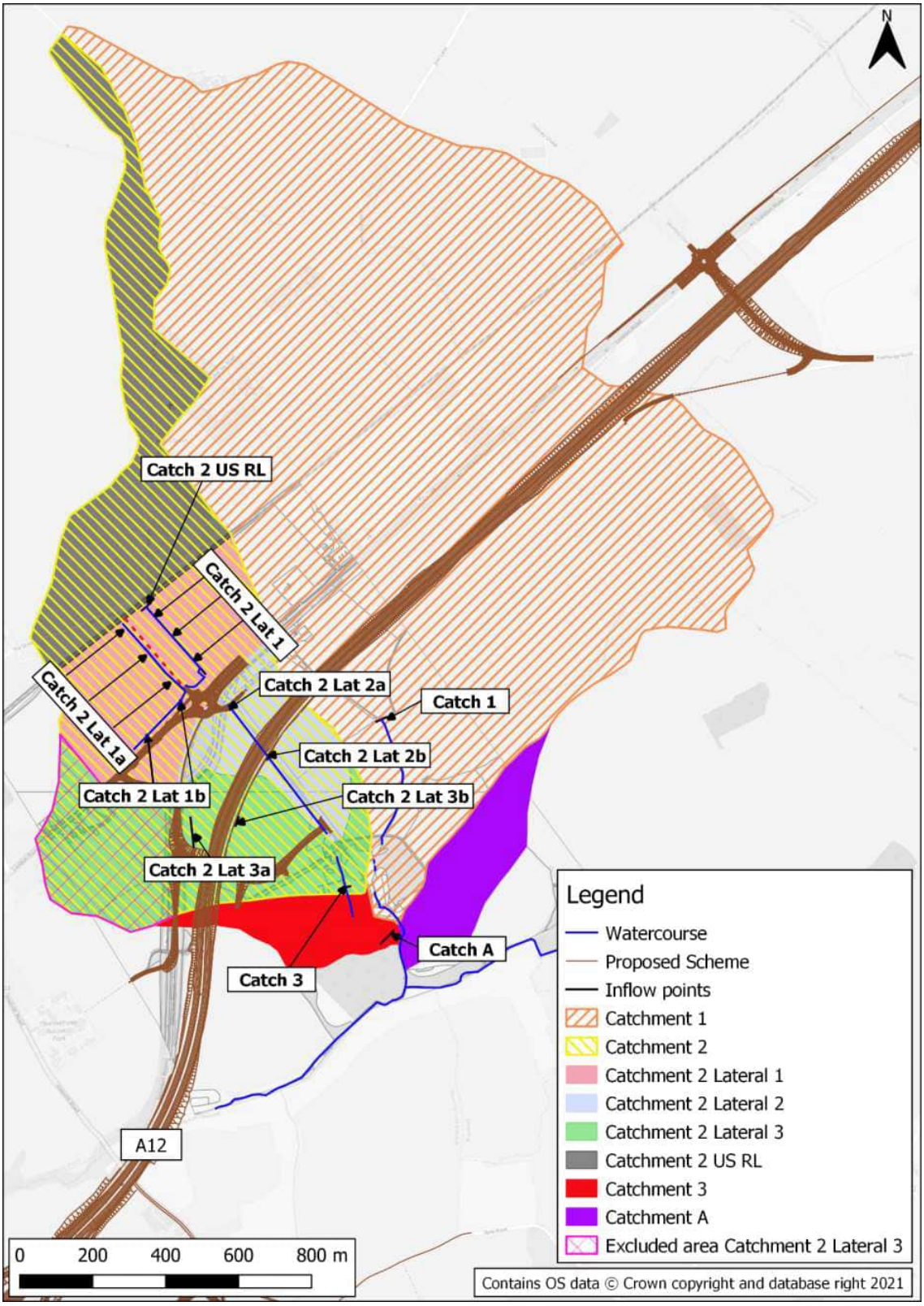


Table 3.1 MWC23 peak flow estimates (m^3/s) for the target catchments (Catchment 1, Catchment 2, Catchment 3 and Catchment A)

Inflow	Peak Flow* (m^3/s) for the following AEP events									
	50% (1 in 2)	20% (1 in 5)	10% (1 in 10)	5% (1 in 20)	4% (1 in 25)	2% (1 in 50)	1% (1 in 100)	1% (1 in 100) +CC 40% uplift	0.5% (1 in 200)	0.1% (1 in 1000)
Catchment 1	0.55	0.74	0.86	0.99	1.03	1.18	1.37	1.97	1.67	2.73
Catchment 2	0.27	0.37	0.44	0.50	0.53	0.60	0.70	1.01	0.84	1.40
Catchment 3	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.15	0.12	0.20
Catchment A	0.04	0.06	0.07	0.09	0.09	0.10	0.12	0.17	0.14	0.23

*Peak flows derived using individual catchment critical storm duration, areal reduction factor and seasonal correction factor.

3.5 Model runs and critical storm duration

Introduction

3.5.1 To assess the critical storm duration for MWC23, the following two model runs were required:

- Run 1: inflow for all (sub)catchments derived using the critical storm duration (CSD), areal reduction factor (ARF) and seasonal correction factor (SCF) as that of Catchment 2; and
- Run 2: inflows for all (sub)catchments derived using the individual catchment CSD, ARF and SCF.

3.5.2 Further details on the hydrology of the model runs and associated peak flow estimates are provided below.

Run 1

For Run 1 all four catchment inflows were derived using the Catchment 2 CSD (7.5hrs), ARF (0.982) and SCF (0.652). This run resulted in the following peak flow estimates for the modelled events.

Table 3.2 Run 1 peak flow estimates (CSD = 7.5 hours)

Inflows	Peak flows (m^3/s)		
	5% (1 in 20) AEP	1% (1 in 100) AEP	1% (1 in 100) AEP +40%CC uplift
Catchment 1	0.99	1.36	1.96
Catchment 2	0.50	0.70	1.01
Catchment 3	0.07	0.10	0.15
Catchment A	0.09	0.12	0.18

Run 2

3.5.3 For Run 2 all inflows were derived using the individual catchment CSD, ARF and SCF. This run resulted in the following peak flow estimates for the modelled events.

Table 3.3 Run 2 peak flow estimates (individual catchment CSD / ARF /SCF)

Inflows	Peak flows (m^3/s)		
	5% (1 in 20) AEP	1% (1 in 100) AEP	1% (1 in 100) AEP +40%CC uplift
Catchment 1 (CSD = 11.0hrs)	0.99	1.37	1.97
Catchment 2 (CSD = 7.5hrs)	0.50	0.70	1.01
Catchment 3 (CSD = 3.25hrs)	0.07	0.10	0.15
Catchment A (CSD = 3.5hrs)	0.09	0.12	0.17

3.6 Inflow hydrograph shape

3.6.1 As the MWC23 catchment is ungauged it was assessed as appropriate to derive hydrographs for input into the hydraulic model using ReFH2.3. For Run 1 the ReFH2.3 hydrographs were derived for Catchments 1, 2,3 and A based on a 7.5hr event storm duration, which is the critical storm duration for Catchment 2. For Run 2 the ReFH2.3 hydrographs were derived using the individual catchment CSD, ARF and SCF.

3.6.2 The Run 1 - 5% (1 in 20) and 1% (1 in 100) AEP event inflow hydrographs for Catchment 1, Catchment 2, Catchment 3 and Catchment A are presented in Plate 3.2 and Plate 3.3.

3.6.3 The shape of the Run 2 - 5% (1 in 20) and 1% (1 in 100) AEP event hydrographs for Catchments 1, 2, 3 and A, are very similar to the Run 1 inflow hydrographs with slight variations in the peak flow estimates as presented in Table 3.2 and Table 3.3. The Run 2 hydrograph shapes are therefore not presented in this document.

Plate 3.2 Run 1 - 5% (1 in 20) AEP event hydrographs

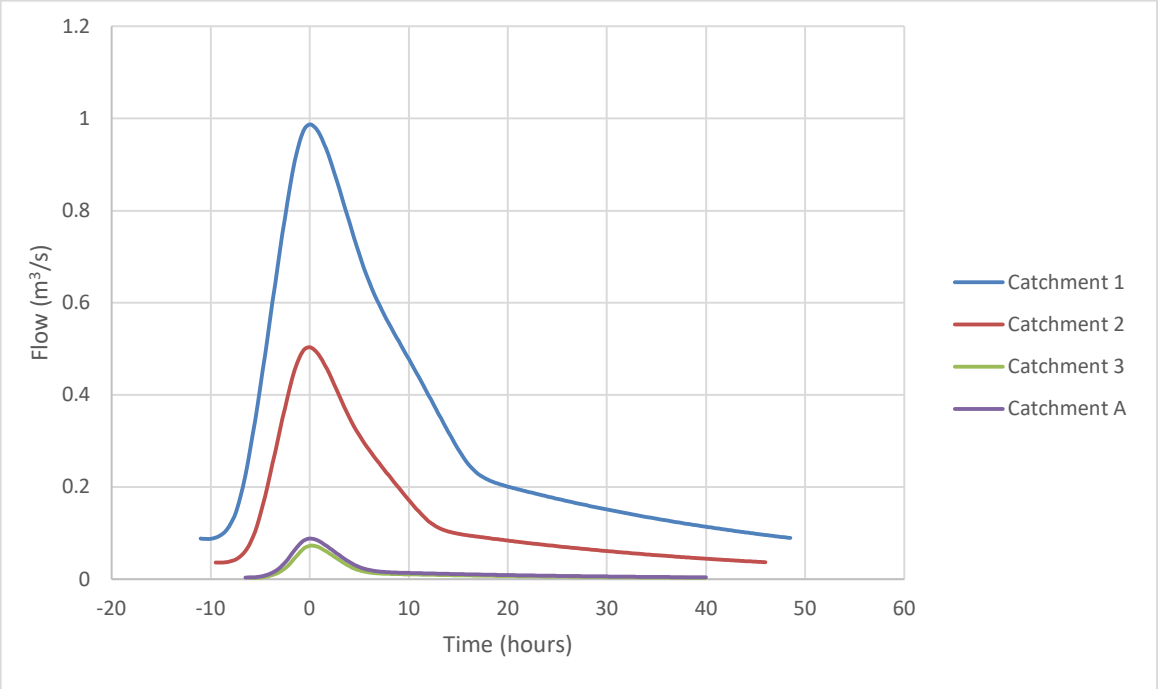
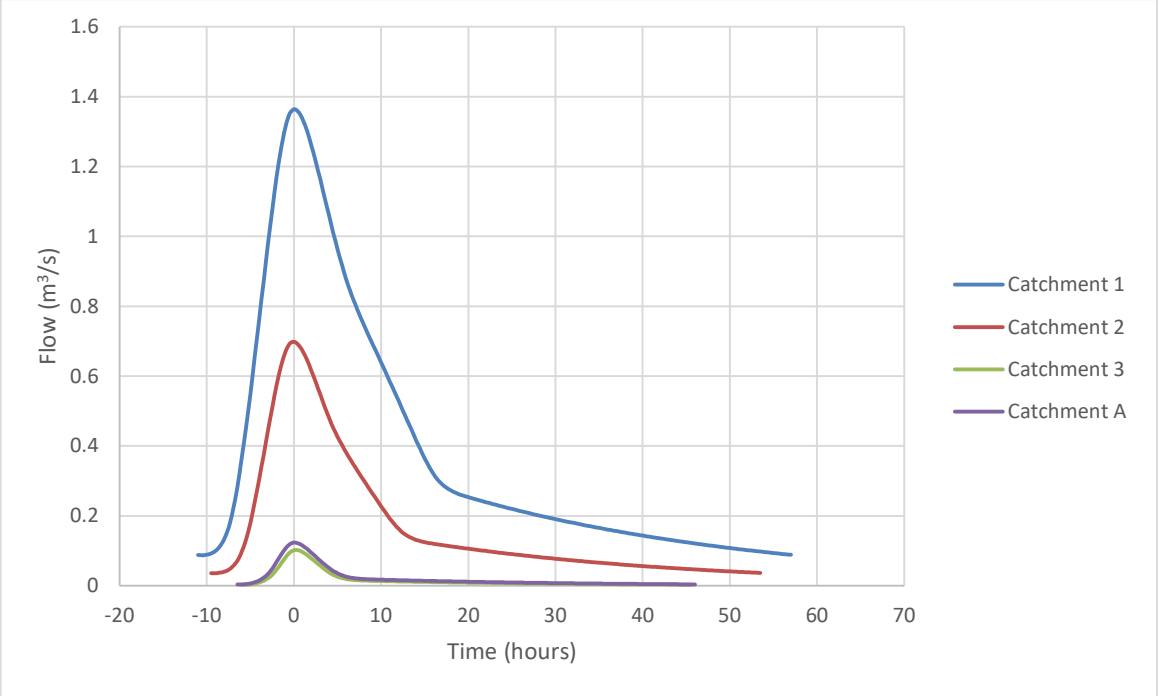


Plate 3.3 Run 1 - 1% (1 in 100) AEP event hydrographs



3.7 Climate change

3.7.1 A climate change allowance for peak rainfall intensity was obtained from the latest government guidance at the time of assessment (Environment Agency 2021). The peak rainfall intensity allowance is appropriate for small catchments (less than 5km^2). An uplift of 40% (the upper end, 2080s scenario) was applied. The peak for the 40% uplift for the 1% (1 in 100) AEP event derived using the individual catchment CSD, ARF and SCF (Run 2) are as follows; Catchment 1 = 1.97 m^3/s, Catchment 2 = 1.01m^3/s, Catchment 3 = 0.148m^3/s and Catchment A = 0.172m^3/s.

3.8 Flow reconciliation

3.8.1 Flow reconciliation was not directly undertaken for MWC23; however, an additional check was undertaken for high level verification purposes only. For this, a lumped catchment was utilised with an area of 3.36km^2 and a target flow was derived for the critical storm duration of 11 hours, which is equivalent to the CSD of Catchment 1. The analysis showed that hydraulic model results broadly matched the estimated target flow.

3.9 Summary

3.9.1 Design peak flows and inflow hydrographs for MWC23 were derived using ReFH2.3 and applying the FEH catchment descriptors available from the FEH Web Service / manual calculated key FEH catchment descriptors where appropriate. This occurred as the catchments are not picked up correctly by the FEH Web Service. Appropriate climate change allowance was applied as per the latest Environment Agency guidance. Design inflow hydrographs were derived for two hydraulic model runs, namely Run 1 – for the critical storm duration of Catchment 2 and Run 2 – for the individual sub-catchment critical storm durations. After undertaking the hydrological analysis with both sets of inflow hydrographs, it was noted that the inflow peaks are not sensitive to the choice of run scenario. Run 1 was therefore selected to take forward for hydraulic modelling. The results of Run 1 are presented in Section 3.5.

4 Baseline modelling

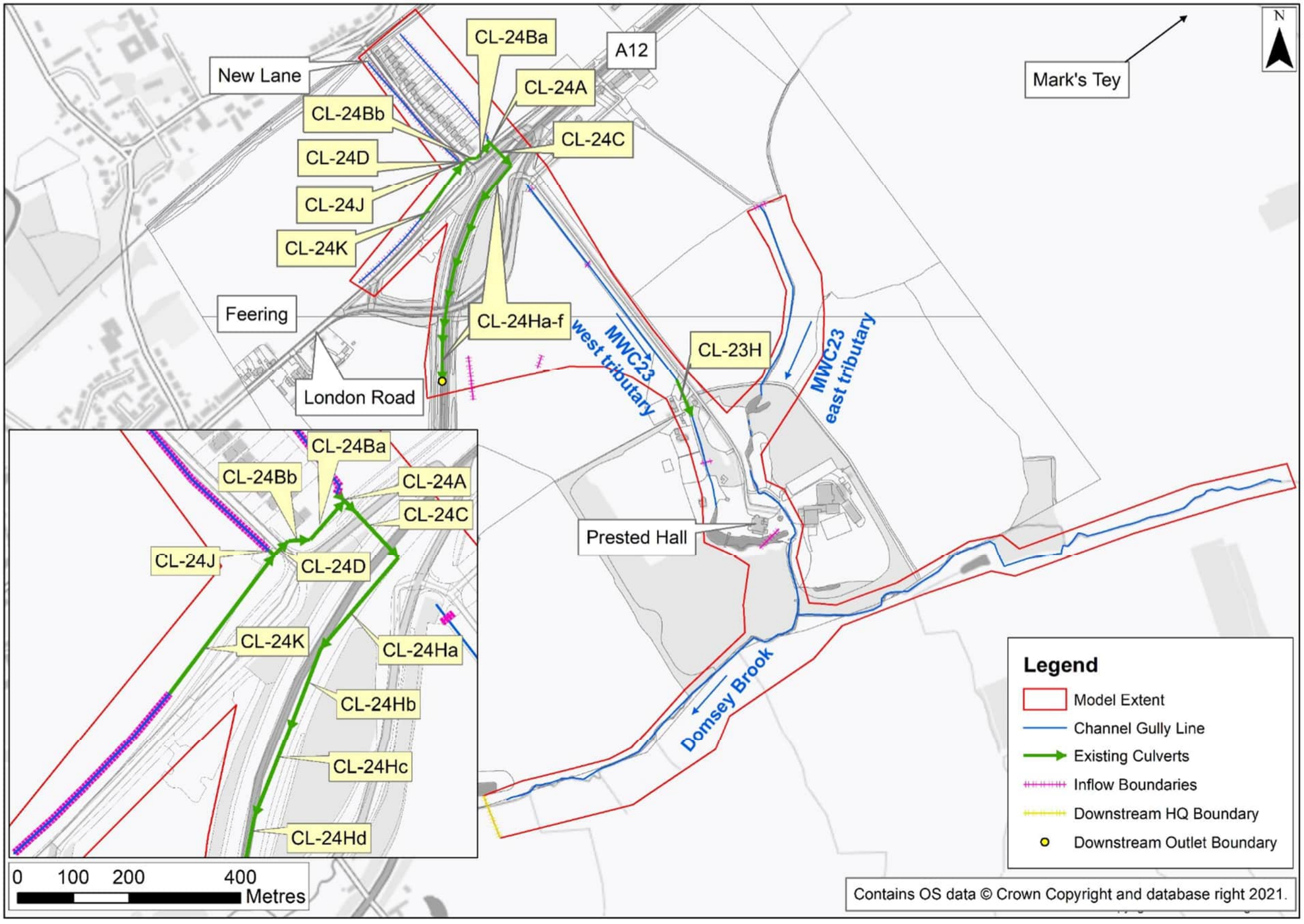
4.1.1 The MWC23 hydraulic model uses a two-dimensional (2D) schematisation, built in TUFLOW version 2020-10-AA (BMT, 2020). 1D 'ESTRY' elements were used to represent the culverts within the TUFLOW model and linked to the 2D domain via SX links. The 2D representation of the channel was considered appropriate because the highway culvert is the main control on pass forward flows and upstream water levels. Therefore, a detailed representation of the channel capacity is not required.

4.1.2 The model was built from scratch and used to simulate the baseline, with-scheme and mitigation scenarios. Three design events were simulated for each scenario: the 5% (1 in 20), 1% (1 in 100) and 1% (1 in 100) AEP +40%CC.

4.1 Model extent

4.1.3 The 2D model domain covers an area of 0.4 km^2. The model extent has been drawn in a way to capture the floodplains of each tributary and the downstream watercourse. Representation of the model extent is shown in Plate 4.1, and model configuration is explained in detail in Section 4.3.

Plate 4.1 Schematisation of the MWC23 baseline model



4.2 Model resolution and topography

4.2.1 1m horizontal resolution LiDAR DTM data from 2020 was available for the model extent and was downloaded from the DEFRA website to inform the hydraulic model with ground level information.

4.2.2 A 1m grid cell resolution was deemed appropriate for the model, as this is sufficient to represent the open channel, the floodplain topography and the active floodplain details while avoiding a long simulation run time. The model grid has been oriented to align with key flow paths.

4.2.3 The open channel sections were represented in the model using gully lines populated with nominal point data taken from the available topographic survey. Footbridge structures on the north tributary have been ignored, as investigation shows that these features would not make any difference to the model results within the area of interest.

4.2.4 All of the modelled culverts are concrete, circular type, with dimensions as outlined below:

- CL-24K has a diameter of 150mm and a length of 124m.

  CL-24J has a diameter of 225mm and a length of 6m.

  These pipes discharge into the culvert sections, CL-24D and CL-24Ba-b which have a 225mm diameter over a total length of 67m.

- CL-24A is the most upstream culvert of CL-24C and has a diameter of 225mm and a length of 8m. CL-24Ba and CL-24A converge at the upstream end of CL-23C, which is a 225mm diameter pipe with a length of 12m. The culvert continues into a diameter of 300mm for a further length of 16m.

  The remainder of this culverted section is A12 eastern surface drainage (CL-24Ha-g) which has a diameter of 450mm and a total length of 420m.

- CL-23H is a 600mm culvert with a length of 72m that conveys flow north to south under the properties at Prested Hall.

- All culverts were represented in the model using embedded 1D ‘ESTRY’ elements with a Manning’s ‘n’ roughness value of 0.015.

4.3 Hydraulic friction

4.3.1 OS MasterMap data was used to identify land use type and inform the TUFLOW model with different hydraulic roughness (Manning's 'n' coefficient) values. Hydraulic roughness coefficients are applied over each grid cell of the 2D domain depending on the land use taken from the MasterMap data, as shown in Table 4.1. Roughness values adopted were taken from standard guidance (Chow, 1959).

4.3.2 As mentioned in Section 4.3 above, all culverts were modelled assuming a roughness value of 0.015, taken from Chow (1959) as a high value for concrete culverts that are straight and free of debris. Note that a high value for roughness was used, to be conservative, since there was no survey or site data available.

Table 4.1 MWC23 roughness

Land Use	Manning's 'n' roughness
Buildings	0.2
General Surface (Multiple)	0.05
General Surface (Step)	0.05
General Surface (Natural)	0.05
Cliffs	0.05
Orchard	0.075
Roads/Tracks/Paths	0.02
Roadside	0.04
Rail	0.05
Manmade Structures	0.025
Slopes	0.05
Unclassified Land	0.05
Buildings/Glasshouses	0.5
Water, inland, tidal	0.025
Natural Land	0.05
Structures (Pylons/on top of Buildings)	0.055
Unclassified	0.035
Channel roughness	0.04

4.4 Boundary conditions

4.4.1 Two types of boundary conditions have been implemented in the model:

- Flow-time boundary lines have been used to represent; the MWC23 upstream tributary inflows, lateral inflows downstream of the A12 crossing within the MWC23 channel, and the lateral inflows upstream of the A12 crossing.
- The downstream boundary is applied as a head-flow (HQ), line normal depth boundary type allowing free flow out of the model. The location is shown in Plate 4.1.

5 Modelled events

5.1.1 Table 5.1 shows the AEP events and model scenarios that were simulated with the hydraulic model. A climate change event was modelled: the 40% uplift in accordance with the latest Environment Agency guidance as detailed in Section 3.7.

5.1.2 As discussed in Section 3, two model runs were required to assess the critical storm duration for MWC23. The two runs were compared for all three events using the baseline model. Model results were seen to be not sensitive to the adjustment of storm duration, and Run 1 was utilised in the analysis going forward.

5.1.3 In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1% (1 in 100) AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients and hydrological inflows (see Section 6.4).

Table 5.1 Modelled events for MWC23

Scenario	5% (1 in 20) AEP	1% (1 in 100) AEP	1% (1 in 100) AEP +40%CC
Baseline	X	X	X
With-scheme	X	X	X
Mitigation	X	X	X
Roughness Sensitivity		X	
Hydrological Inflow Sensitivity		X	

6 Model proving

6.1 Introduction

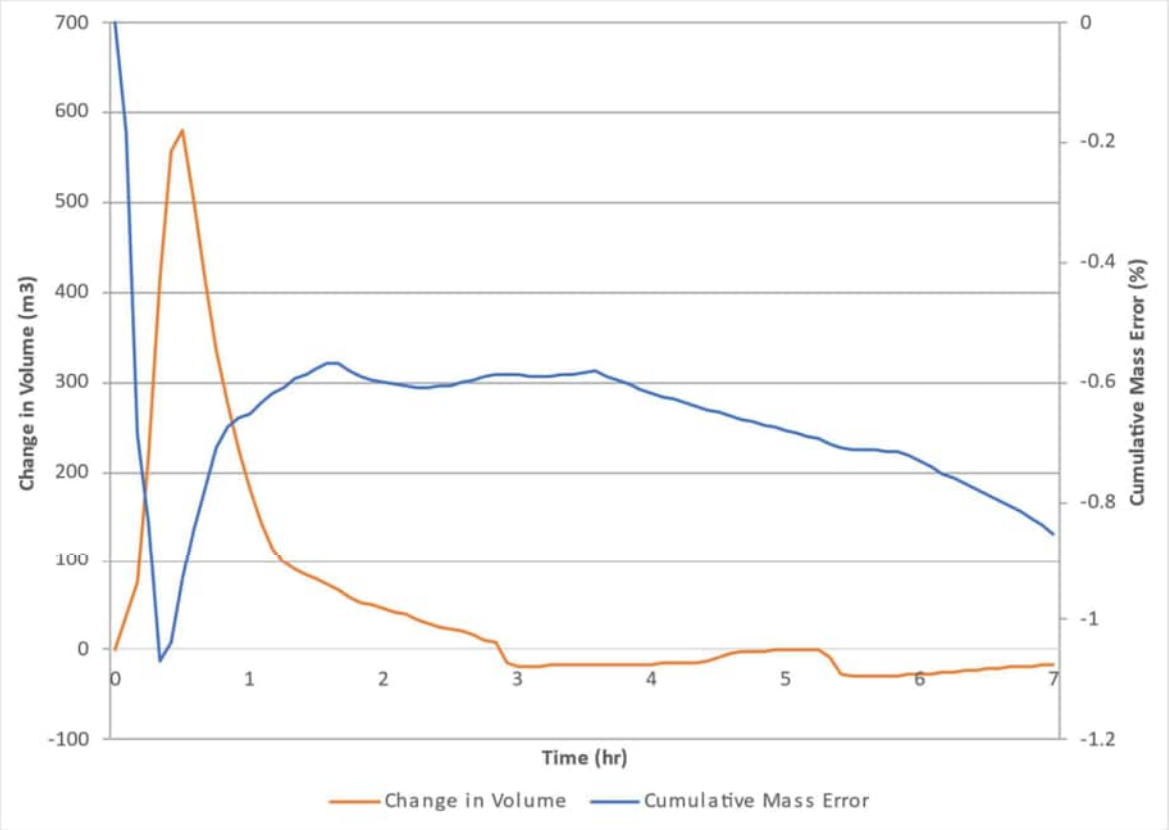
6.1.1 The following sections discuss the model performance and the verification process. In addition, details relating to the additional runs carried out to test the sensitivity of the model to key variables are also discussed.

6.2 Model performance

6.2.1 Run performance has been monitored throughout the model build process and then during each simulation carried out, to ensure a suitable model computational performance was achieved. The cumulative mass error diagnostics output from the model have been checked for each model simulation. Plate 6.1 shows the cumulative mass error and the change in volume for the baseline 1% (1 in 100) AEP event simulation. This plot is typical for all the events and scenarios simulated. The tolerance range recommended by the software manual is +/-1% mass balance error.

6.2.2 Although the cumulative mass error tolerance is slightly exceeded during the onset of the floodplain wetting, it reduces to less than 1% error prior to the event peak.

Plate 6.1 Model performance for the 1% (1 in 100) AEP event

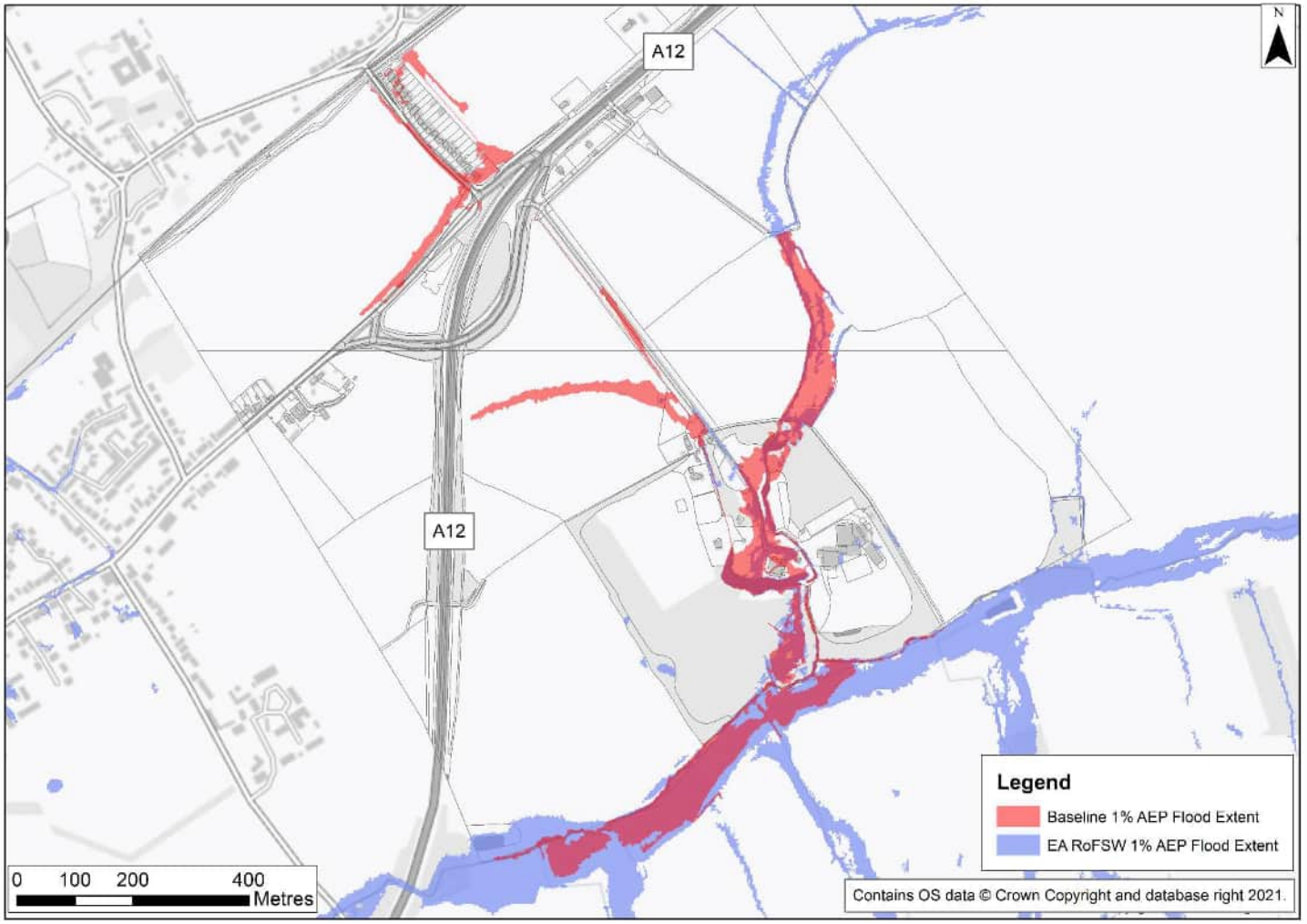


6.3 Comparison with EA flood maps

6.3.1 MWC23 has not been included in the Environment Agency published Flood Zone mapping of fluvial flood risk. Therefore, a direct comparison between model results and Environment Agency flood maps is not possible. However, the Environment Agency Risk of Flooding from Surface Water (RoFSW) mapping can be used to identify areas of pooling.

6.3.2 Plate 6.2 shows the modelled 1% (1 in 100) AEP event flood extent and the EA RoFSW mapping extents. Flood extents are generally similar in all areas apart from along Domsey Brook, which was not explicitly included in the MWC23 model. Upstream of the A12, the surface water maps show slightly larger extent. There are also areas where the surface water map shows no risk of flooding, whereas the model predicts more widespread flooding across the floodplain. The disparity may relate to the different methodologies for applying rainfall. Considering this, verification of the model cannot be based on the comparison of results with EA flood maps. The analysis is therefore reliant on the use of industry standard modelling and hydrology tools, as is common for small watercourse studies.

Plate 6.2 Environment Agency surface water map comparison with modelled 1% (1 in 100) AEP event



6.4 Sensitivity analysis

6.4.1 Simulations were carried out to determine the sensitivity of the model to roughness and hydrological inflows. All sensitivity tests were performed using the 1% (1 in 100) AEP event. 2D model outputs for the sensitivity testing are presented in Appendix B.

Roughness sensitivity

6.4.2 In-channel and floodplain roughness coefficients (Manning's 'n') were changed by +20% and -20%. Appendix B show the impact of changing the model roughness on depth of flooding. The results show that the changes in flood depth are mostly negligible with a water level change of up to +/-15mm upstream of the A12, and +/-20mm around the north tributary.

Hydrological inflow sensitivity

6.4.3 The flows into the model were adjusted by +20% and -20%. Appendix B show the impact of changing the model inflows on depth of flooding. The results show that the changes in flood depth are moderately sensitive especially upstream of the A12 and around Prested Hall, with a water level change of up to +/-62mm upstream of the A12 and widespread response of up to +/-75mm along the north tributary.

7 Design modelling

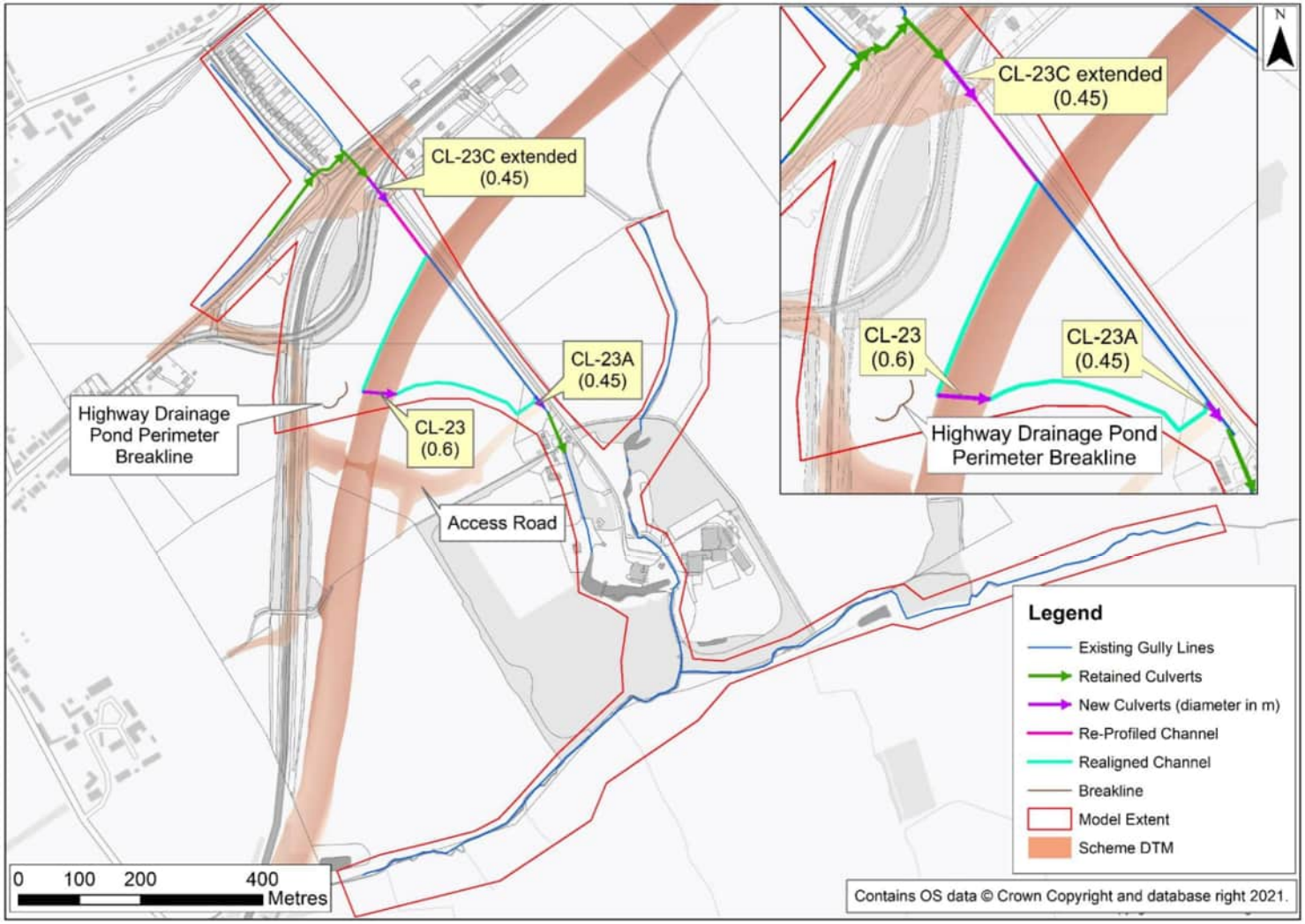
7.1 Design model build

7.1.1 The baseline model was updated to include the proposed A12 scheme. Changes to the model were as follows and illustrated in Plate 7.1:

- The proposed A12 and the side road topography was included in the model using an Ascii DTM surface stamped onto the baseline model topography.
- The west tributary was disconnected from existing A12 road drainage (CL-24C) and reconnected to the MWC23 west tributary south of A12, via an extended section of the 450mm concrete circular culvert connected to the CL-24C culvert.
- As shown in Plate 7.1 the channel immediately downstream from the existing A12 was re-routed south-west along the toe of proposed offline A12 embankment, prior to passing under the new A12 road in a 600mm culvert (CL-23). The watercourse is then routed back to the original west tributary, via a new open channel, and re-joining the original watercourse immediately upstream of the new access road.
- The retained MWC23 west tributary immediately downstream of the existing A12 was resized and regraded; its channel width was set to 1m and the bed level at the downstream end was lowered by 0.5m, to ensure all flows are appropriately managed.
- An additional 450mm culvert (CL-23A) was included where the new access road crosses the realigned channel as shown in Plate 7.1.
- The inlet and outlet invert levels for each proposed culvert were tied into the upstream and downstream channel bed level.

7.1.2 As part of the proposed-scheme, a highway drainage attenuation pond will partly occupy the west floodplain in between the existing A12 and new A12 crossing. To represent this feature, a 2D breakline is included in the model as shown in Plate 7.1, using TUFLOW Zshape line features, with a crest level set at a nominal high value of 35.0mAOD.

Plate 7.1 With-scheme scenario layout



8 Model results

8.1 Baseline results

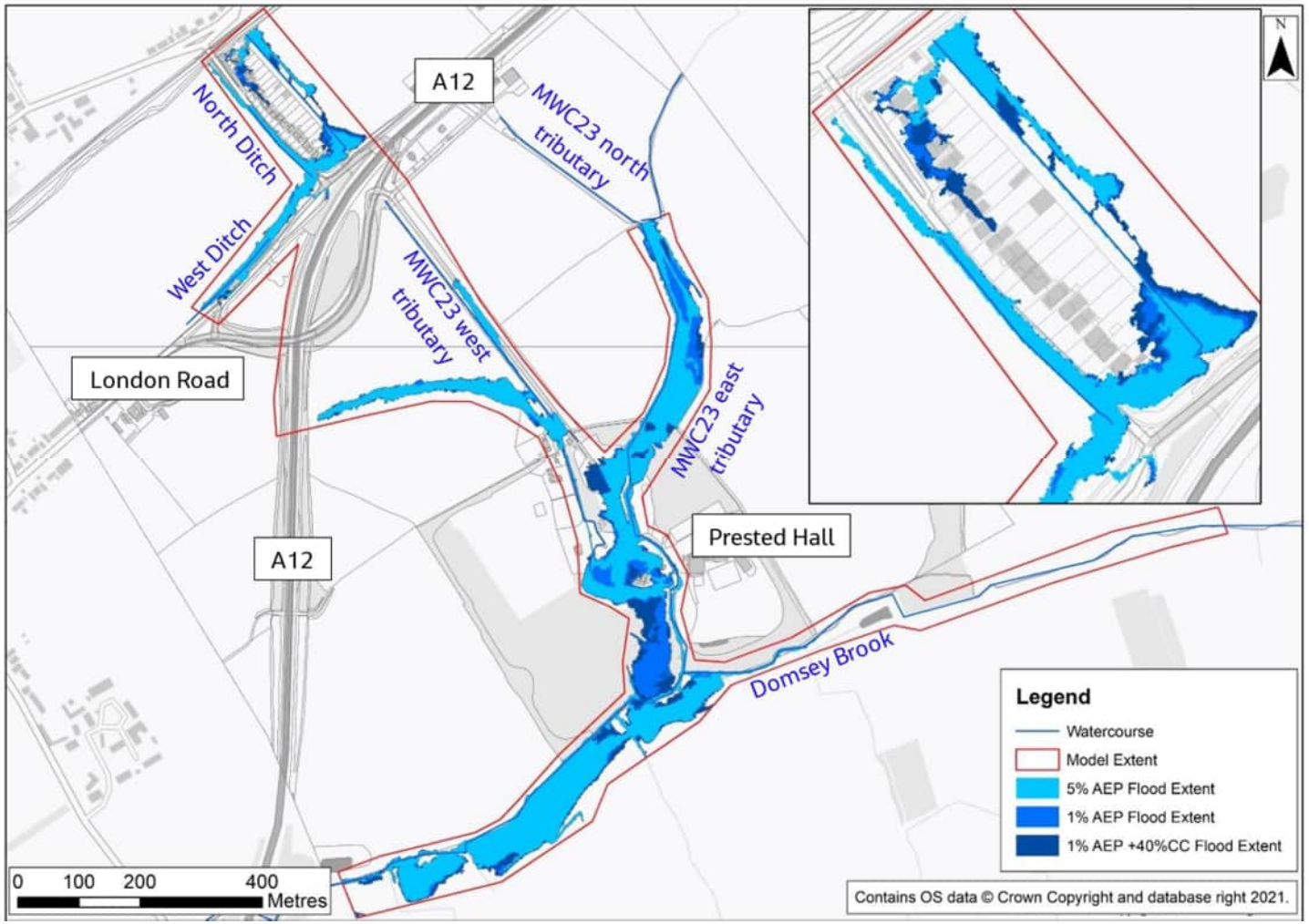
8.1.1 Plate 8.1 shows the maximum flood extent for all modelled events for the baseline scenario. The flood extents for each of the modelled events can also be found in Appendix C.

8.1.2 The area upstream of the A12 is flooded of the highway for all events simulated with high flows backing up from the culvert system and ponding affecting properties on New Lane.

8.1.3 There is no flooding to the A12 highway.

8.1.4 There is active flooding on the channels downstream of the A12 crossing, for all modelled events, and part of the Prested Hall Hotel complex is affected.

Plate 8.1 Baseline flood extents for all modelled events



8.2 With-scheme model results

8.2.1 Plate 8.2 shows the flood extents for the with-scheme scenario events modelled. It shows that flooding occurs on the proposed-scheme roundabout, and access road in all events (see Plate 8.3). The re-aligned channels are shown to have capacity to carry design flows in all events and baseline flooding is removed along this flow path.

8.2.2 Plate 8.3 shows the water level difference map between the with-scheme scenario and the baseline scenario for the 5% (1 in 20) AEP event. It shows an adverse water level impact that is restricted to the pond feature on the south side of Prested Hall. There is also a small area of increased flood risk on the retained West Tributary channel immediately upstream of the new A12.

8.2.3 Plate 8.4 shows the water level difference between the with-scheme scenario and the baseline scenario for the 1% (1 in 100) AEP event. For this event the flood risk impacts of the proposed-scheme show a moderate increase in floodplain water levels over the open ground to the south of Prested Hall.

8.2.4 Plate 8.5 shows the water level difference between the with-scheme scenario and the baseline scenario for the 1% (1 in 100) AEP +40%CC event. It shows that in general the flood risk impact downstream of the scheme is negligible. The only area of increased flood risk is the small area on the retained West Tributary channel immediately upstream of the new A12.

8.2.5 There is no change in flood risk to the properties on New Lane for all modelled events.

Plate 8.2 With-scheme flood extents for all modelled events

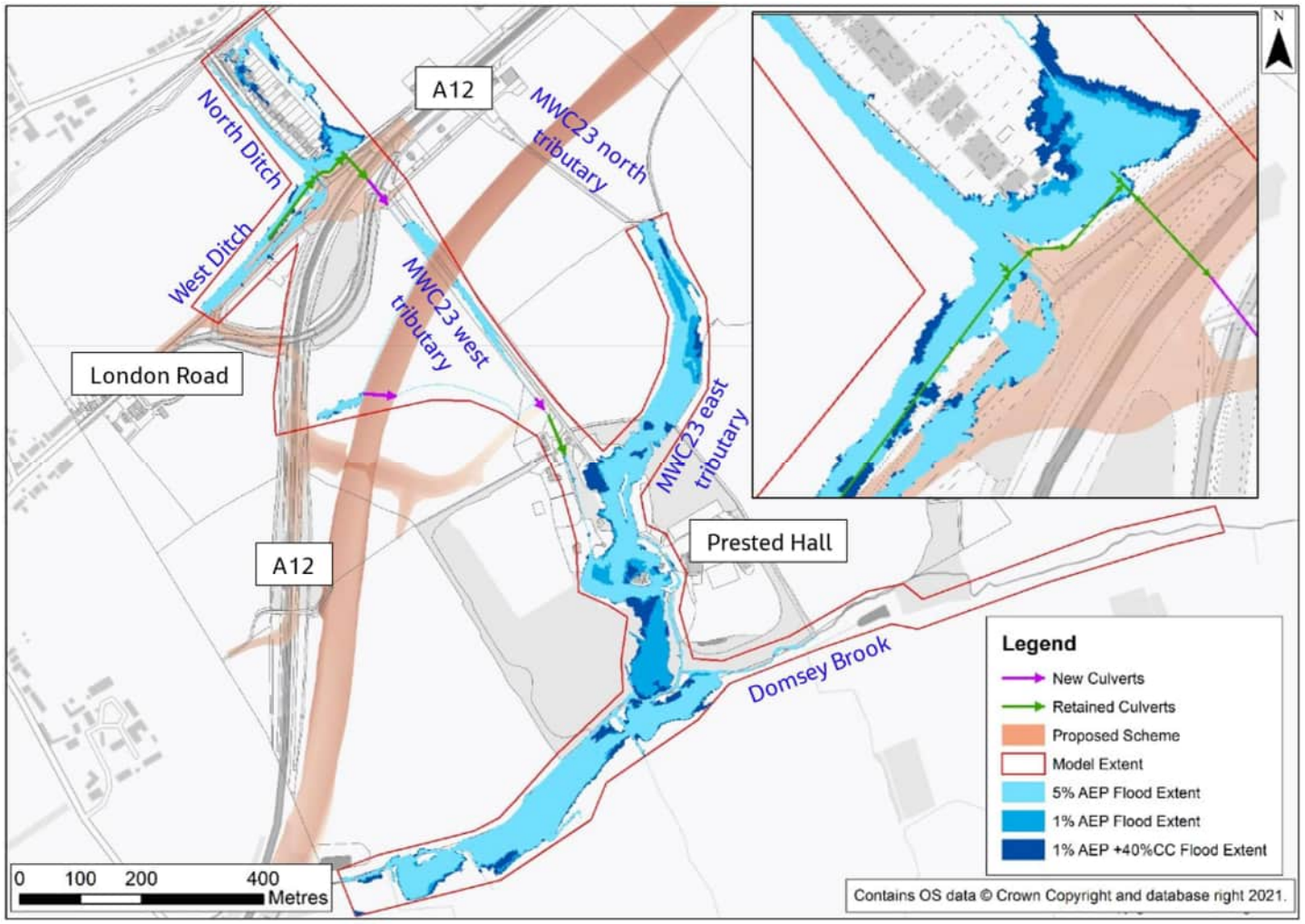


Plate 8.3 Water level difference (with-scheme minus baseline) map for the 5% (1 in 20) AEP event

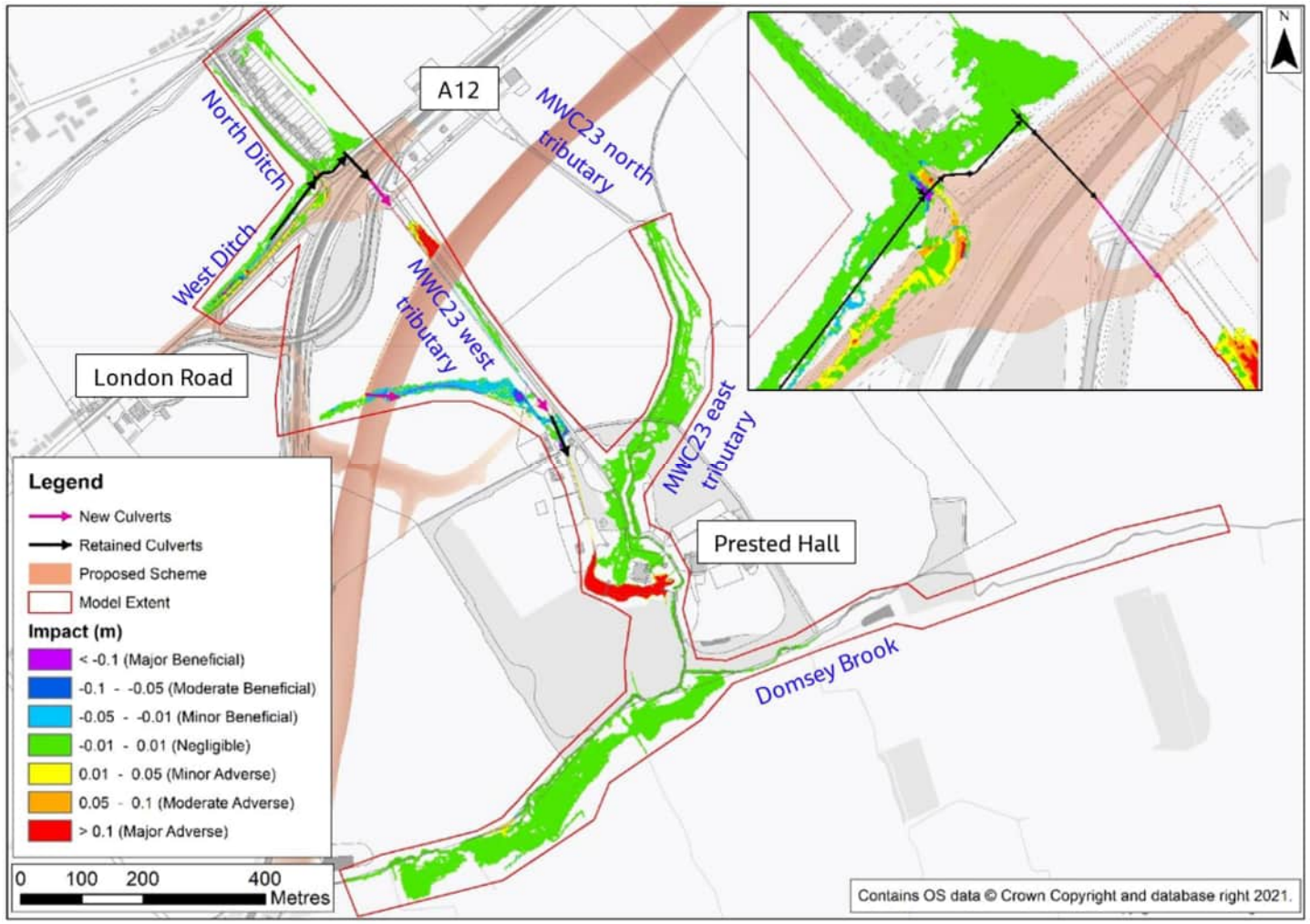


Plate 8.4 Water level difference (with-scheme minus baseline) map for the 1% (1 in 100) AEP event

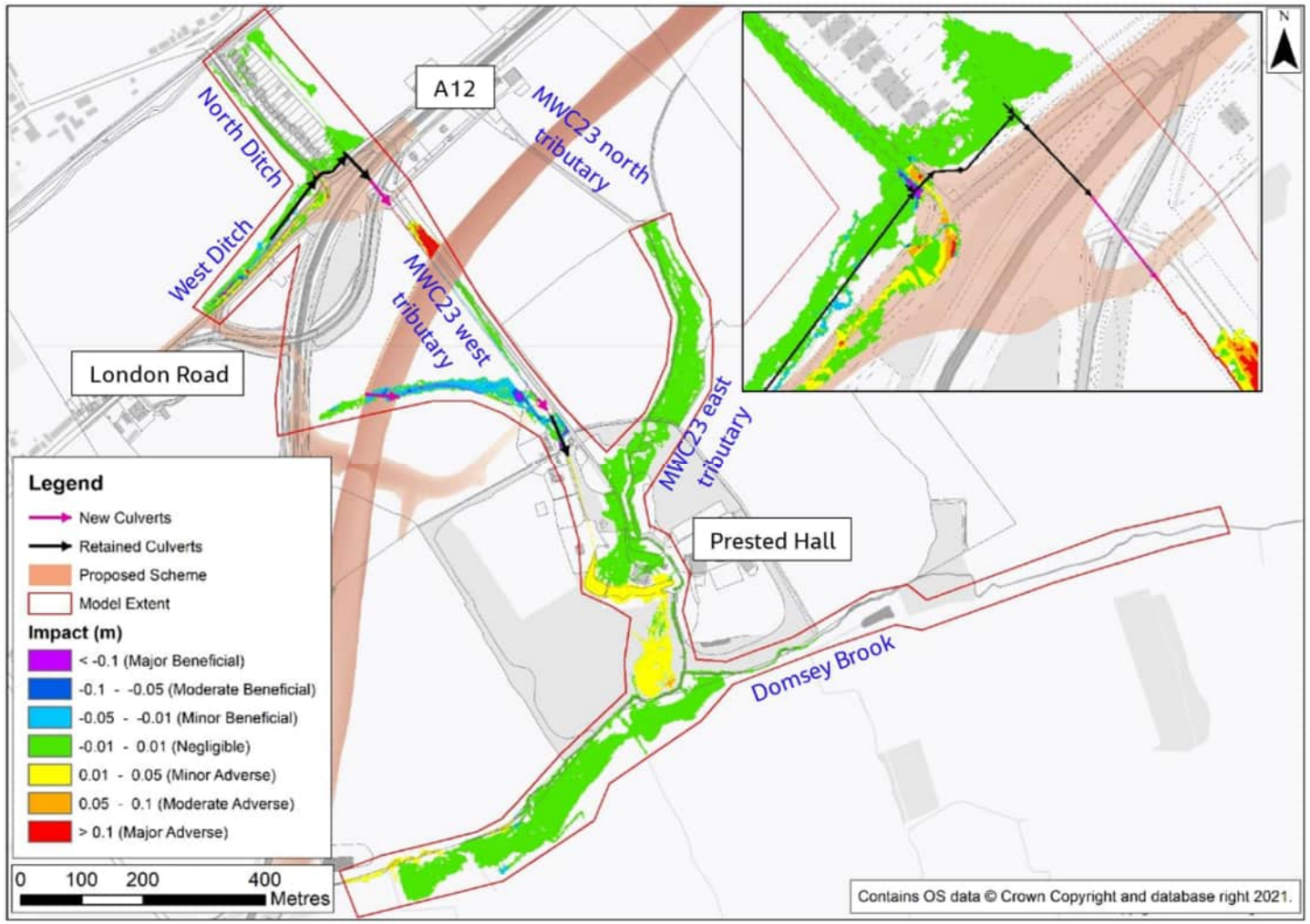
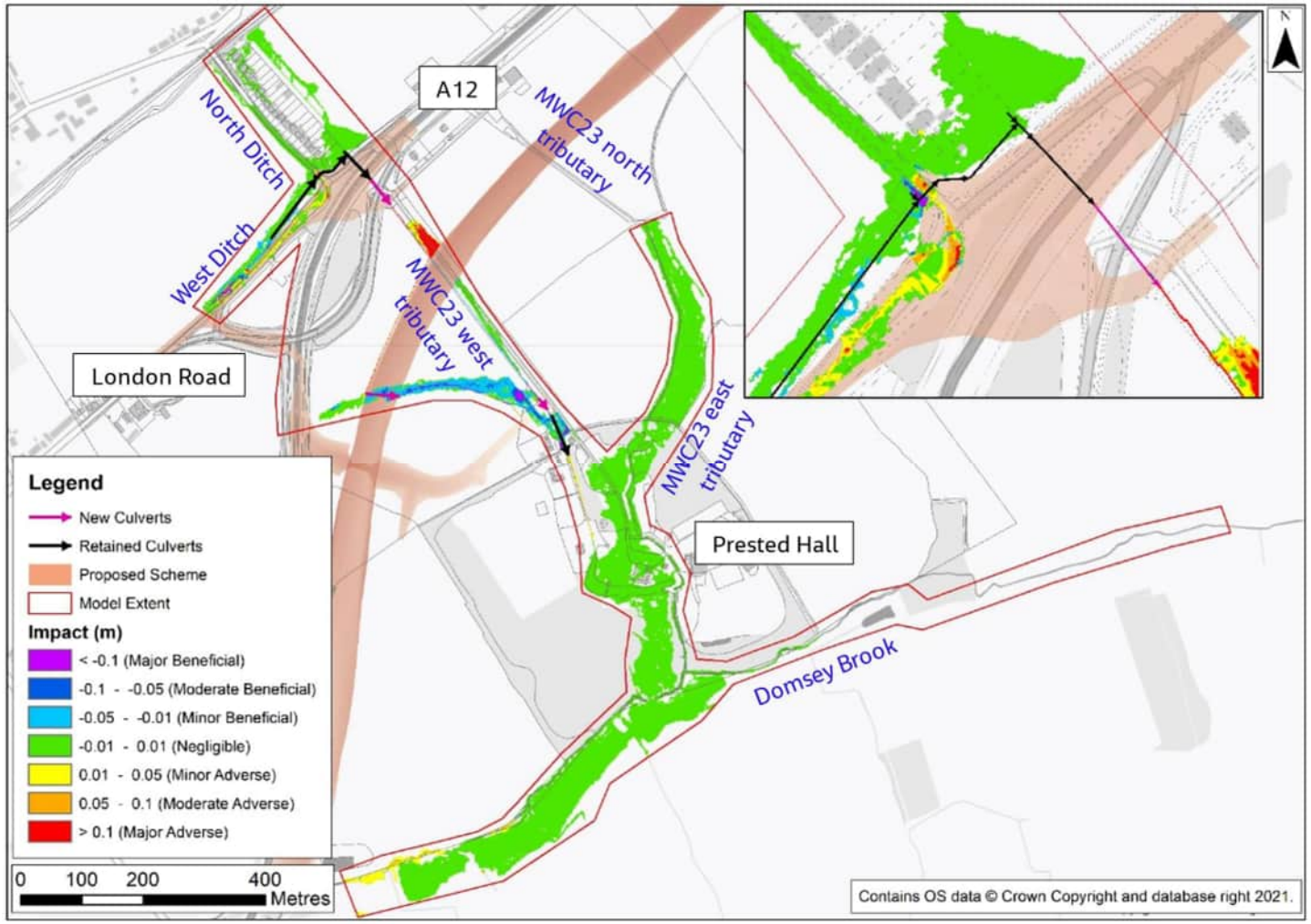


Plate 8.5 Water level difference (with-scheme minus baseline) map for the 1% (1 in 100) AEP +40%CC event



9 Mitigation modelling

9.1 Mitigation model build

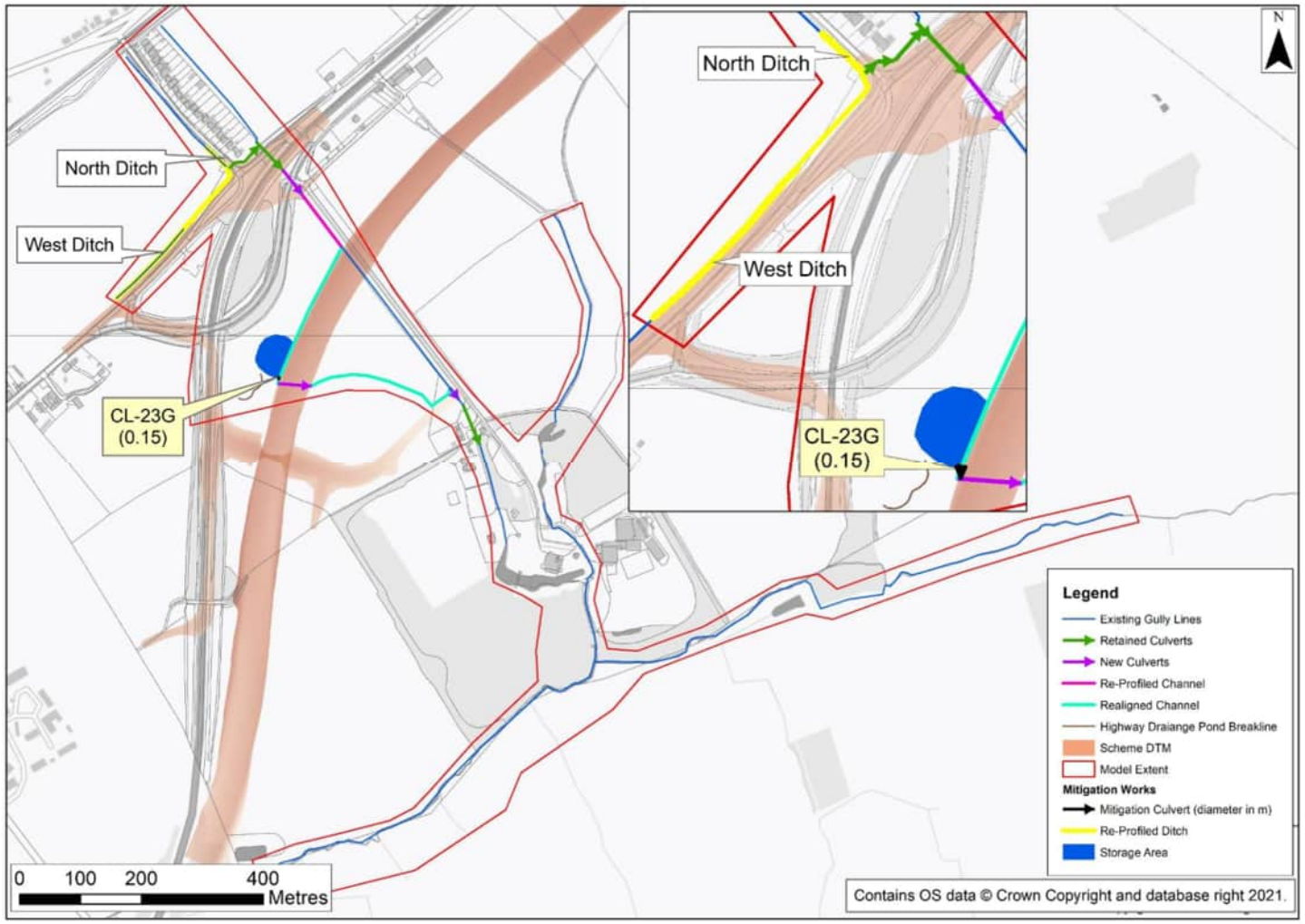
9.1.1 The with-scheme model was updated to include the mitigation measures to address the impacts resulting from the proposed-scheme.

9.1.2 Changes to the model were as follows and illustrated in Plate 9.1:

- The 150mm diameter culvert, CL-23K, west of New Lane is replaced with a ditch system, to contain flows and protect the new A12 access road. The ditch is formed of two “legs”; (1) the West Ditch is adjacent to the A12 access road and is up to 3m width, with an average depth of 1.2m, and (2) the North Ditch which will connect directly to CL-24D, following removal of CL-24F, is 1m wide.

- A small flood attenuation storage area was implemented immediately upstream of the new offline A12 embankment, ensuring that additional flow from the CL-24C pipe reconnection is mitigated to avoid increased peak pass-forward flows further downstream. The pond itself is an excavated area that is connected to the channel by a 150mm culvert (CL-23G) to throttle flows and hold back water and provide the flow attenuation. The excavation depth was set to a notional bed level which is up to a maximum of 2.6m depth, with a gradient in the topography of the pond has been included ensure it drains to empty. The size of the storage has not been optimised and final dimensions for the facility should be determined at future design phase.

Plate 9.1 Mitigation scenario layout



9.2 Mitigation model results

9.2.1 Plate 9.2 shows the flood extents for the With-Mitigation Scenario. It shows that for all events there is no longer any flooding at the proposed-scheme roundabout.

9.2.2 Plate 9.3 shows the water level difference due to the mitigation measures implemented relative to the baseline scenario for the 1% (1 in 100) AEP +40%CC event. The maximum flood level difference between baseline and “with-scheme plus mitigation” are categorised into levels of impact. The impact of the mitigation on water level for all flood events can be found in Appendix C as well as the flood extents for all flood events modelled.

9.2.3 It can be seen that the use of the attenuation pond limits the effect of the scheme on downstream flood risk, with only negligible differences in depth of flooding relative to the baseline scenario.

9.2.4 The new North Ditch and West Ditch mitigation features contain flows for all modelled events, and the proposed-scheme highway elements remain free from flooding for all modelled AEP events.

Plate 9.2 Mitigation flood extents for all modelled events

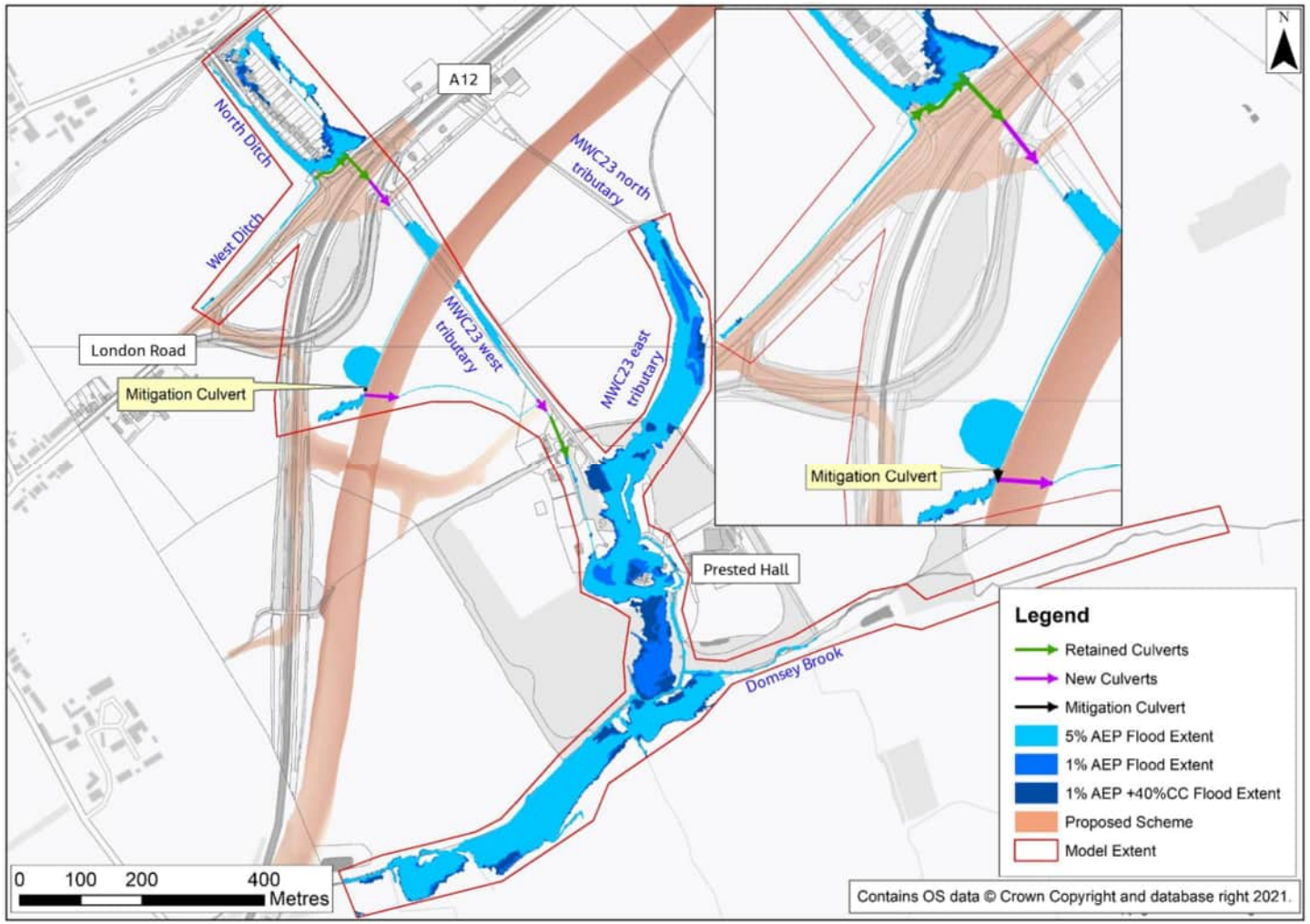
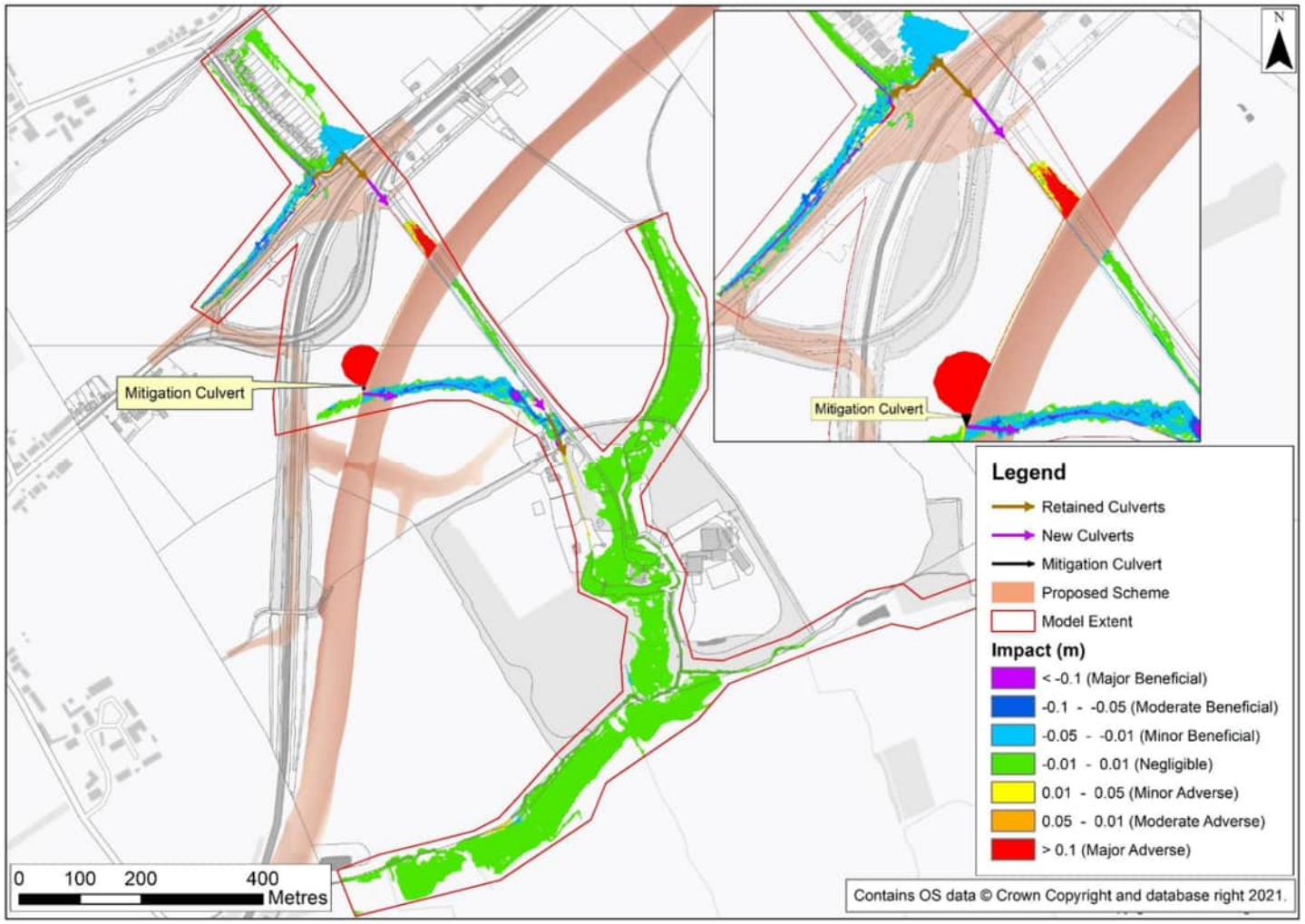


Plate 9.3 Water level difference (mitigation minus baseline) map for the 1% (1 in 100) AEP +40%CC event



10 Model assumptions and limitations

10.1 Introduction

10.1.1 The accuracy and validity of the hydraulic model results is dependent on the accuracy of the hydrological, surveyed, and topographic data included in the model. While the most appropriate available information has been used to construct the model to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the model. These include assumptions made as part of the model build process.

10.1.2 Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are generally conservative for modelled water levels at the proposed-scheme and are therefore, appropriate for the flood risk assessment.

10.2 Hydrology

10.2.1 The main hydrological assumptions and limitations used in this investigation are as follows:

- As MWC23 is a small ungauged catchment there is inevitable uncertainty associated with flow estimation for small, ungauged catchments.
- ReFH2.3 is most appropriate for the target catchment as it uses BFIHOST19 and more recent (FEH13) rainfall data.
- FEH Statistical is not suitable due to incorporation of BFIHOST and due to the catchment containing HOST Class 25 soils.

10.3 Hydraulic modelling

10.3.1 The key sources of uncertainty and the limitations associated with the modelling undertaken for MWC23 are as follows:

- Culvert and model roughness values have been assigned across the model using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow 1959).
- Hydraulic coefficients for structures have been applied using available guidance within the TUFLOW Manual. The dimensions for structures have been based on detailed survey measurements.
- The LiDAR data is assumed to appropriately represent the floodplain.
- A 1m grid has been used. This is deemed a sufficient level of detail to represent floodplain topography and flooding mechanisms predicted by the model in the vicinity of the proposed-scheme.

- No calibration was carried out as the catchment is ungauged.
- The upstream part of Catchments 1 and 2 fully discharges into the relevant downstream watercourse tributary, with no flow restrictions caused by the railway culverts. This is a conservative assumption.
- The west tributary freely discharges into the online pond located west of Prested Hall. As there was no survey available, the feature was represented using LiDAR data and the available site photography.

11 Conclusion

11.1.1 This report has detailed the modelling carried out to assess the baseline and with-scheme flood risk for Minor Watercourse 23 and its tributaries with respect to the proposed highway scheme.

11.1.2 The results of the baseline modelling have shown that the A12 carriageway is unaffected by flooding although there are a number of properties affected upstream of the A12 on New Lane and around Prested Hall.

11.1.3 The proposed scheme includes new topography, channel re-alignment and culverts under the new roads. The inclusion of the proposed scheme in the model, causes increased flooding on the open area to the south of Prested Hall, and a small area immediately upstream of the proposed offline highway route. there is no increase in flood risk to any properties, notably those on New Lane upstream of the current A12.

11.1.4 To mitigate the change in flood risk, a storage pond with culvert flow control and modified ditches were introduced. This ensured that any change in flood risk due to the scheme remains negligible.

11.1.5 The A12 highway, junctions and associated access roads remain free from flooding in all the events modelled.

Acronyms

Abbreviation	Term
AEP	Annual exceedance probability
ARF	Areal reduction factor
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CC	Climate Change
CSD	Critical storm duration
EA	Environment Agency
FEH	Flood Estimation Handbook
GIS	Geographic Information System
HOST	Hydrology of Soil Types
MWC	Minor watercourse
QMED	Median Annual Flood (with return period 2-years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
RoFSW	Risk of Flooding from Surface Water
SAAR	Standard Average Annual Rainfall (mm)
SCF	Seasonal correction factor
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

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Appendix A – Minor Watercourse 23 Hydrology

See document HE551497-JAC-EWE-SCHW-RP-LE-0092

Appendix B – Sensitivity Testing

Plate B.1 Water level difference map for the reduced Manning's 'n' sensitivity test

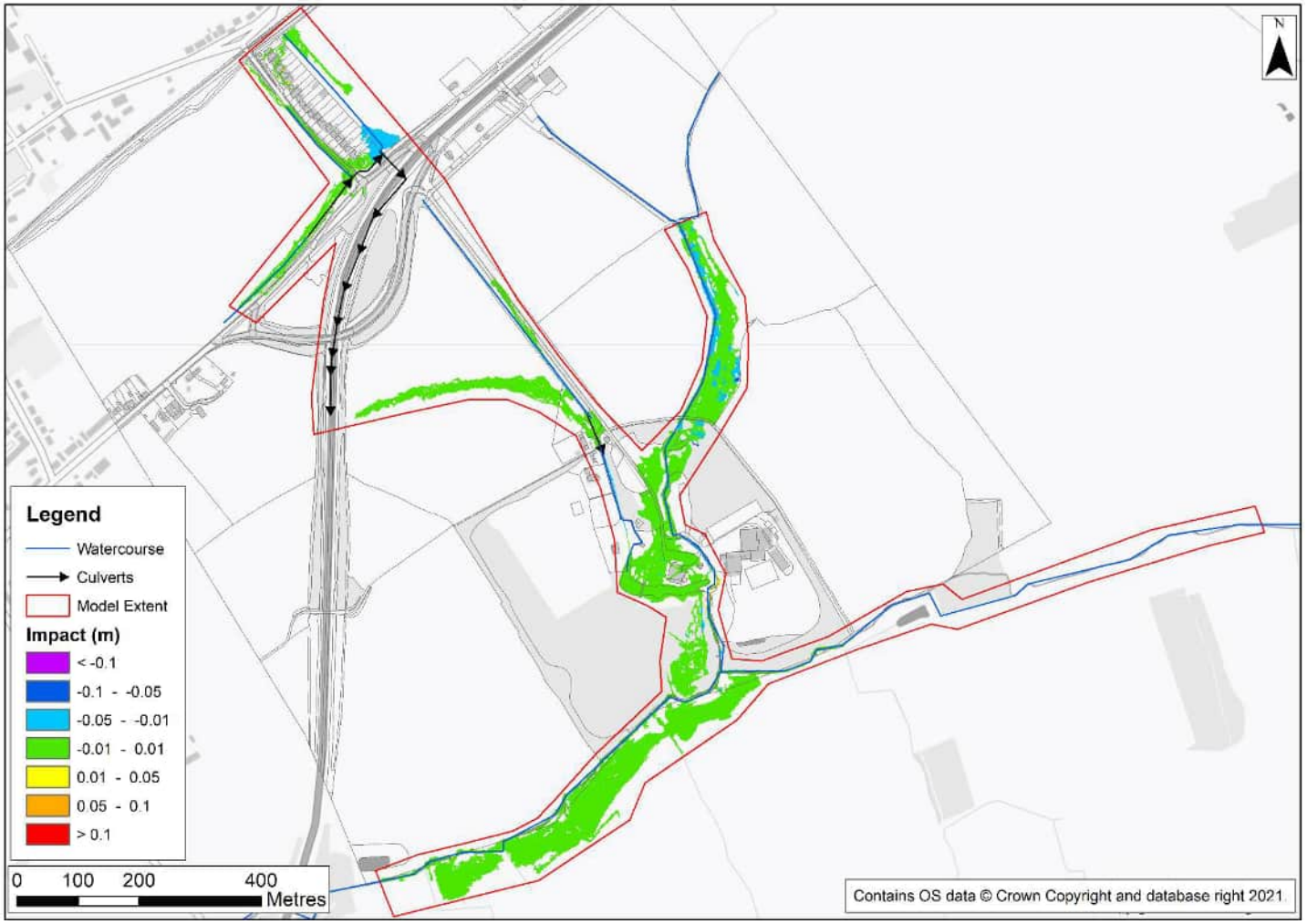


Plate B.2 Water level difference map for the increased Manning's 'n' sensitivity test

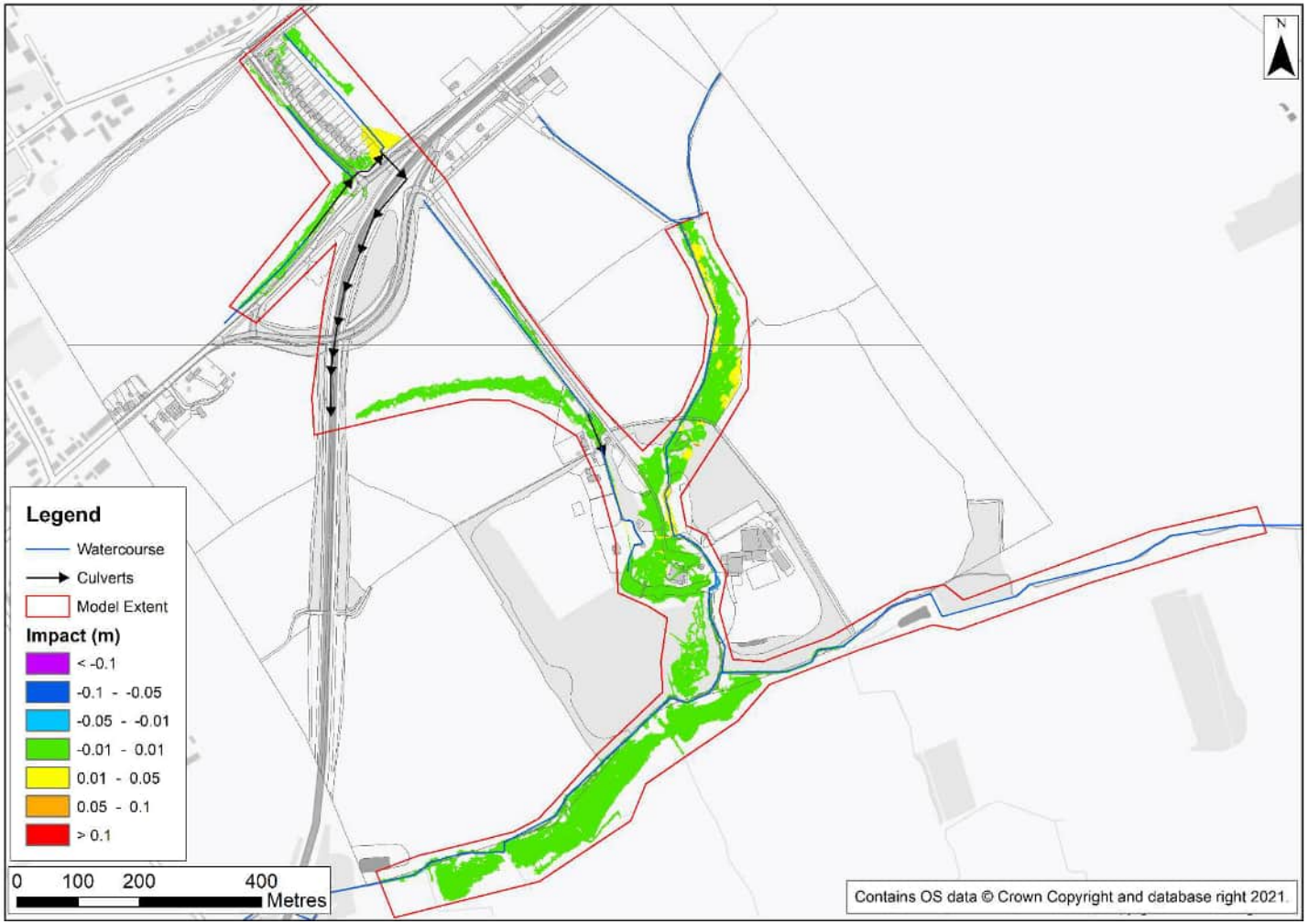


Plate B.3 Water level difference map for the reduced inflow sensitivity test

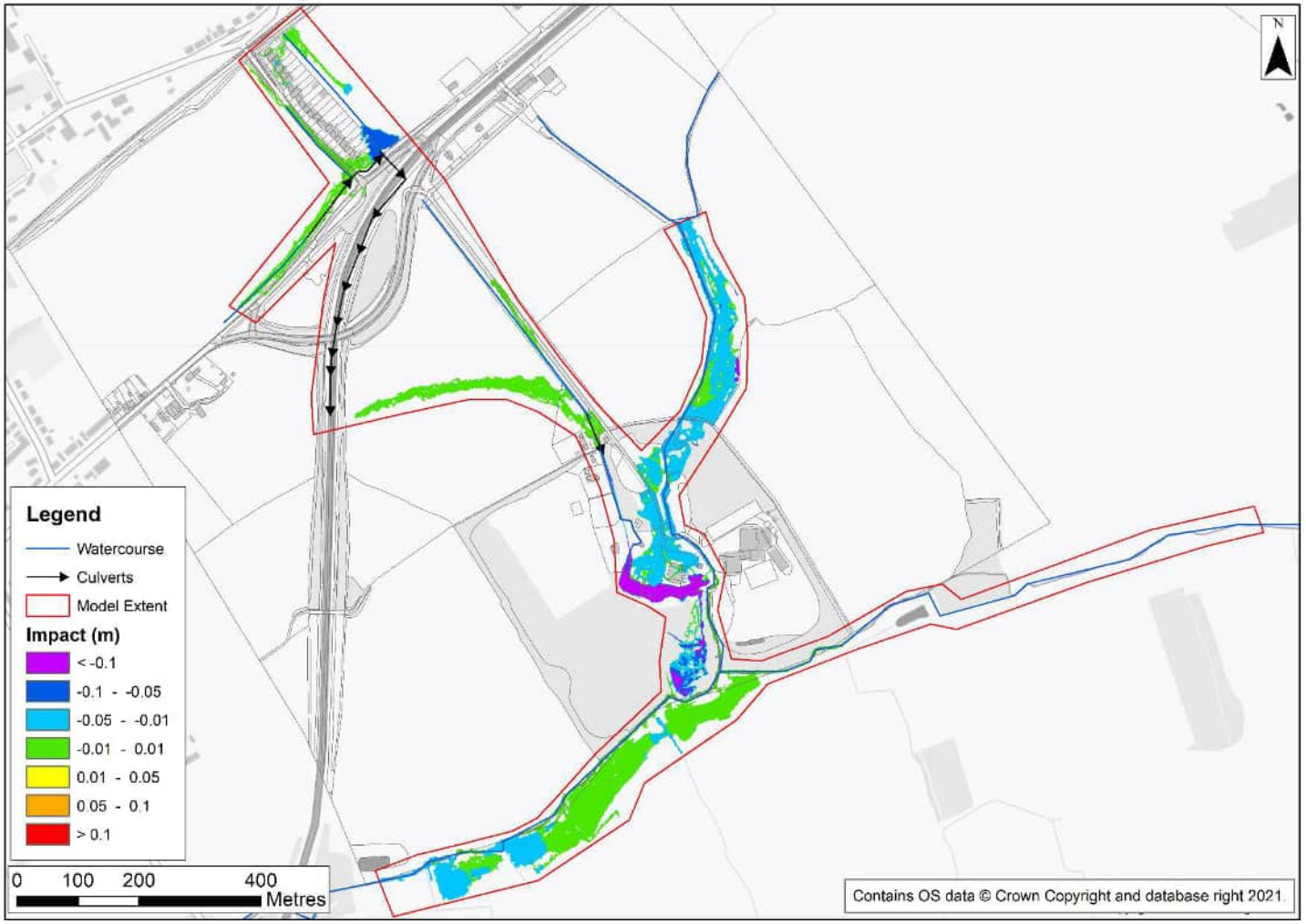
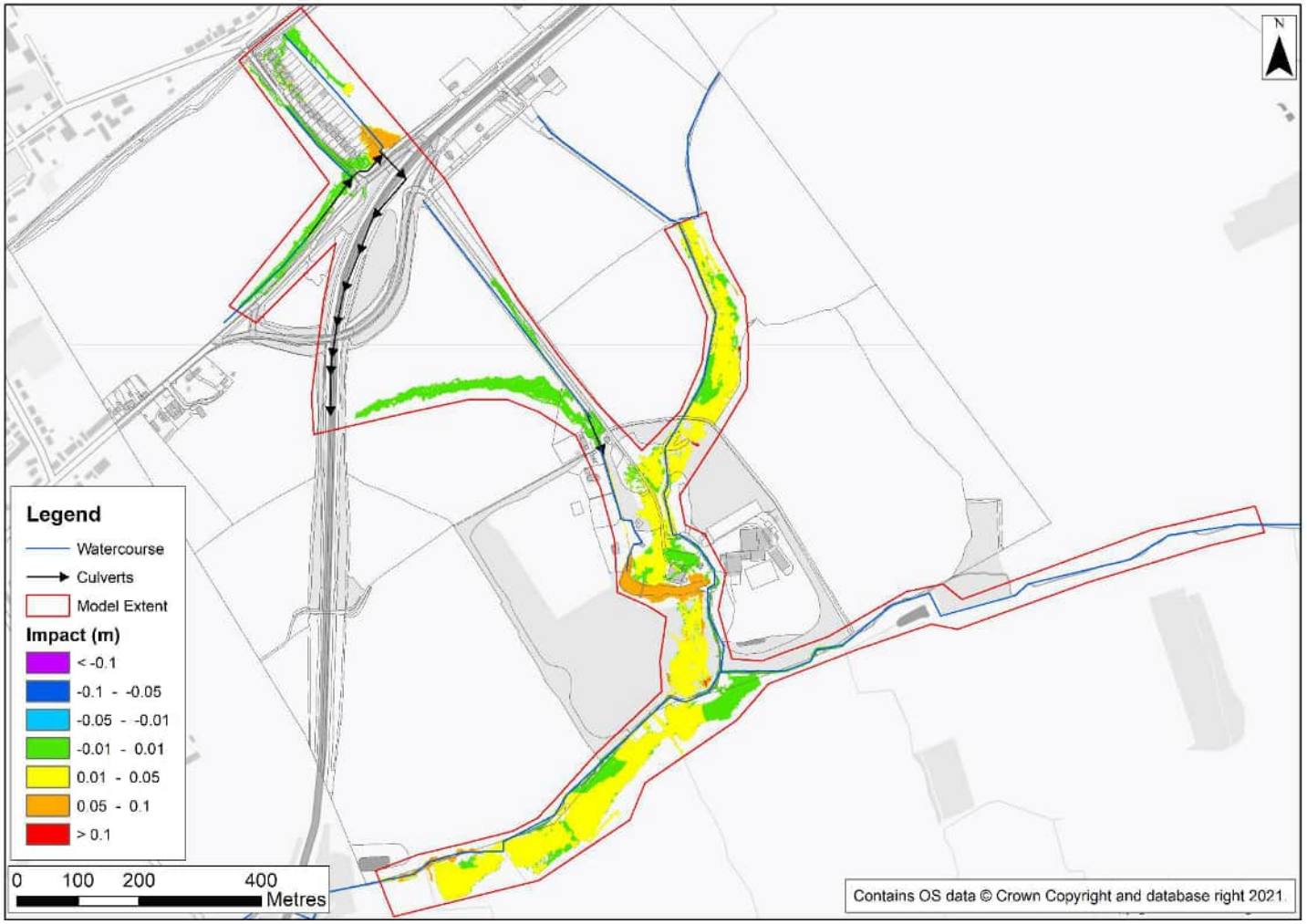


Plate B.4 Water level difference map for the increased inflow sensitivity test



Appendix C – With-Scheme Flood Mapping

Plate C.1 5% AEP maximum flood extent for baseline and with-scheme scenarios

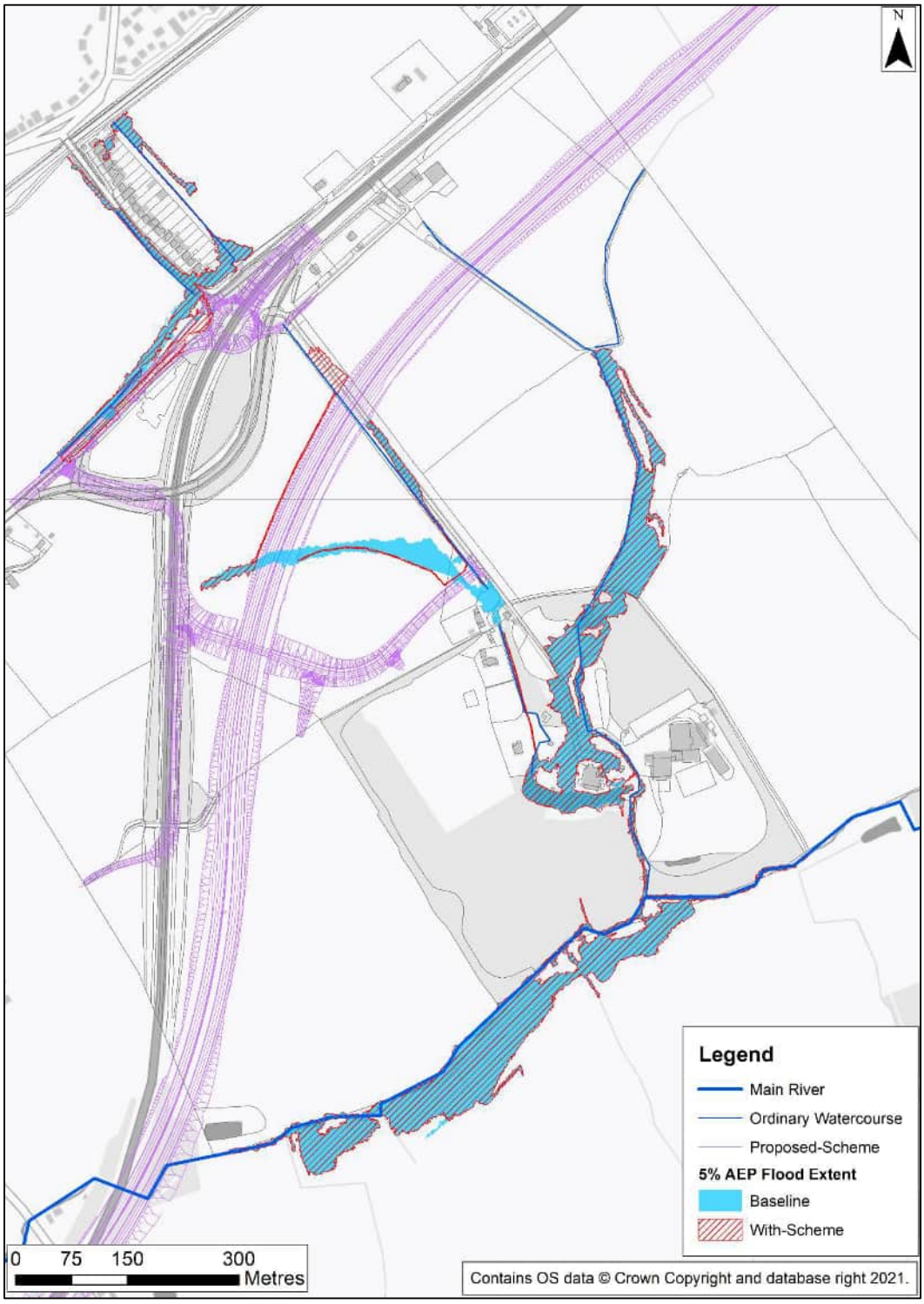


Plate C.2 1% AEP maximum flood extent for baseline and with-scheme scenarios

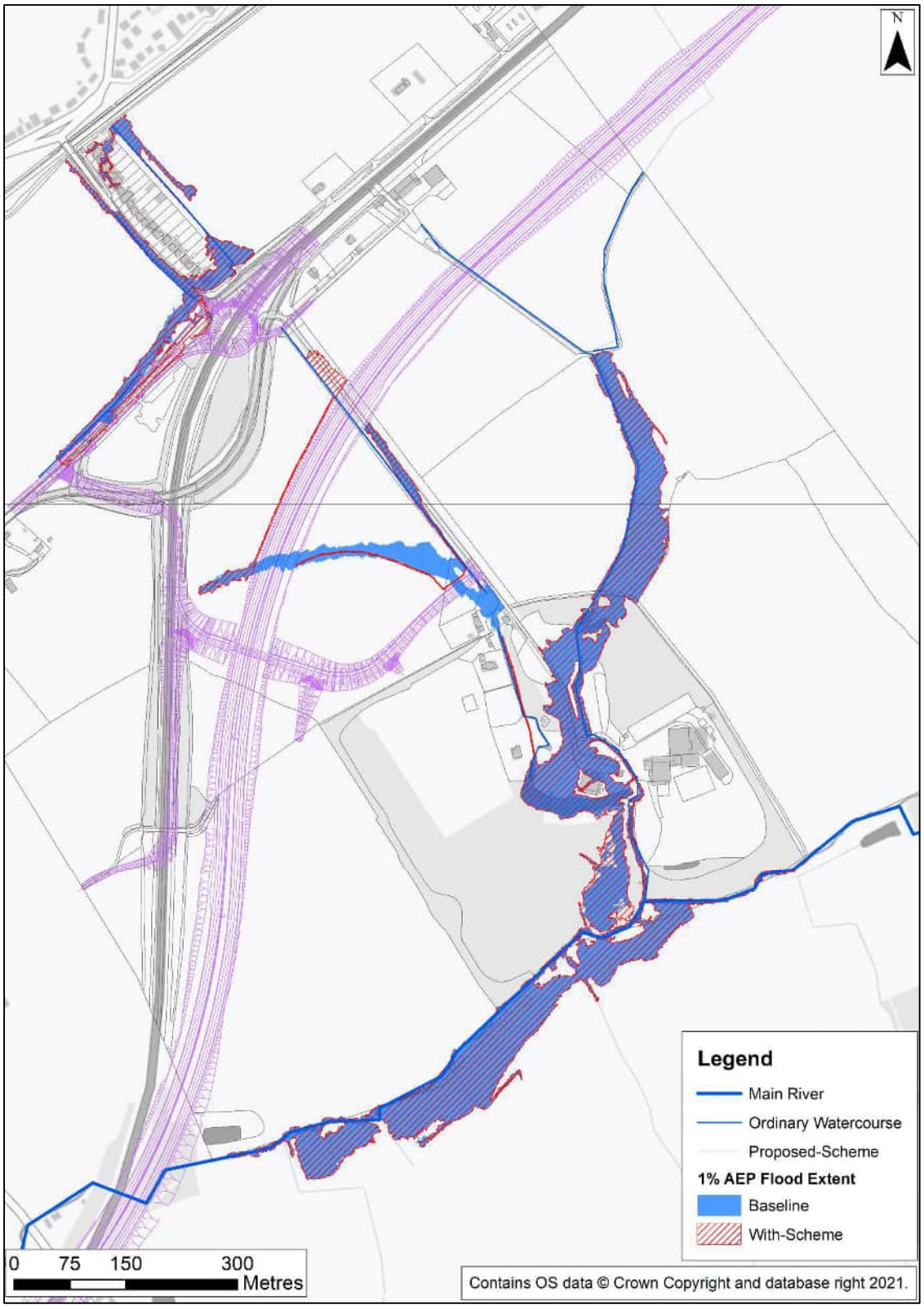


Plate C.3 1% AEP +40%CC maximum flood extent for baseline and with-scheme scenarios

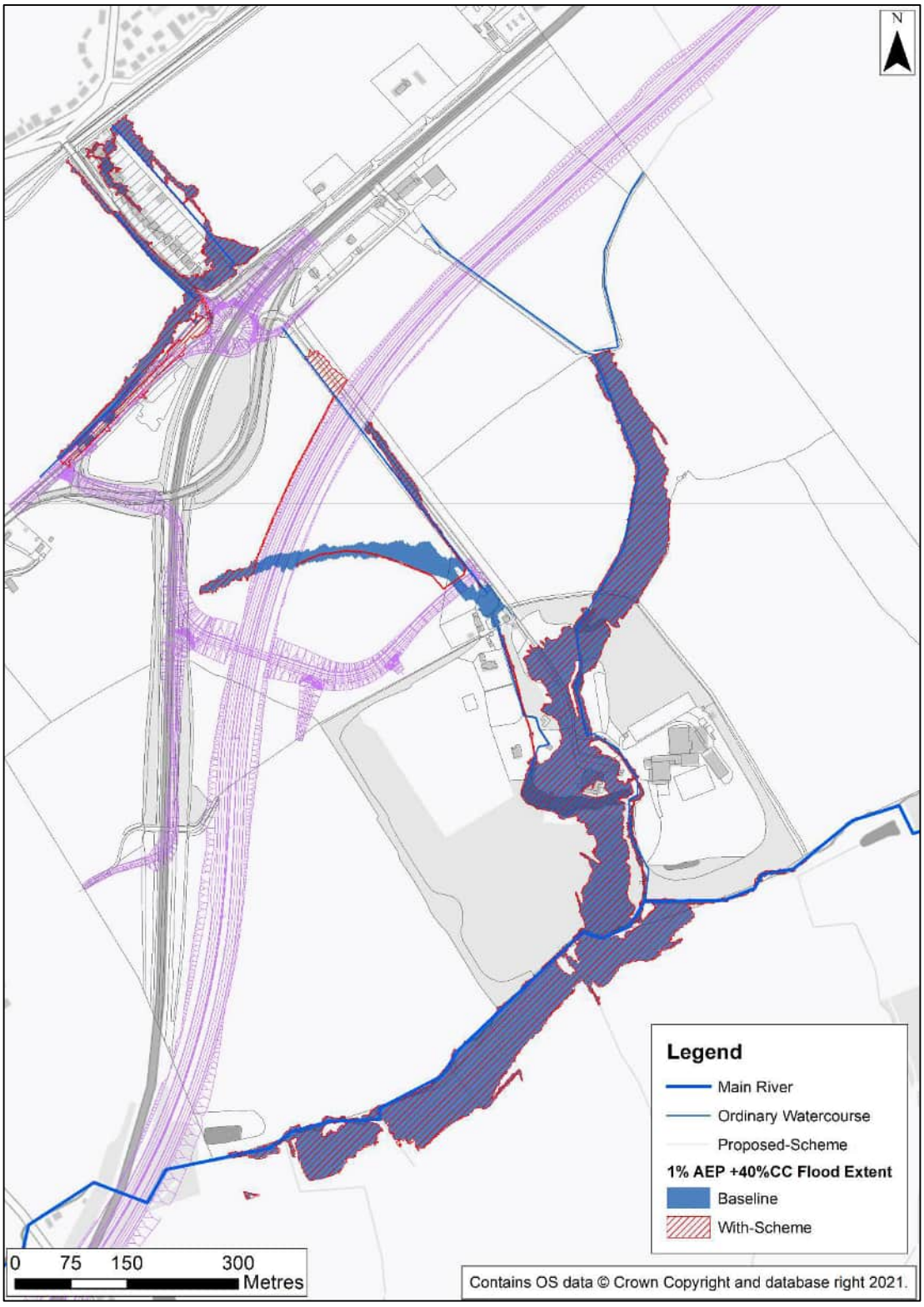


Plate C.4 Water level difference (with-scheme minus baseline) map for the 5% (1 in 20) AEP event

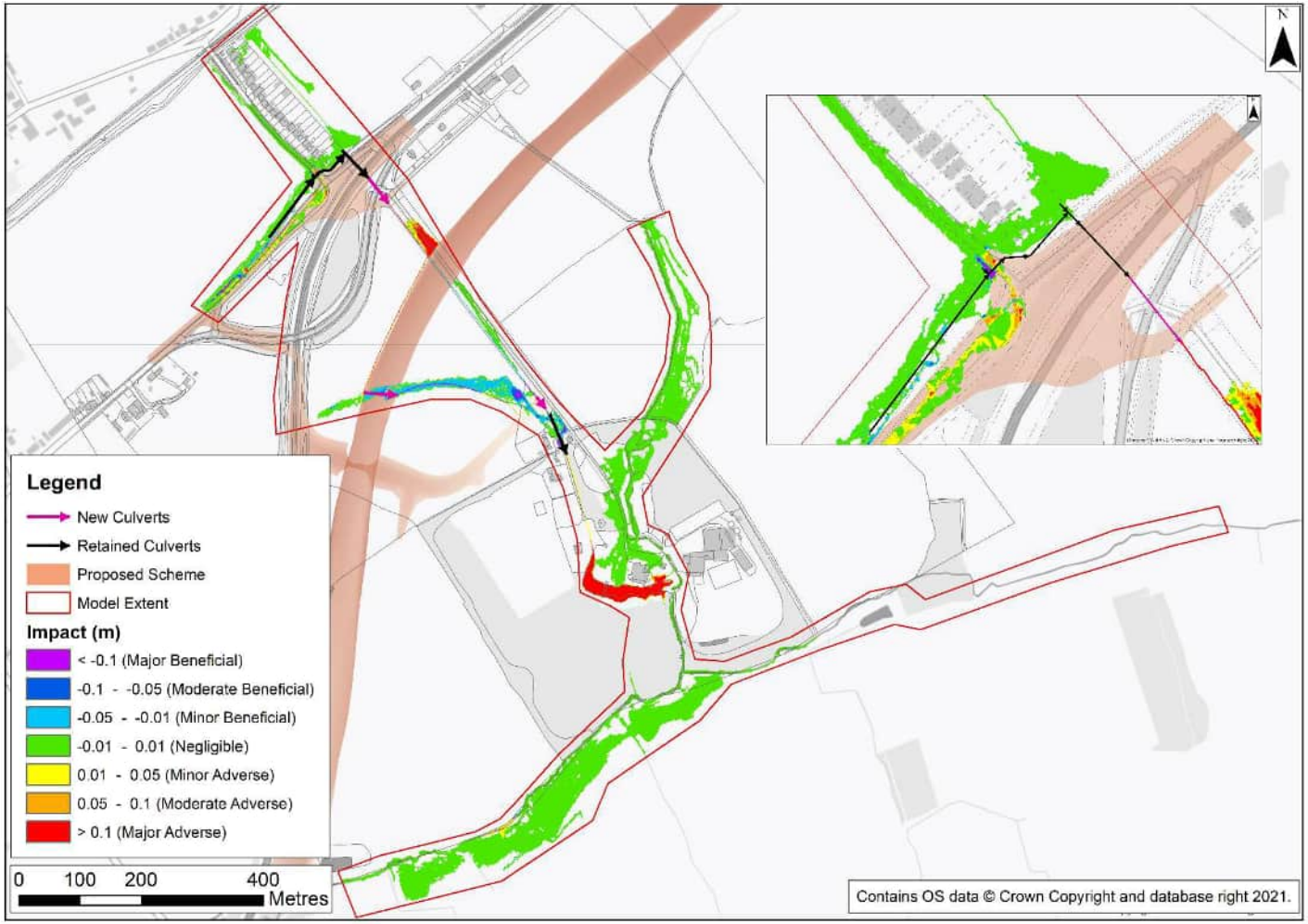


Plate C.5 Water level difference (with-scheme minus baseline) map for the 1% (1 in 100) AEP event

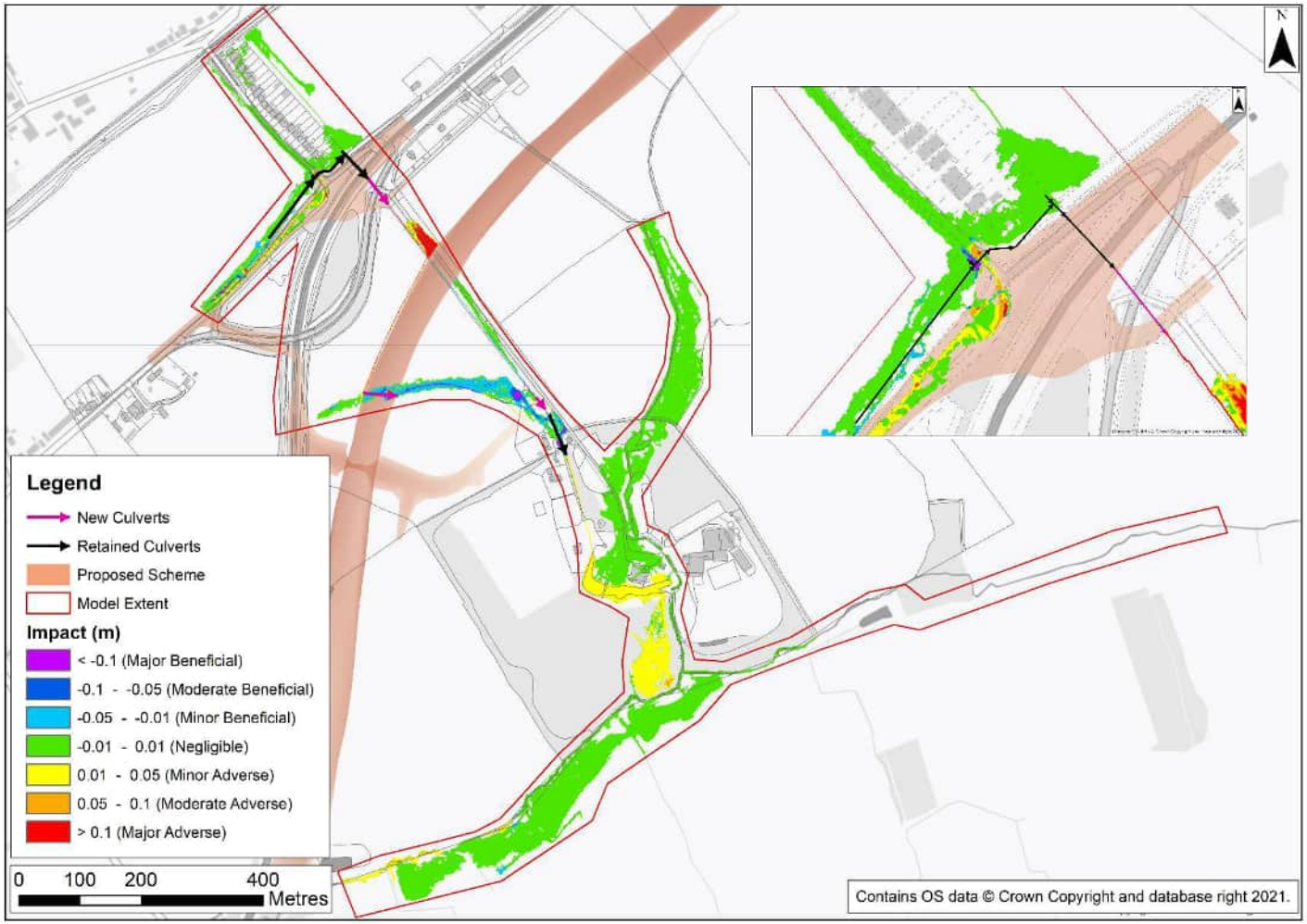
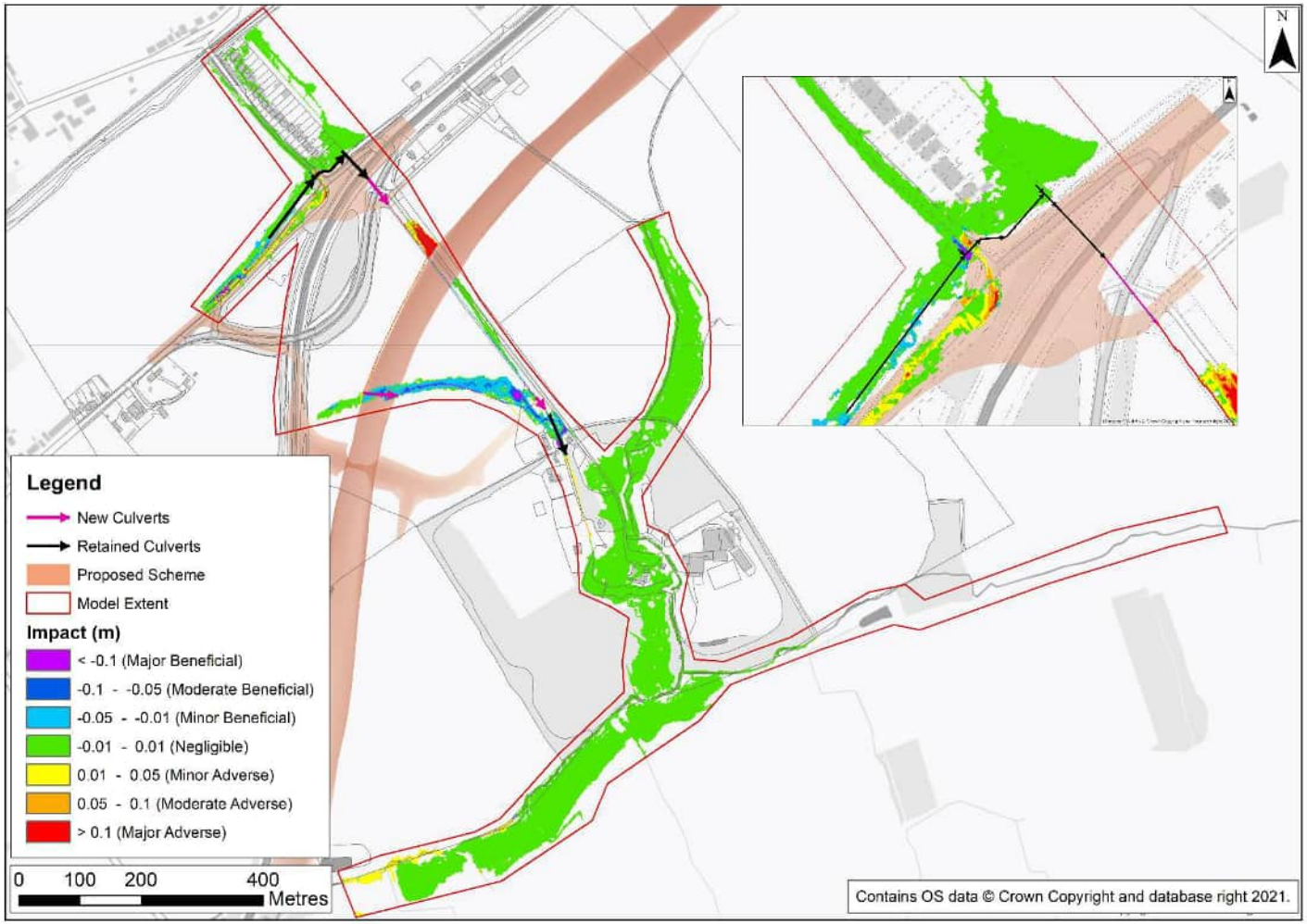


Plate C.6 Water level difference (with-scheme minus baseline) map for the 1% (1 in 100) AEP +40%CC event



Appendix D – Mitigation Flood Mapping

Plate D.1 5% AEP maximum flood extent for baseline and mitigation scenarios

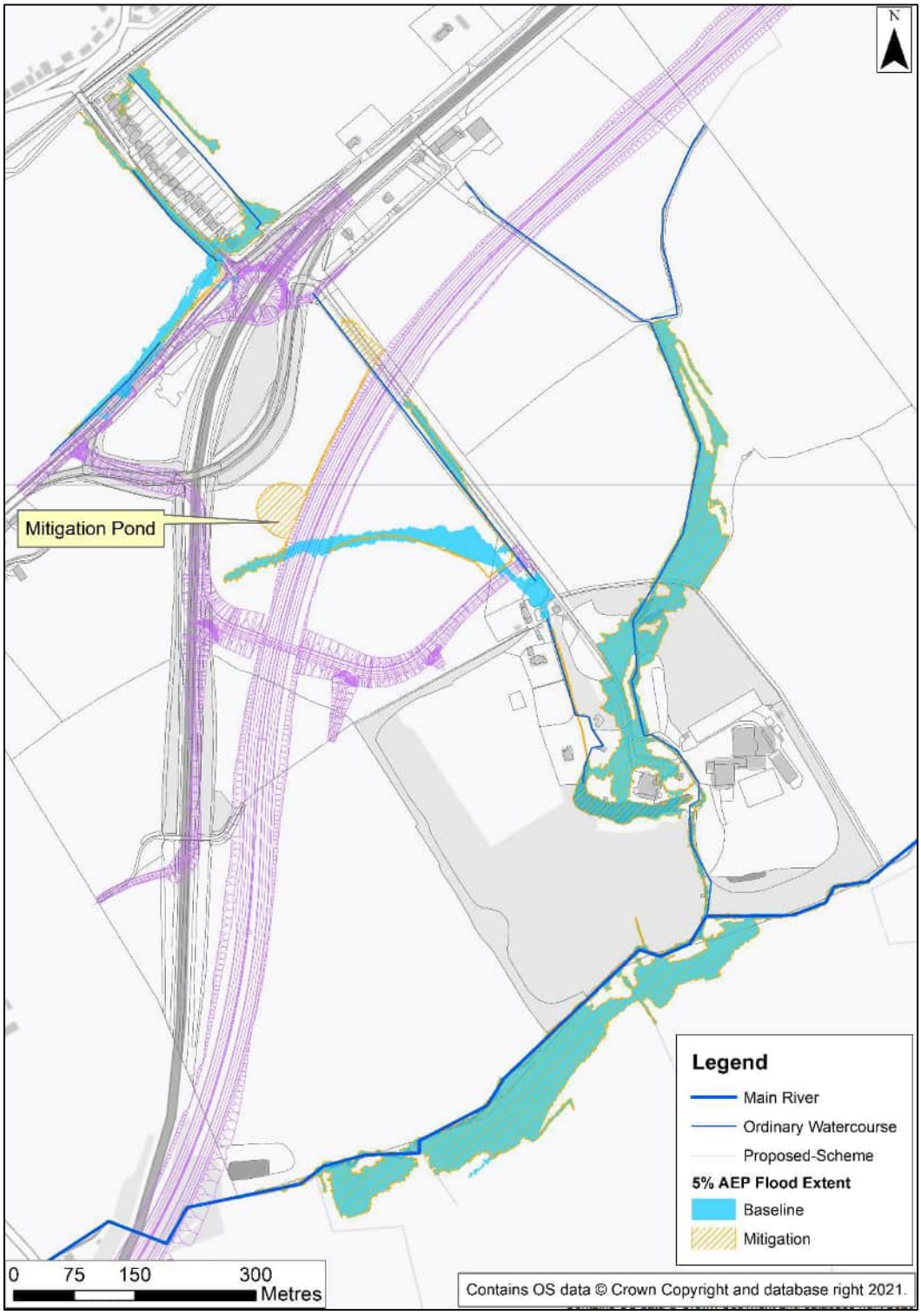


Plate D.2 1% AEP maximum flood extent for baseline and mitigation scenarios

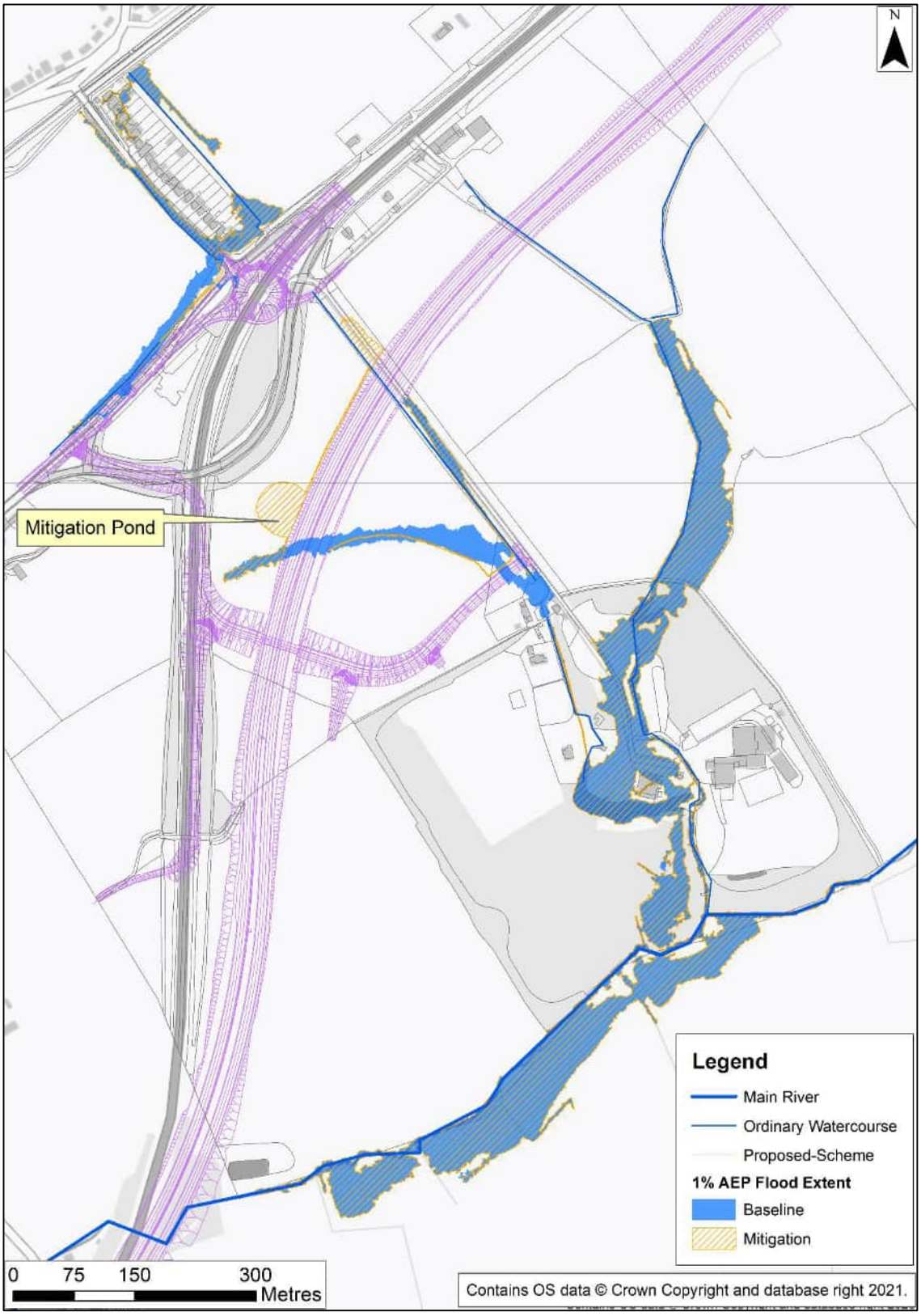


Plate D.3 1% AEP +40%CC maximum flood extent for baseline and mitigation scenarios

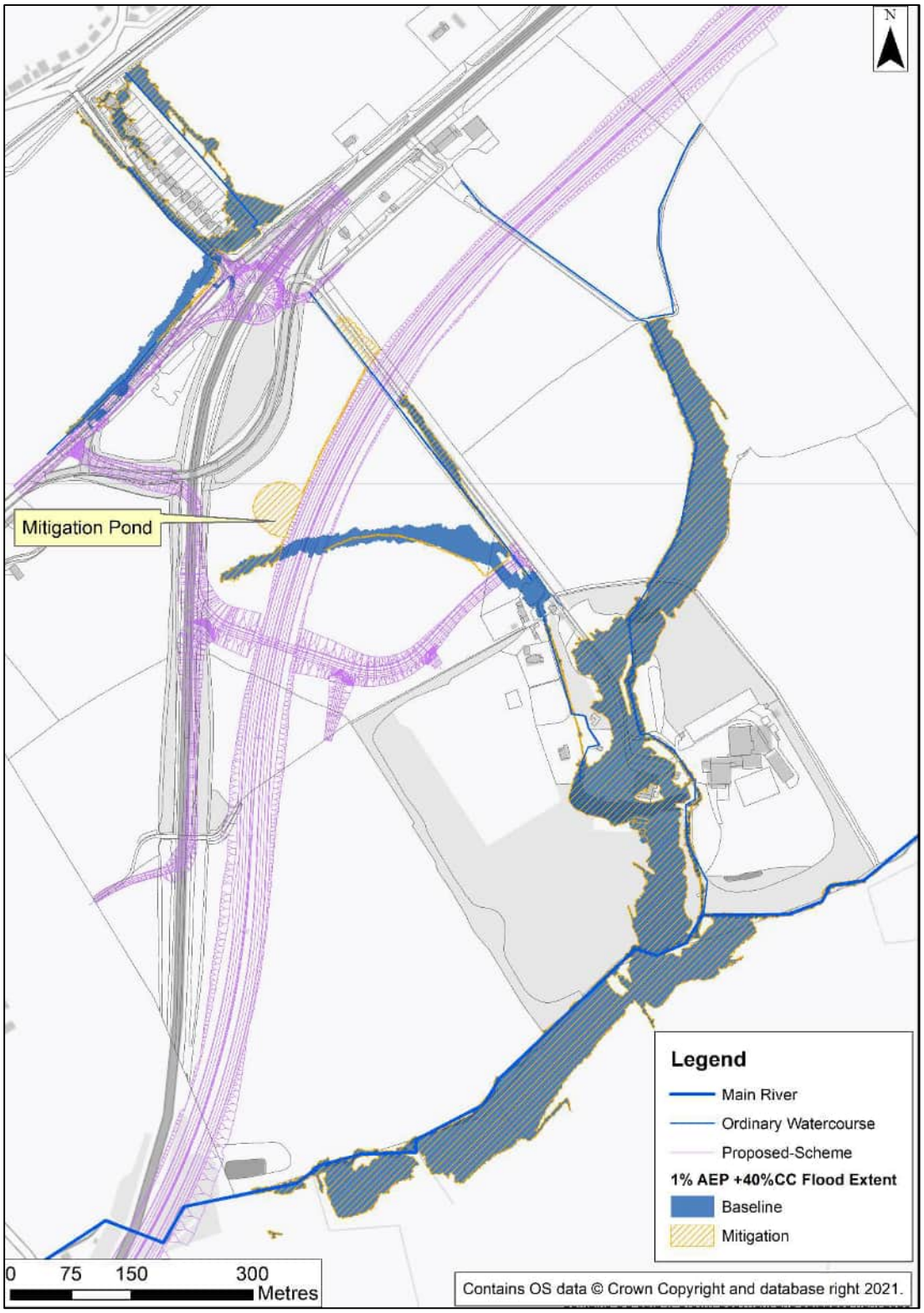


Plate D.4 Water level difference (mitigation minus baseline) map for the 5% (1 in 20) AEP event

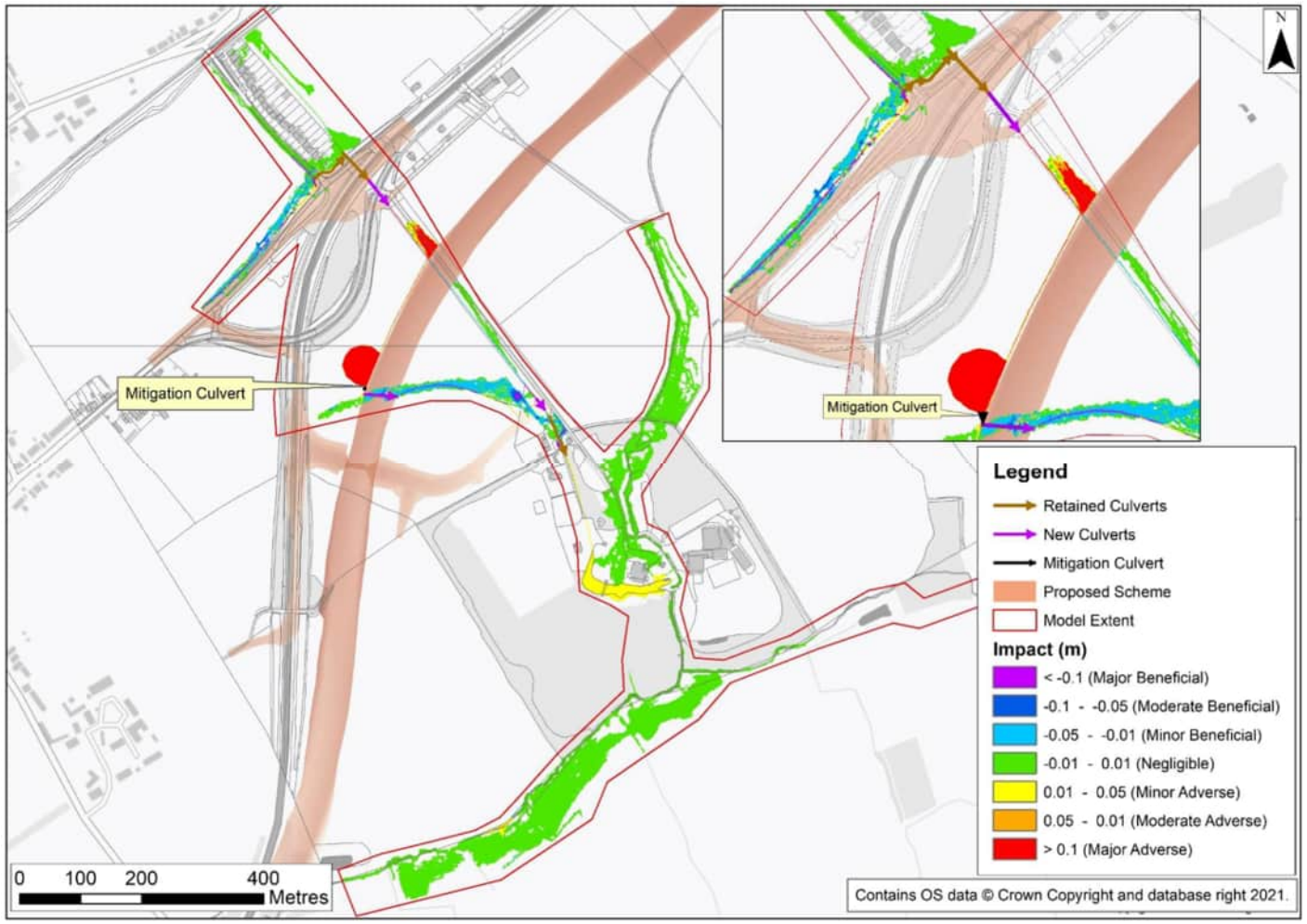


Plate D.5 Water level difference (mitigation minus baseline) map for the 1% (1 in 100) AEP event

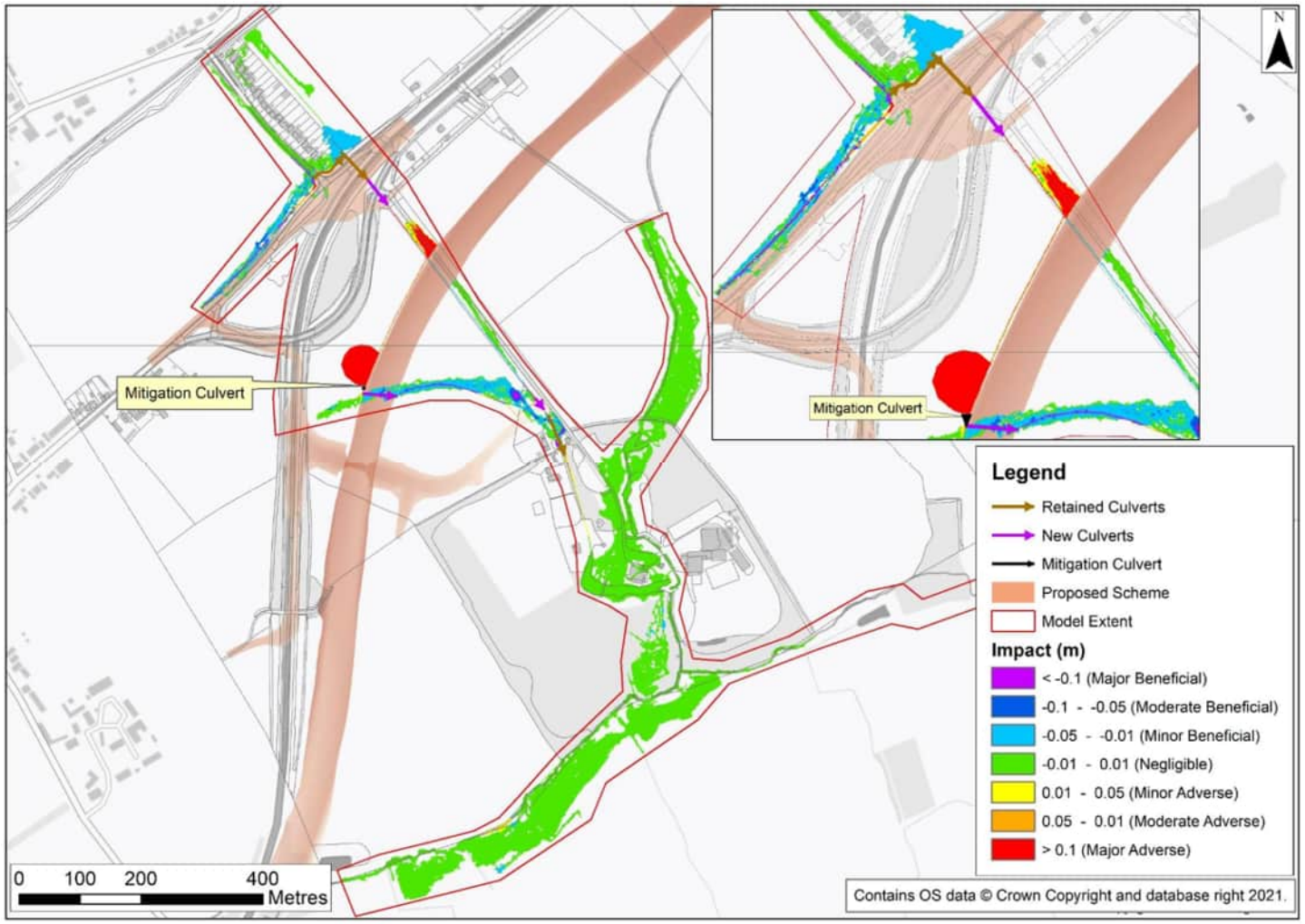
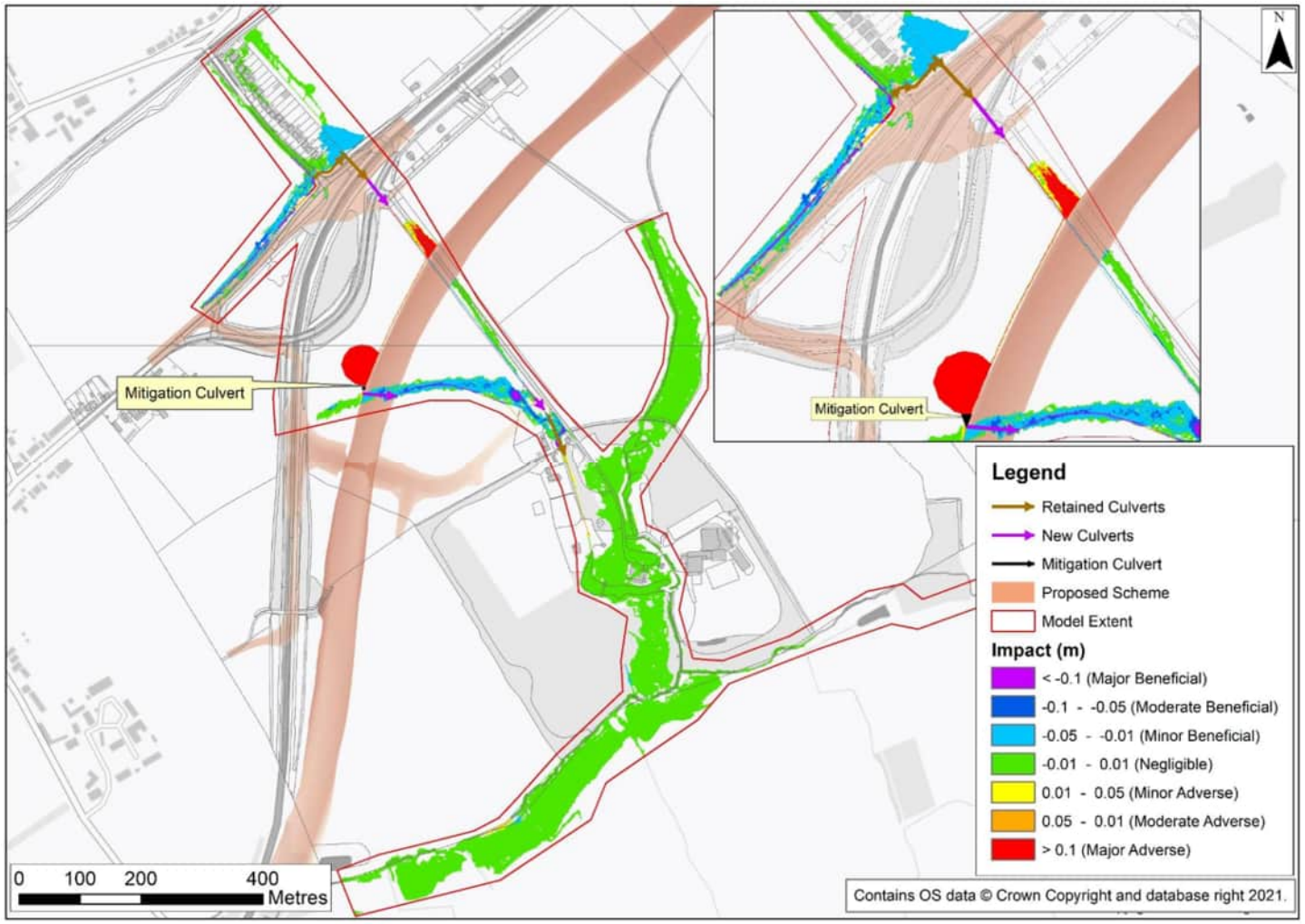


Plate D.6 Water level difference (mitigation minus baseline) map for the 1% (1 in 100) AEP +40%CC event



A12 Chelmsford to A120 widening scheme

TR010060

6.3 ENVIRONMENTAL STATEMENT

FLOOD RISK ASSESSMENT WATERCOURSE 23 HYDROLOGY ANNEX

APFP Regulation 5(2)(e)

Planning Act 2008

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

Volume 6

August 2022

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Table 3.1 Annual exceedance probability (AEP) and related return period reference table.6

1 Executive Summary

This document has been prepared to support the A12 Minor Watercourse 23 with scheme modelling report.

2 Introduction

2.1.1 This document is to provide a record of the calculations and decisions made during flood estimation for the Minor Watercourse 23 (MWC23) catchment which crosses the A12. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future.

3 Summary of assessment

3.1 Summary

3.1.1 A summary of the key information contained within the detailed hydrological assessment of MWC23 is provided in the table below.

Catchment location	Minor watercourse 23 (MWC23) is located in South East England, just outside Ferring and crosses the current A12 at approximately NGR: TL 87866 20275. This minor watercourse flows in a south-easterly direction before discharges to the Domsey Brook approximately 1.1km downstream of the current A12 crossing.
Purpose of study and scope	The A12 is proposed to be upgraded and therefore peak flows and inflow hydrographs are required for the numerical hydraulic modelling of this watercourse. Flows have been derived for the following Annual Exceedance Probability (AEP) events: 5%, 1% and 1% plus climate change (plus cc), equivalent to the 20-year, 100-year and 100-year plus cc return periods events respectively.
Key catchment features	The minor watercourse has a rural catchment with predominantly agricultural land use. There is a solar farm to the western side of catchment 2.
Flooding mechanisms	Peak flows / volumes.
Gauged / ungauged	Ungauged.
Final choice of method	ReFH2.3
Key limitations / uncertainties in results	Small ungauged watercourse therefore flow estimation is open to greater uncertainty.

3.2 Note on flood frequencies

3.2.1 The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

3.2.2 Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval.

3.2.3 The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Table 3.1 Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

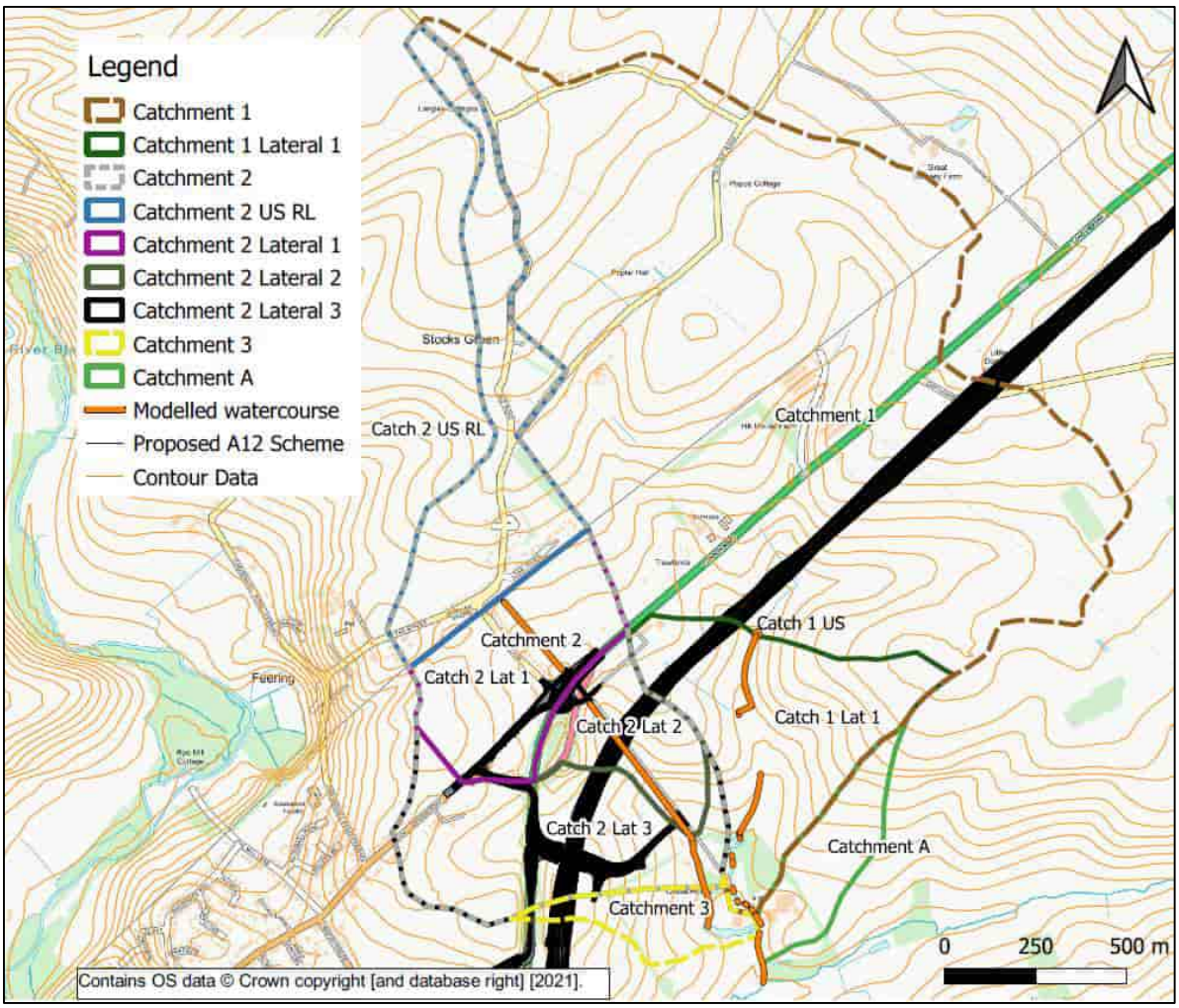
4 Method statement

4.1 Requirements for flood estimates

Overview	The proposed A12 upgrade route crosses the Minor Watercourse 23 (MWC23) and therefore peak flows and hydrographs are required for the hydraulic modelling of this watercourse. Design peak flows and hydrographs are required for the following AEP events: 5%, 1% and 1% plus climate change (plus cc). This is equivalent to the 20-year, 100-year and 100-year plus cc return periods). A climate change uplift factor of plus 40% will be applied for the 1% AEP (100-year return period) event. This is based on current guidance[1] (peak rainfall allowance, upper end, 2080s scenario). MWC23 is a small and ungauged watercourse. For hydraulic modelling purposes four flow estimation points have been adopted; one for Catchment 1 (eastern tributary), one for Catchment 2 (western watercourse), one for Catchment 3 (lower western tributary and one for Catchment A (lower eastern tributary).
Project scope	This is a relatively straightforward hydrological study for one minor watercourse with a catchment area up to the model extent of approximately 3.36km^2. ReFH model parameter calibration for this ungauged catchment is outwith the scope of this project.

[1] Environment Agency (2016). Flood risk assessments: climate change allowances (updated 2020).

4.2 The catchment



Description	As shown in the above figure, Minor Watercourse 23 has two main branches upstream of the A12, namely, Catchment 1 (to the east) and Catchment 2 (to the west). These tributaries combine downstream of the A12 crossing, just upstream of the watercourse's confluence with the Domsey Brook. Catchment 3 and Catchment A further flow into MWC23 upstream of the Domsey Brook confluence. The catchment is classified as essentially rural with a few dwellings present throughout the catchment and a series of local roads and the A12 cutting across the catchment. Land use is predominantly agricultural. There is also a solar farm to the western side of Catchment 2. The soils within the catchment were assessed using the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983). The soil types within the catchment were found by this

	assessment to be composed of predominantly soil type 411d (Hanslope) with a small area of 571s (Efford 1) soils to the western side of the catchment. The soils within the catchment were also assessed using Soilscapes[2], which indicates that the soils are predominantly lime-rich loamy and clayey soils with impeded drainage with a small area to the west of the catchment being composed of freely draining slightly acid loamy soils. According to the Geology of Britain Viewer[3] the geology of the catchment is composed of London Clay Formations consisting of clay, silt and sand. The superficial deposits were noted as being composed of Lowestoft Formation (Diamicton).

4.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released 24th September 2020. This contains provisional data up to water year 2019-20.

4.4 Gauging stations (flow or level)

Watercourse	Station name	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
River Blackwater	Blackwater @ Appleford Bridge	37010	247.3	Double throated trapezoidal flume and high flow rated spillway.	1962 to present
River Blackwater	Blackwater @ Stisted	37017	139.2	Post 1979 flow is derived through the rating curve. Under non-modular conditions a macro is applied within WISKI which applies a reduction factor.	1969 to present

[2] http://www.landis.org.uk/soilscapes/

[3] geology from the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

Watercourse	Station name	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
Roman River	Roman River @ Bounstead Bridge	37021	52.6	'Essex' profile weir. One rating applied across period of record.	1965 to August 1991
River Ter	Ter @ Crabbs Bridge	37003	77.8	Theoretically rated	1932 to present

4.5 Data available at each flow gauging station

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED ?	OK for pooling ?	Data quality check needed ?	Other comments on station and flow data quality
Blackwater @ Appleford Bridge	1962 to present	N	Yes	Yes	N	Groundwater influenced regime. Intermittently affected by the Ely / Ouse transfer scheme.
Blackwater @ Stisted	1969 to present	N	Yes	No	N	Predominantly Chalk catchment affected by artificial influences including urban runoff and Ely Ouse/Essex scheme. No POT data are presented for this station as they have no non-modular flow correction applied. AMAX data have been corrected, but in some cases can be below the daily flow as these are not corrected.
Roman River @ Bounstead Bridge	N/A	N	No	No	N	No longer appearing as suitable for QMED. Groundwater influenced regime. Data since 31/08/1991 not stored on NRFA due to continuing problems with drowning.

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED ?	OK for pooling ?	Data quality check needed ?	Other comments on station and flow data quality
Ter @ Crabbs Bridge	1963 to present	N	Yes	No	N	Recently removed from suitable for pooling category. This was noted as due to many AMAX events being out of bank and this is not accounted for in rating.

4.6 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings	N/A			Not required for this assessment.
Historical flood data	N/A			Not Available
Flow or river level data for events	N/A			Not available for this assessment.
Rainfall data for events	N/A			Not required for this assessment.
Potential evaporation data	N/A			Not required for this assessment.
Results from previous studies	N/A			Not available for this assessment.
Other data or information	N/A			Not available for this assessment.

4.7 Hydrological understanding of catchment

Conceptual model	Hydraulic modelling of MWC23 is required to assess potential impacts of upgrading the A12. The target location is the proposed A12 crossing of MWC23. Flooding is likely to be caused by peak flows/volumes at the A12 crossing location.
Unusual catchment features	The catchment is a small rural catchment and therefore flow estimation is open to greater uncertainties than for a larger gauged catchment.

4.8 Initial choice of approach

Is FEH appropriate?	Yes, FEH methods are appropriate.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	ReFH2.3 and FEH Statistical methods have been applied to derive the peak flow estimates and hydrographs for this catchment. The hydrographs have been derived using ReFH2.3 method. Flow estimates have been derived for four flow estimation points: Catchment 1, Catchment 2, Catchment 3, and Catchment A. This flow has been area scaled within the numerical hydraulic model for Catchment 1 and Catchment 2 so flows can be applied at the start of the modelled reaches with lateral flows being applied for key reaches downstream. For Catchment 3 and Catchment A the flows have been applied directly within the model as lateral inflows.
Software to be used (with version numbers)	FEH Web Service[4] / WINFAP 4[5] / ReFH2.3

4.9 Locations where flood estimates required

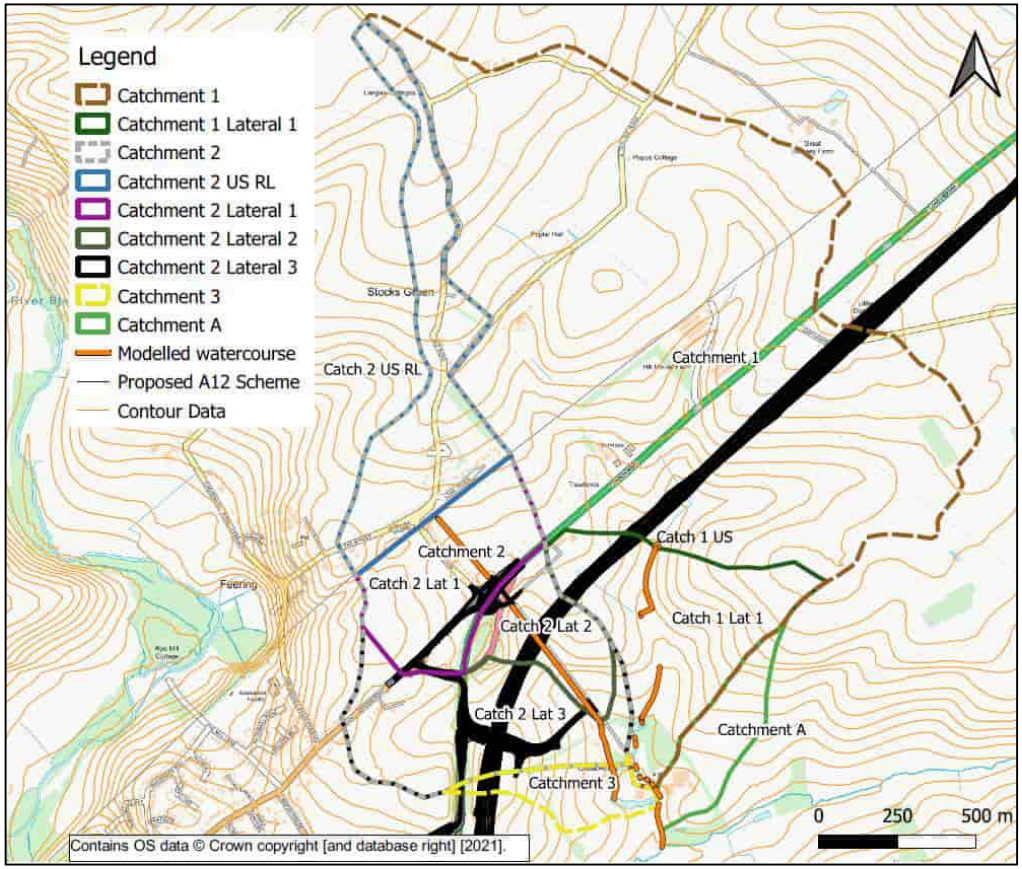
4.9.1 The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

[4] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

[5] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.10 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Catchment 1	L1*	MWC23	Eastern catchment.	588358	219647	1.75	2.253
Catch 1 US	S11	MWC23	Upstream eastern catchment.	588364	220425	1.64	1.932
Catch 1 Lat 1	S12	MWC23	Residual downstream eastern catchment.	588359	220087	-	0.321
Catchment 2	L2**	MWC23	Western catchment.	588272	219751	-	0.936
Catch 2 US RL	S21	MWC23	Upstream western catchment (upstream of the railway line).	587653	220591	-	0.298
Catch 2 Lat 1	S22	MWC23	Western catchment lateral inflow between railway line and current A12.	587777	220361	-	0.220
Catch 2 Lat 2	S23	MWC23	Western catchment lateral inflow between current A12 and Catch 2 Lat 3.	588004	220101	-	0.128
Catch 2 Lat 3	S24	MWC23	Residual downstream western catchment.	588196	219793	-	0.290
Catchment 3	L	MWC23	Lower western catchment.	588156	219610	-	0.076
Catchment A	L	MWC23	Lower eastern catchment.	588448	219650	-	0.099
*L1 = S11+S12 **L2 = S21 + S22 + S23 + S24							

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required. Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced. The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.			 *Catchment 2, Catchment 3 and Catchment A are not picked up by the FEH Web Service.					

4.11 Important catchment descriptors at each subject site (incorporating any changes made)

4.11.1 The FEH Web Service does not pick up the MWC23 catchment correctly. Areas of the catchment can, however, be selected using the FEH Web Service. The FEH Web Service catchment descriptors to the Domsey Brook confluence (TL 88300 19550) have been selected to use for the MWC23 catchments. Key FEH catchment descriptors (area, DPSBAR, DPLBAR and URBEXT) were further calculated for the target locations using standard FEH methodologies.

Site code	FARL	PROPWET	BFIHOST	BFIHOST 19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
Catchment 1	1	0.23	0.363	0.347	1.561	9.27	565	0.006	0.181
Catchment 2	1	0.23	0.363	0.347	0.964	8.92	565	0.024	0.181
Catchment 3	1	0.23	0.363	0.347	0.244	15.10	565	0.023	0.181
Catchment A	1	0.23	0.363	0.347	0.282	12.93	565	0.000	0.181

4.12 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	The catchment areas for this watercourse were assessed using contour data (1m and 0.25m) and LIDAR data (1m).
Record how other catchment descriptors were checked and describe any changes.	FEH catchment descriptors at TL 88300 19550 were adopted. Key catchment descriptors such as FARL and soils were checked using mapping data and key catchment descriptors were calculated using standard methods (e.g. area, DPSBAR, DPLBAR, URBEXT).
Source of URBEXT	URBEXT1990 and URBEXT2000 calculated using URBAN50K method.
Method for updating of URBEXT	URBEXT calculated using URBAN50K and therefore updating URBEXT to current year not required.

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	The FEH Statistical method was used as a comparison against ReFH2.3 peak flow estimates.

5.2 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjust-ment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor			
						Weight	Weighted ave. adjustment		
Catchment 1	0.401	CDS	-	-	-	-	-	1.006	0.404
Catchment 2	0.190	CDS	-	-	-	-	-	1.023	0.194
Catchment 3	0.015	CDS	-	-	-	-	-	1.022	0.016
Catchment A	0.018	CDS	-	-	-	-	-	1.000	0.018

Are the values of QMED spatially consistent?	N/A
Method used for urban adjustment for subject and donor sites	Not required as URBAN50K method used to derive URBEXT.

Parameters used for WINFAP v4 urban adjustment if applicable		
Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010) in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016).

5.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Donor gauges were assessed for the purposes of QMED adjustment; however, no donor was selected. The River Blackwater @ Appleford Bridge gauge (37010) was assessed as it is the closest gauge, however, the River Blackwater @ Appleford Bridge gauges a much larger catchment with groundwater influence in the upper reaches of the catchment. This gauge was therefore assessed as not suitable as a donor for this small ungauged catchment. The River Ter @ Crabbs Bridge (37003) was also assessed as a donor for QMED adjustment, but this gauge was found to have varying FEH cds from the target site. The River Blackwater @ Stisted (37017) was further assessed as a donor for QMED adjustment but the gauge was found to not be appropriate due to varying geology / FEH catchments descriptors to the target catchment. No gauge(s) were therefore assessed as suitable donors for QMED adjustment for the target site.

5.4 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)

5.5 Derivation of pooling groups

5.5.1 Given the target watercourse is not picked up correctly by the FEH Web Service and given the similarity in FEH catchment descriptors between the target catchments and a local donor (donor catchment TL 86150 18150), it was decided to use the local donor growth curve for MWC23. This is the same growth curve used for MWC21A.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
MWC21 Donor 1 Final Pooling Group	Donor	No	Removed: 27073 / 26016 (due to high BFIHOST), 47022 / 106002 (due to low FARL) and 49005 (due to relatively high BFIHOST / high discordance). Added 206006 / 36010	L-CV: 0.226 L-Skew: 0.268

Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).

5.6 Derivation of flood growth curves at subject sites

Site code	Method	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
Donor	P	Donor 1 Final Pooling Group	GL: distribution gives an acceptable fit.	Urban Adjustment	Location: 1.000 Scale: 0.221 Shape: -0.268	3.003

Notes
Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis
Urban adjustments are all carried out using the method of Kjeldsen (2010).
Growth curves were derived using the procedures from Science Report SC050050 (2008).

5.7 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	25	50	100	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events									
	50	20	10	5	4	2	1	0.5	0.2	0.1
Catchment 1	0.404	0.554	0.671	0.804	0.852	1.02	1.21	1.45	1.83	2.19
Catchment 2	0.194	0.267	0.323	0.387	0.410	0.489	0.584	0.697	0.882	1.05
Catchment 3	0.016	0.022	0.026	0.031	0.033	0.040	0.047	0.057	0.072	0.086
Catchment A	0.018	0.024	0.029	0.035	0.037	0.045	0.053	0.063	0.080	0.096

6 Revitalised flood hydrograph 2 (ReFH2) method

6.1 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 has been applied to derive both peak flow estimates and hydrographs for the target watercourse at key locations for input into a hydraulic model.

6.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp}	BL (hours)	BR
Catchment 1	ReFH2.3	6.5	-	298.08	-	35.18	1.1
Catchment 2	ReFH2.3	5.0	-	298.08	-	31.67	1.11
Catchment 3	ReFH2.3	1.93	-	298.08	-	23.47	1.14
Catchment A	ReFH2.3	2.2	-	298.08	-	24.22	1.14

Brief description of any flood event analysis carried out	

Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)

6.3 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
Catchment 1	Rural	Winter	11.00
Catchment 2	Rural	Winter	7.50
Catchment 3	Rural	Winter	3.25
Catchment A	Rural	Winter	3.50

6.4 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm

6.5 Flood estimates from the ReFH2 method

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	50	100	100 + CC 40% Uplift	200	500	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	5	4	2	1	1 + CC 40% Uplift	0.5	0.2	0.1
Catchment 1	0.550	0.738	0.864	0.990	1.03	1.18	1.37	1.97	1.67	2.25	2.73
Catchment 2	0.273	0.371	0.437	0.503	0.525	0.601	0.698	1.01	0.844	1.14	1.40
Catchment 3	0.037	0.052	0.063	0.074	0.077	0.089	0.102	0.148	0.121	0.161	0.199
Catchment A	0.044	0.061	0.074	0.086	0.090	0.103	0.119	0.172	0.142	0.188	0.234

**Peak flows derived using individual catchment critical storm duration, areal reduction factor and seasonal correction factor.*

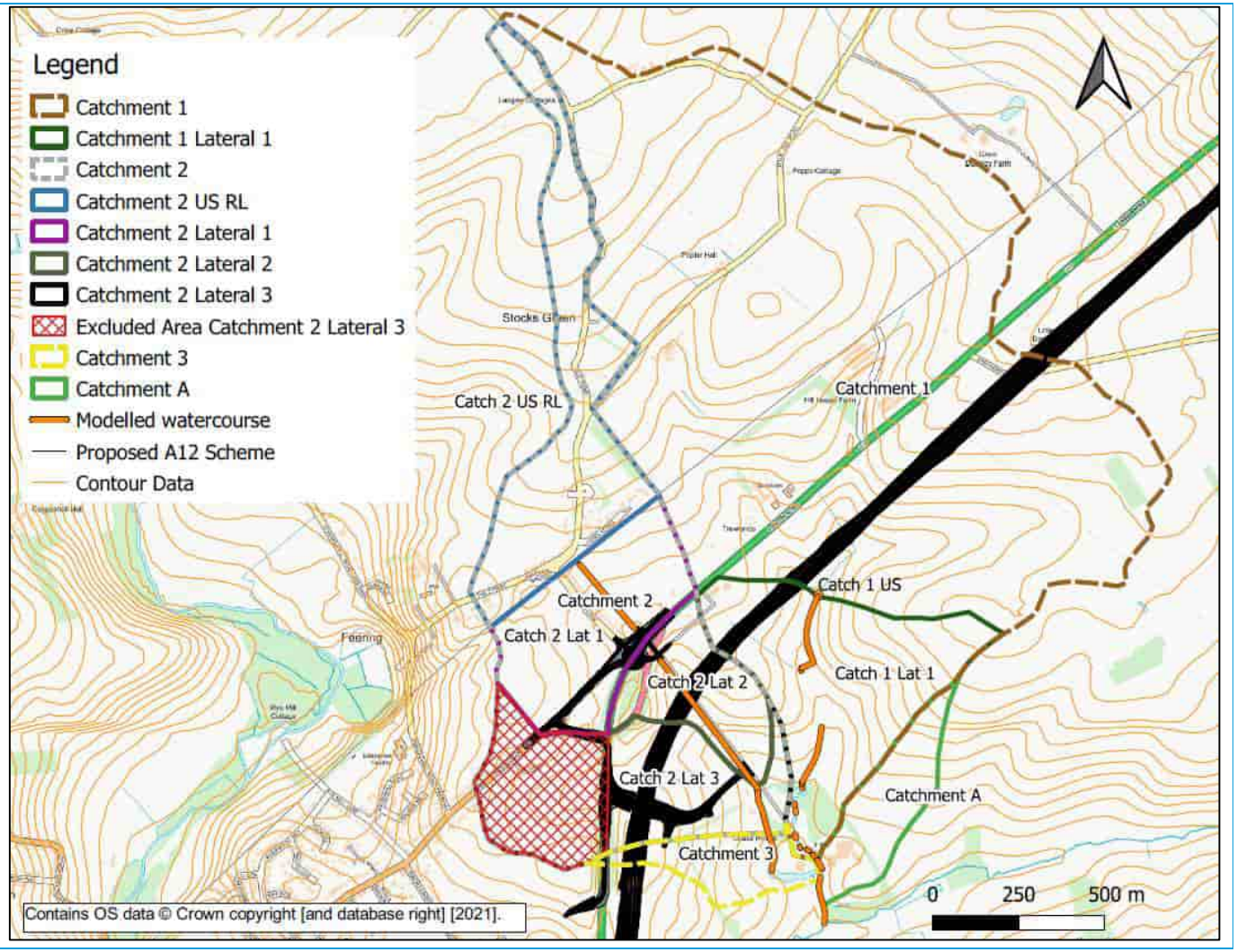
7 Discussion and summary of results

7.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak			
	Return period 20-years / 5% AEP		Return period 100-years / 1% AEP	
	FEH Statistical	ReFH2.3	FEH Statistical	ReFH2.3
Catchment 1	0.804	0.990	1.21	1.37
Catchment 2	0.387	0.503	0.584	0.698
Catchment 3	0.031	0.074	0.047	0.102
Catchment A	0.035	0.086	0.053	0.119

7.2 Final choice of method

Choice of method and reasons	The final choice of method is ReFH2.3 as it incorporates updates to BFIHOST19 and uses latest rainfall data (FEH13).
How will the flows be applied to a hydraulic model?	The ReFH2.3 design hydrographs will be applied in the hydraulic model for Catchment 1, Catchment 2, Catchment 3, and Catchment A. This flow will be area scaled within the hydraulic model for Catchment 1 and Catchment 2 so flows can be applied at the start of the modelled reaches with lateral flows being applied for key reaches downstream. For Catchment 1, the total flow will be applied to the numerical hydraulic model as following: 86% of the total will be applied at the start of the model, and 14% of the total will be applied as a lateral inflow between the start of the model and Catchment A. For Catchment 2, the total inflow will be applied to the numerical hydraulic model as following: 31.9% of the total will be applied at the start of the model upstream of the railway (Catch 2 US RL), 23.5% of the total will be applied as a lateral inflow between the railway and current A12 (Catch 2 Lat 1), 13.6% of the total will be applied as a lateral inflow between the A12 and Catch 2 Lat 3 (Catch 2 Lat 2) and 16.2% of the total will be applied as a lateral inflow between Catch 2 Lat 2 and Catchment 3 (Catch 2 Lat 3 area). *It should be noted that the total distributed catchment flow of Catchment 2 to be applied to the model is only 85.2%* (i.e., 31.9+23.5+13.6+16.2) *of the total flow. This is because, following the hydrological analysis the road drainage team confirmed that an area in the upper catchment is currently redirected towards the Domsey Brook and therefore 14.8% of the total inflows are discharging out of the MWC23 catchment (see the excluded area in the figure below). The Catch 2 Lat 3 area upstream of the A12 will therefore be excluded from the modelled inflows.* For Catchment 3 and Catchment A the flows will be applied directly within the model as lateral inflows.


Legend
Catchment 1
Catchment 1 Lateral 1
Catchment 2
Catchment 2 US RL
Catchment 2 Lateral 1
Catchment 2 Lateral 2
Catchment 2 Lateral 3
Excluded Area Catchment 2 Lateral 3
Catchment 3
Catchment A
Modelled watercourse
Proposed A12 Scheme
Contour Data
Catch 2 US RL
Catchment 1
Catch 1 US
Catchment 2
Catch 2 Lat 1
Catch 2 Lat 2
Catch 1 Lat 1
Catch 2 Lat 3
Catchment A
Catchment 3
Feering
Contains OS data © Crown copyright [and database right] [2021].
0
250
500 m

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are as follows: • ReFH2.3 design peak flows / hydrographs are most appropriate for the target catchment. • MWC21A FEH Statistical growth curve is appropriate to use to derive design peak flows for MWC23 as a comparison.
Discuss any particular limitations	Small ungauged catchment.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	The ReFH2.3 method has been used to derive design peak flows and hydrographs. This method is appropriate but there is inherent uncertainty regarding using default model parameters. The standard approach in this situation is to use default model parameters. If the catchment was gauged the ReFH parameters could be adjusted to reduce the uncertainty.
Comment on the suitability of the results for future studies	The design peak flows have been estimated for the purposes of the A12 road improvements project only with the target location being the A12 crossing. Future studies should assess the appropriateness of these peak flow estimates / hydrographs for the purposes of their investigation.
Give any other comments on the study	None

7.4 Checks

Are the results consistent, for example at confluences?	N/A
What do the results imply regarding the return periods / frequency of floods during the period of record?	N/A ungauged catchment.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The 1 % AEP growth factor is 3.0. This is within the expected range.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the 0.1% AEP / 1% AEP is 1.81.

How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No other studies were available for this minor watercourse.
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	The peak flow estimates will be checked within the hydraulic model at the downstream model extent.

7.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	**5**	**10**	**20**	**25**	**50**	**100**	**100 + CC 40% Uplift**	**200**	**500**	**1000**
	Flood peak (m^3/s) for the following AEP (%) events										
	50	**20**	**10**	**5**	**4**	**2**	**1**	**1 + CC 40% Uplift**	**0.5**	**0.2**	**0.1**
Catchment 1	0.550	0.738	0.864	0.990	1.03	1.18	1.37	1.97	1.67	2.25	2.73
Catchment 2	0.273	0.371	0.437	0.503	0.525	0.601	0.698	1.01	0.844	1.14	1.40
Catchment 3	0.037	0.052	0.063	0.074	0.077	0.089	0.102	0.148	0.121	0.161	0.199
Catchment A	0.044	0.061	0.074	0.086	0.090	0.103	0.119	0.172	0.142	0.188	0.234

7.6 Uncertainty bounds

7.6.1 The Flood Estimation Guidelines 197_08 (2020) notes that it is more difficult to quantify uncertainty in design flows estimated from the ReFH rainfall-runoff model than for the FEH Statistical method which it reports approximate uncertainty bounds for. It is therefore highlighted that the final flow estimates are open to uncertainty, but uncertainty bounds have not been provided for this assessment.

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hydrographs are required for the hydraulic modelling of MWC23. See: MWC23 ReFH2.3 Hydrographs 1

Acronyms

Abbreviation	Term
AEP	Annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

References

British Geological Survey. The Geology of Britain Viewer accessed 2021 at: [redacted]

Centre for Ecology and Hydrology. Flood Estimation Handbook Web Service accessed 2021 at: [redacted]

Environment Agency (2021). Flood risk assessments: climate change allowances guidance.

Environment Agency (June 2020). Flood Estimation Guidelines, Technical guidance 197_08

Soil Survey of England and Wales (1983). Soil Maps of England and Wales Scale 1:250,000

Annex A

A.1 Pooling Group Details

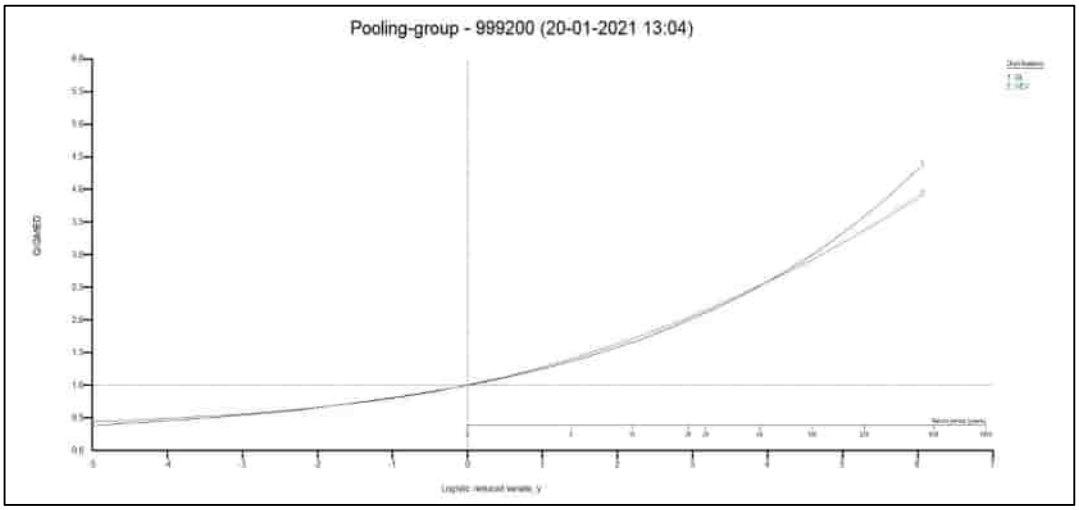
A.1.1 Initial Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.523	42	1.84	0.163	0.301	0.577
27051 (Crimple @ Burn Bridge)	3.278	47	4.524	0.218	0.156	0.357
45816 (Haddeo @ Upton)	3.285	26	3.456	0.3	0.406	0.925
27073 (Brompton Beck @ Snainton Ings)	3.508	39	0.812	0.215	0.035	2.071
28033 (Dove @ Hollinsclough)	3.569	44	4.177	0.228	0.371	0.563
25019 (Leven @ Easby)	4.067	41	5.09	0.342	0.386	1.547
26016 (Gypsey Race @ Kirby Grindalythe)	4.085	22	0.1	0.321	0.266	1.432
91802 (Allt Leachdach @ Intake)	4.107	34	6.35	0.153	0.257	0.677
25011 (Langdon Beck @ Langdon)	4.207	33	15.647	0.232	0.328	0.923
71003 (Croasdale Beck @ Croasdale Flume)	4.225	37	10.9	0.212	0.323	0.233
47022 (Tory Brook @ Newnham Park)	4.227	25	6.176	0.257	0.191	0.559
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	4.247	9	5.777	0.271	0.151	3.026
25003 (Trout Beck @ Moor House)	4.266	46	15.142	0.168	0.29	0.52
106002 (Laxdale @ Laxdale)	4.322	12	17.449	0.103	0.132	1.308
54022 (Severn @ Plynlimon Flume)	4.338	38	14.988	0.156	0.171	1.19
27010 (Hodge Beck @ Bransdale Weir)	4.455	41	9.42	0.224	0.293	0.092
Total		536				
Weighted means				0.221	0.26	

A.1.2 Final Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.523	42	1.84	0.163	0.301	0.579
27051 (Crimple @ Burn Bridge)	3.278	47	4.524	0.218	0.156	1.394
45816 (Haddeo @ Upton)	3.285	26	3.456	0.3	0.406	0.802
28033 (Dove @ Hollinsclough)	3.569	44	4.177	0.228	0.371	0.844
25019 (Leven @ Easby)	4.067	41	5.09	0.342	0.386	1.182
91802 (Allt Leachdach @ Intake)	4.107	34	6.35	0.153	0.257	0.718
25011 (Langdon Beck @ Langdon)	4.207	33	15.647	0.232	0.328	1.6
71003 (Croasdale Beck @ Croasdale Flume)	4.225	37	10.9	0.212	0.323	0.174
25003 (Trout Beck @ Moor House)	4.266	46	15.142	0.168	0.29	0.361
54022 (Severn @ Plynlimon Flume)	4.338	38	14.988	0.156	0.171	0.982
27010 (Hodge Beck @ Bransdale Weir)	4.455	41	9.42	0.224	0.293	0.028
206006 (Annalong @ Recorder)	4.468	48	15.33	0.189	0.052	1.925
36010 (Bumpstead Brook @ Broad Green)	4.793	52	7.395	0.382	0.181	2.412
Total		529				
Weighted means		529		0.226	0.268	

A.2 Final Growth Curve



A.3 Diagnostic Plots

