A1 in Northumberland: Morpeth to Ellingham

Scheme Number: TR010059

6.50 River Coquet Hydraulic Modelling Report

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1 INTRODUCTION

1.1 BACKGROUND

1.1.1. The A1 in Northumberland: Morpeth to Ellingham, (Scheme), comprises two sections known as Part A: Morpeth to Felton (Part A) and Part B: Alnwick to Ellingham (Part B). The Scheme aims to increase capacity by widening the existing single carriageway to a dual carriageway along an approximately 12.6 km section of Part A (approximately 6.5 km of online widening and approximately 6.1 km of new offline highway) and along an approximately 8 km section of Part B.

1.1.2. An application for development consent for the Scheme was submitted by Highways England (Applicant) on 7 July 2020. The application was accepted for Examination on 4 August 2020. The Applicant submitted a change request to the Examining Authority (ExA) at Deadline 4 of the Examination (Change Request). On 9 April 2021, the ExA accepted the Change Request as part of the Application.

1.1.3. The Change Request incorporated three proposed changes:

a. The Earthworks Amendments;
b. The Stabilisation Works; and
c. The Southern Access Works.

1.1.4. Further details as to the nature of each of these changes is set out in the Environmental Statement: Earthworks Amendments [REP4-061], Environmental Statement Addendum: Stabilisation Works for Change Request [REP4-063] and Environmental Statement Addendum: Southern Access Works for Change Request [REP-064] submitted at Deadline 4 of the Examination.

1.1.5. As stated at paragraphs 9.5.3 of the Environmental Statement Addendum: Stabilisation Works for Change Request [REP4-063] and paragraphs 8.5.3 of the Environmental Statement Addendum: Southern Access Works for Change Request [REP-064] , the assessments submitted at Deadline 4 of the Examination in respect of the Stabilisation Works and the Southern Access Works was prepared on the basis of a Manning's calculation and, in order to verify those assessments, hydraulic modelling of the River Coquet was required.

1.1.6. The existing River Coquet Bridge is located approximately 1.5km to the southwest of Felton in Northumberland where the A1 is orientated north to south, with the River Coquet flowing from west to east through a series of meanders to the North Sea. The centre of the existing River Coquet Bridge is located at approximate Ordnance Survey Grid Reference (OSGR) 417436E 599810N.

1.1.7. This report provides an overview of the hydraulic modelling undertaken at the River Coquet in order to verify the following assessments:

a. Flood Risk Assessment Addendum – River Coquet (Document Reference 7.9.1.2)

b. River Coquet Fluvial Geomorphological Assessment (Document Reference 6.47)

1.1.8. Each of these assessments have been updated in light of the hydraulic modelling and are submitted to the Examination alongside this report (Flood Risk Assessment Addendum – River Coquet (Document Reference 7.9.1.2) and River Coquet Fluvial Geomorphology Assessment (Document Reference 6.47)).

1.2 MODELLED SCENARIOS

1.2.1. The hydraulic modelling exercise reported in this document has produced a 2 - dimensional (2D) hydraulic model of the River Coquet that can be used to represent the Baseline, Scheme Construction and Scheme in Operation scenarios. The Baseline scenario represents the existing situation, without the Scheme. The Scheme Construction scenario represents the situation during the construction of the Scheme, and takes account of the temporary river training works, temporary bridge to the south bank and installation of the north bank stabilisation works. Finally, the Scheme in Operation scenario represents the situation following the construction of the Scheme and takes account of the permanent scour protection on north and south banks and associated works. These works, specifically Work No. 4 a-d, are described in Schedule 1 (authorised development) of the Development Consent Order (Application Document Reference: TR010041/APP/3.1). The related Works Plans to these works are shown in Works Plans for Change Request Sheet Number 09 of 19 [REP4-036].

1.2.2. This report provides information regarding the representation of the Baseline Scenario, Scheme Construction and Scheme in Operation scenarios in the hydraulic model and their associated hydraulic model files. Sensitivity testing of key model parameters has been undertaken; model results associated with these tests are also described in this report.

1.2.3. Discussion of the model results is provided within the Flood Risk Assessment Addendum – River Coquet (Document Reference 7.9.1.2) and River Coquet Fluvial Geomorphology Assessment (Document Reference 6.47) submitted to the Examination at Deadline 7 alongside this report.

1.3 MODELLING SOFTWARE

1.3.1. The 2D hydraulic model has been developed using TUFLOW Heavily Parallelised Compute (HPC) version 2020-10-AA-iSP-w64[1], which is a widely accepted format. The model is based on a grid comprising cells of 2m size.

1.3.2. 2D Hydraulic models use numerical solvers to simulate two-dimensional surface flows such as occurs from flood and tides, with the 2D solution computed over a regular grid of square cells at defined computational timesteps. The extent of the model has been determined

[1] TUFLOW by BMT WBM Pty Ltd 2021

based on the study reach area of interest and suitable distances upstream and downstream of this area. The upstream boundary represents the main inflow to the model with additional lateral watercourses also including inflow boundaries. The downstream boundary represents the outflow from the model domain. The bed, banks and floodplain are represented as terrain information in the model grid. Hydraulic friction is overlaid on the model grid to represent the different landuse classifications and 'roughness' of the bed and banks. Flow hydrographs for the flood events of interest are introduced into the model and run for the duration of the flood events and results are provided for key parameters such as velocity, depth and associated parameters of interest (e.g. bed shear stress). Sensitivity tests examine the sensitivity of the model to changes in the hydraulic friction, flow and changes in the boundary representation. This allows for the assessment of the level of confidence that can be placed in the results provided by the hydraulic model.

1.4 MODELLING REVIEW

1.4.1. As set out in the Applicant's comments on the Environment Agency's response submitted in response to the ExA's Rule 17 Letter of 30 March [REP5-044], the Baseline model runs and associated reporting were submitted to the Environment Agency on Monday 19th April, the Scheme Construction and Scheme in Operation model runs and associated reporting (i.e. for the temporary construction phase and the permanent operational phase) were submitted on Thursday 29th April and the sensitivity test model runs and associated reported were submitted on the 7th of May. Environment Agency comments have been received on the baseline model runs and associated reporting and where appropriate, these comments have been addressed within this report.

1.5 DEADLINE 8A UPDATE

1.5.1. As outlined in Section 1.4 of this document, Environment Agency comments were received on the Baseline model runs and associated reporting on the 7th of May, where appropriate, these comments were addressed in the original submission of the report. There were two outstanding comments which were received following the Baseline review and the Applicant committed to providing a response to these once comments on the Scheme model runs and associated reporting were received (see below at 1.5.2). The Environment Agency have subsequently completed their review of the Scheme Construction and Scheme in Operation model runs, sensitivity test model runs and associated reporting on the 27th of May and provided no additional comments. The Environment Agency review summary conclusion is that the Baseline, Scheme Construction and Scheme in Operation models are considered reasonable. No updates to this report are therefore required beyond the matters set out in 1.5.2.

1.5.2. This hydraulic modelling report has been updated to address the two outstanding comments received from the Environment Agency on the Baseline model review. These outstanding matters relate to:

a. Providing photographs of key structures to aid in review of the flow constriction layers and adopted form loss values.

b. Provide details on how the outputs compare against anecdotal historic evidence (historic flood extents).

1.5.3. These inclusions can be found in Table 2-4 and Section 7, respectively.

2 BASELINE SCENARIO MODEL BUILD

2.1 INTRODUCTION

2.1.1. The Baseline Scenario represents the existing situation, without the Scheme. The following sections describe the schematisation of this scenario in the hydraulic model.

2.2 MODEL EXTENT

2.2.1. The model covers an area of 1.7km^2 and includes the watercourses listed in Table 2-1. The model domain is shown in Figure 2-1 and remains consistent for each scenario. Figure 2-1 also shows the Order limits following the Change Request.

Table 2-1 - Modelled Watercourses

Watercourse	Upstream End Location (Grid Reference)	Downstream End Location (Grid Refence)	Length (km)
River Coquet	To the east of Elyhaugh Farm Cottage (415960E 599785N)	200m downstream of Felton New Bridge (418695E 600434N)	4.1
Fence Burn	To the east of Felton Fence (416235E 601026N)	Confluence with the River Coquet (416534E 600500N)	0.7
Back Burn	To the east of the B6345 road (417875E 600807N)	Confluence with the River Coquet upstream of Felton Old Bridge (418432E 600236N)	0.9

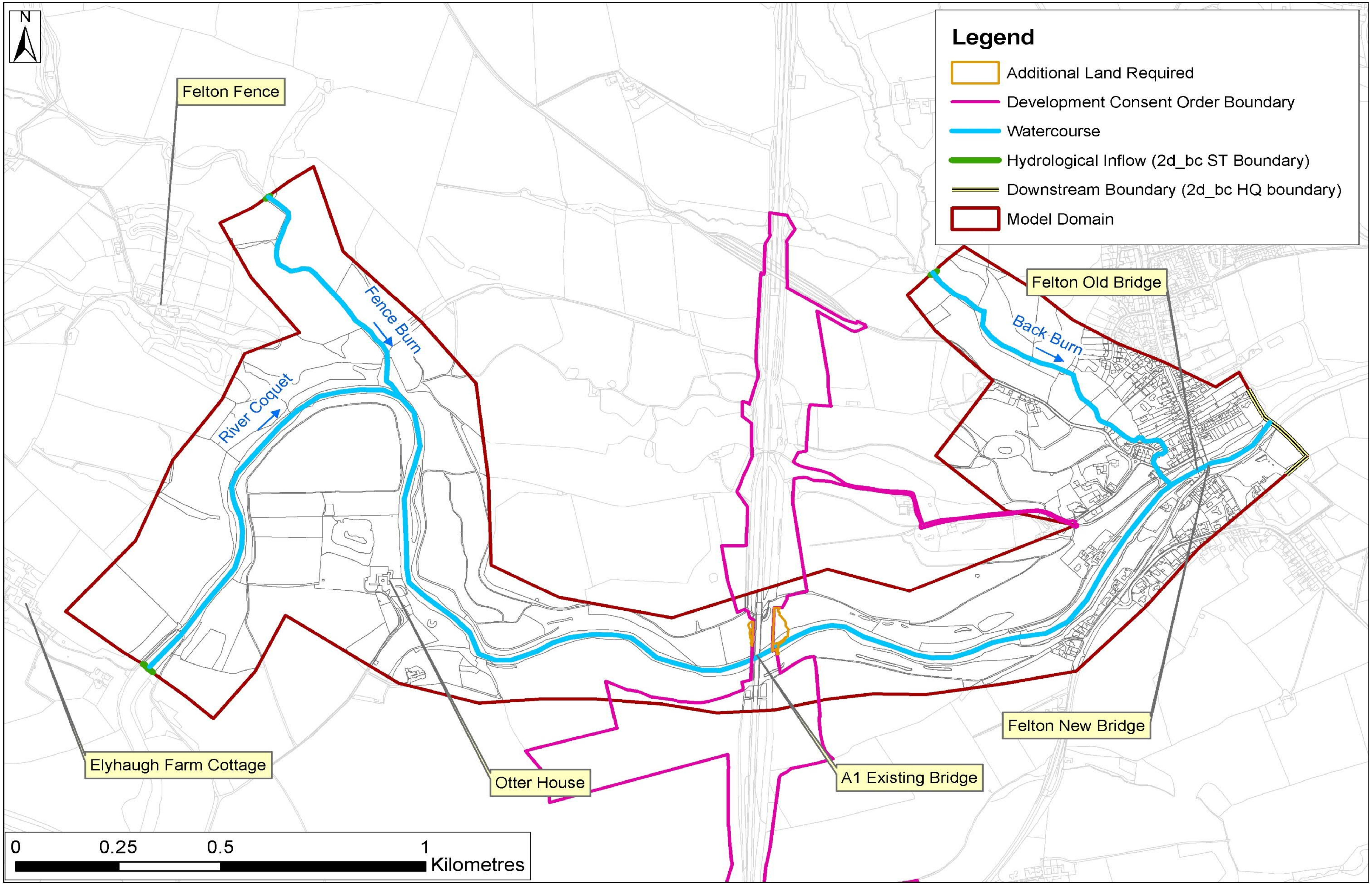

Figure 2-1 - Model Domain

2.3 TOPOGRAPHIC REPRESENTATION

2.3.1. The model topography representation is based on the datasets listed in Table 2-2. Datasets have been collected form a number of sources comprising Light Detection and Ranging (LiDAR) data, bathymetric data and topographic survey. The collection of the bathymetric and topographic surveys were procured and commissioned by the Applicant.

Table 2-2 - Datasets Used to Inform the Model Topography

Data	Description
Bathymetric survey data (HE551459-COS-VTO-M2F_MLT-DA-X-0002) – March 2021	Survey undertaken by Survey & Engineering Projects. (SEP), commissioned by the Applicant. This dataset covers most of the bed of River Coquet.
Riverbank survey data (HE551459-COS-VTO-M2F_MLT-PL-X-0002) – February 2021	Survey undertaken by SEP, commissioned by the Applicant This dataset includes survey data for the south bank of the River Coquet beneath the A1 bridge.
Costain topographic survey (HE551459-JAC-VTO-M2F_MLT-SU-Z-0001)	This dataset includes bank level contour information beneath the A1 bridge. Survey commissioned by the Applicant.
Composite Light Detection and Ranging (LiDAR) Digital Terrain Model (DTM)	1m horizontal resolution LiDAR DTM downloaded on the 8th of December 2020 from: http://environment.data.gov.uk/ds/survey/#/survey This dataset covers the entire model domain.

2.3.2. The Bathymetric survey data was used as the primary source of ground level information for the bed of the River Coquet. Ground level information for other areas (including the banks of the River Coquet) is based on LiDAR DTM data. Discrepancies in ground levels were found between the LiDAR DTM and the bathymetric survey data where the two datasets overlap. In order to reconcile ground levels between these two datasets, LiDAR DTM and Bathymetric surveyed levels were blended along the edges of the bathymetric survey extent (within a 2.5m buffer).

2.3.3. For the south bank of the River Coquet beneath the A1 bridge, the ground level information is based on a blend of ground level information extracted from the riverbank survey data and Costain topographic survey. No bathymetry survey data is available at this location.

2.3.4. The ground level information described above has been used to inform the 2m model grid with topographic elevation.

TOPOGRAPHIC MODIFICATIONS

2.3.5. The following modifications to the model topography have been undertaken (these are shown in Figure 2 2):

a. Some of the bathymetric survey data upstream of the A1 bridge was found to be inaccurate. Upon inspection of the dataset an area shown in Figure 2-2 was found to be inaccurate where some of the surveyed points showed bed levels approximately 1.5m higher than nearby surveyed points resulting in the creation of a small dam across the channel. These points where confirmed to be inaccurate by SEP due to loss of Global Positioning System (GPS) signal. Therefore, bathymetry survey data was not used at this location, instead, model ground levels were informed based on blended bathymetric survey data (from areas with accurate surveyed levels), riverbank survey data and Costain topographic survey. A TUFLOW 2d_zshape layer has also been used to interpolate bed levels using available datasets at this location.
b. Downstream of the confluence of Fence Burn with the River Coquet, the River Coquet bed levels are based on bathymetric survey data. Upstream of the confluence, the River Coquet bed levels are based on LiDAR DTM data as bathymetry of this smaller watercourse was not considered to be necessary for model accuracy and due to the distance from the likely area of potential impact. As a result, at the confluence of the River Coquet with Fence Burn, there is approximately a 1m discrepancy between the LiDAR DTM levels and the bathymetric surveyed levels (LiDAR DTM levels are higher). A TUFLOW 2d_zshape polygon has been used to interpolate the River Coquet bed levels over a distance of 150m upstream of the confluence with Fence Burn. The interpolation is based on bathymetry data and LiDAR DTM which provides a smooth transition from the LIDAR DTM levels to the bathymetric surveyed levels and avoids the creation of a 1m sudden drop in bed levels at this location.
c. TUFLOW 2d_zshape polygons have been used to carve-in Back Burn channel within Felton at Low Close, the B6245 and Riverside roads. The LiDAR DTM included existing bridge deck levels at this location which would have cause an obstruction to flow in the model, contrary to the situation on the ground.

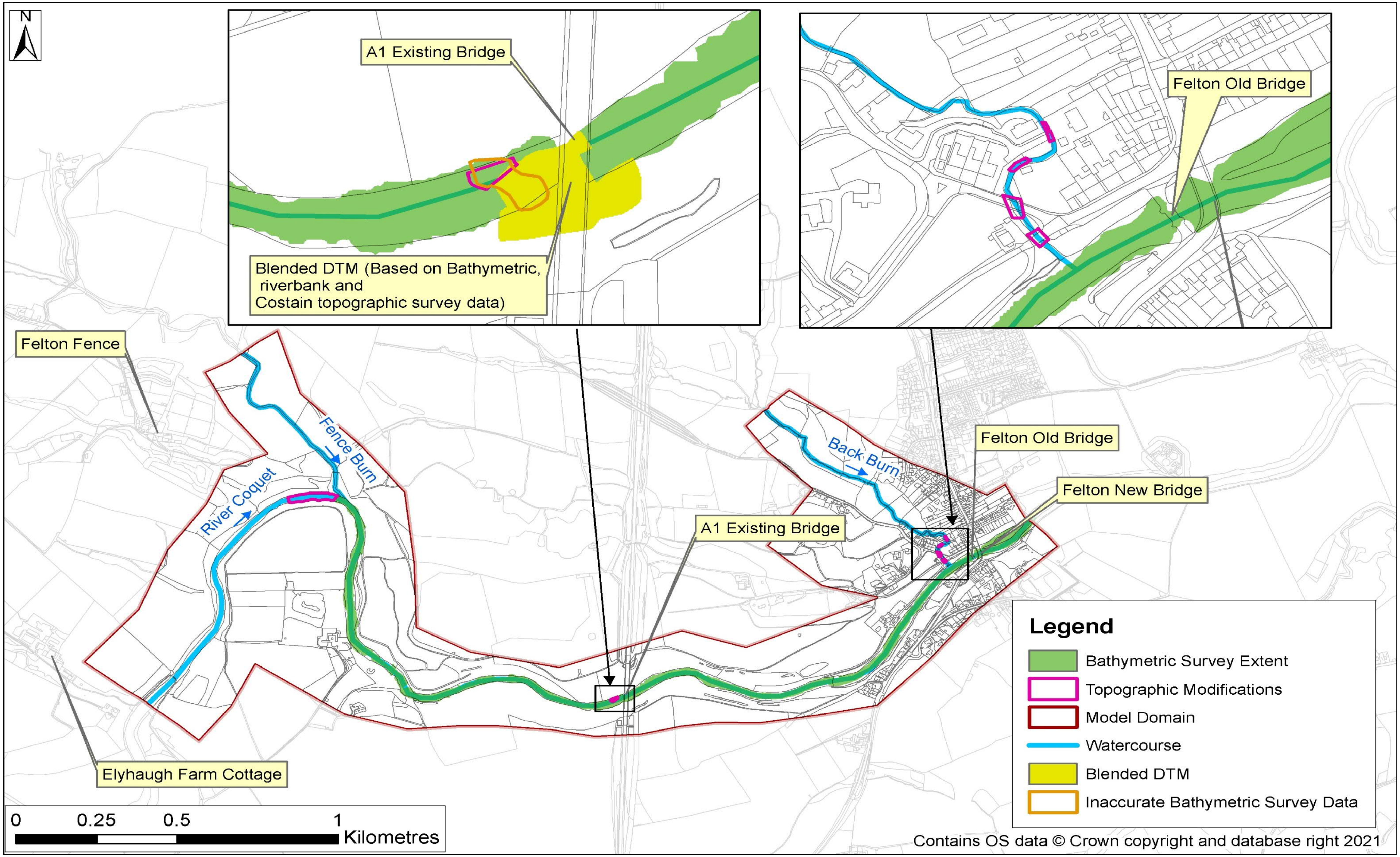

Figure 2-2 - Topographic Modifications

2.4 HYDRAULIC FRICTION

2.4.1. The Applicant's MasterMap data was used for this study, covering the entire model domain. This dataset was used to identify land use types and inform the 2D model with different Manning's 'n' roughness coefficient values, as shown in Table 2-3.

Table 2-3 - Manning's 'n' Roughness Coefficients – 2D Model Domain

MasterMap Feature Code	Land Use	Manning's 'n' Roughness Coefficient
10021	Buildings	1
10053	General Surface, Residential yards	0.05
10062	Building Glasshouse	1
10089	Watercourses (including the River Coquet) bed	0.035
10096	Slope	0.05
10099	Embankments/Cliff	0.05
10111	Woodland and forest (most of which covers the River Coquet valley sides)	0.07
10119, 10172	Roads, Tracks and Paths, Manmade	0.025
10123	Roads, Tracks and Paths Tarmac or dirt tracks	0.03
10183	Roadside	0.025
10185	Roadside structures	0.025
10217	Land, (unclassified), Industrial, Yards Car parks	0.025
10056	General surface	0.025 – 0.055

2.4.2. The hydraulic modelling of the River Coquet carried out by the Applicant prior to the development of the hydraulic model described in this report is presented in the River Coquet Geomorphology Assessment [REP3-009]. The Manning's 'n' roughness coefficient values for watercourses (Manning's 'n' roughness of 0.035) and for the River Coquet valley sides (Manning's 'n' roughness of 0.07) used for that modelling exercise were agreed with the Environment Agency, as detailed in Appendix B of the River Coquet Geomorphology Assessment [REP3-009]. These values have been adopted for the hydraulic modelling presented in this report.

2.4.3. TUFLOW 2d_mat polygons were created to apply a Manning’s ‘n’ roughness coefficient value of 0.050 within the River Coquet bed at locations of emerging bedrock and boulders (based on information collected during site visits). A 2d_mat polygon was also created to define the existing scour protection area (concrete surface) on the south bank beneath the existing A1 bridge, for which a Manning’s ‘n’ value of 0.025 was applied (see Figure 2-3).

2.4.4. As shown in Figure 2-3, the MasterMap data include road landuse polygons across the River Coquet channel at the location of the A1, Felton Old Bridge and Felton New Bridge. In order to correct this, roughness patches were created to enforce watercourse Manning’s ‘n’ roughness of 0.035 along the River Coquet and woodland Manning’s ‘n’ roughness of 0.07 (for the River Coquet valley sides beneath the A1 bridge).

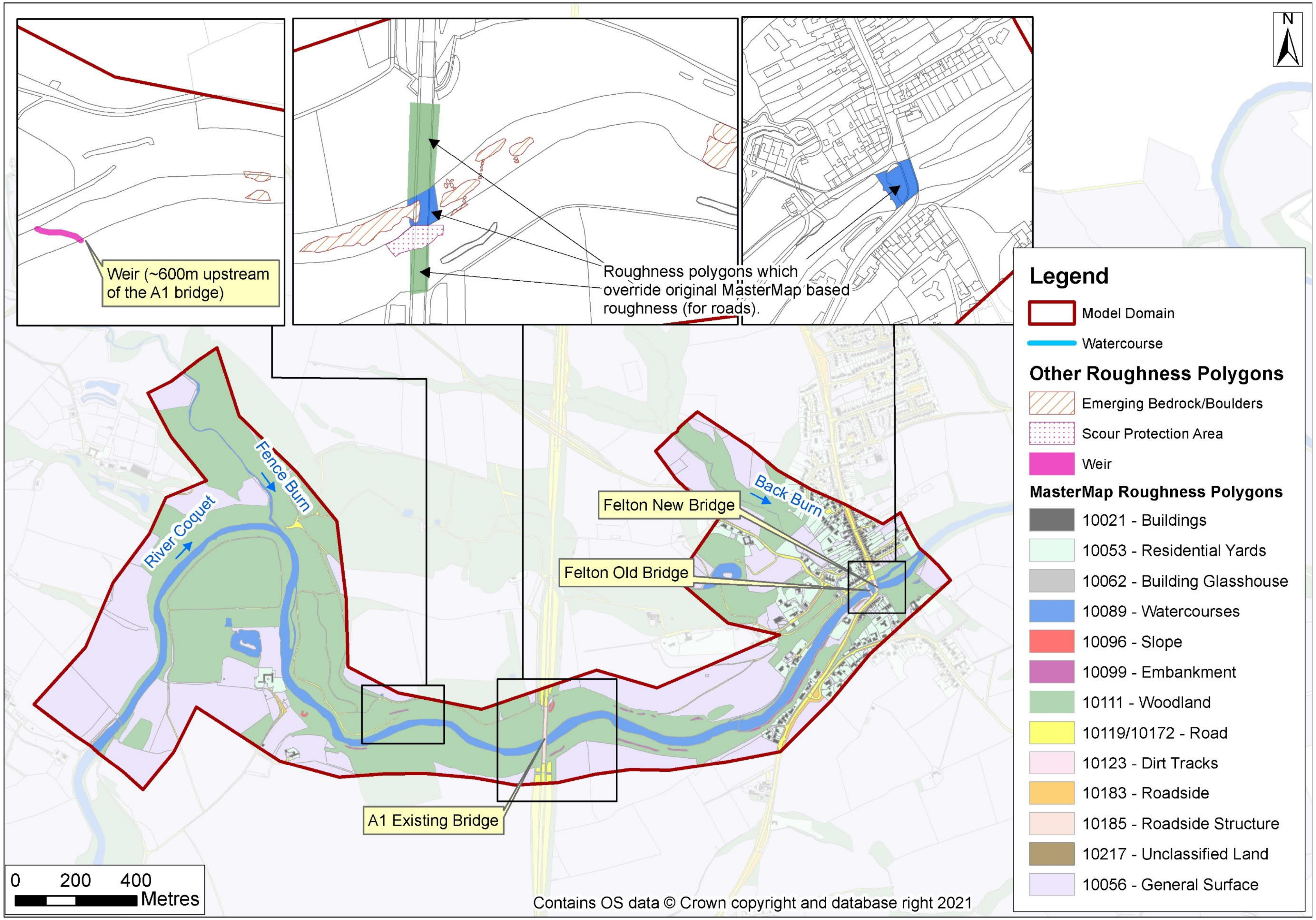

Figure 2-3 - Areas with Modified Roughness Values

2.5 REPRESENTATION OF CHANNEL STRUCTURES

2.5.1. Table 2-4 provides a description of the River Coquet channel structures represented in the hydraulic model (see Figure 2-4).

Table 2-4 - River Coquet Structures Represented in the 2D Model

Structure	Photograph	Representation
Existing A1 Bridge, North Pier		– Represented using a flow constriction (2d_lfcsh) polygon layer over two grid cells perpendicular to the direction of flow (due to grid alignment) and along the length of the pier (9.4m). – The pier has a width of 1.4m, therefore, a blockage of 35% was applied to each grid cell within 2d_lfcsh polygon. – No form loss coefficient was applied given the relatively small footprint of the pier (1.4m wide) over the entire length of the bridge (wet section) ~ 100m.
Existing A1 Bridge, South Pier		– Represented using a flow constriction (2d_lfcsh) polygon layer over one grid cell perpendicular to the direction of flow (due to grid alignment) and along the length of the pier (9.4m). – The pier has a width of 1.4m, therefore, a blockage of 70% was applied to the grid cell covered by the 2d_lfcsh polygon. – No form loss coefficient was applied given the relatively small footprint of the pier (1.4m wide) over the entire length of the bridge (wet section) ~ 100m.

Structure	Photograph	Representation
Weir (700m downstream of the A1 bridge)		– The bathymetric survey data included crest level information for this weir, which has been enforced using a TUFLOW 2d_zshape line.
Weir (600m upstream of the A1 bridge)		– The weir/bedrock ledge is included within the bathymetric survey data used to inform the model topography. – A higher manning's 'n' roughness value of 0.07 have been enforced at the location of the weir (see Figure 2-3).

Structure	Photograph	Representation
Felton Old Bridge		– There is no survey data available for this structure. – Represented using a flow constriction (2d_lfcsh) polygon layer over the entire length (48m) and width (6.5m) of the bridge. Bridge width and length are based on MasterMap data. – The bridge includes 2 piers, each with a width of 5m (based on MasterMap data). A form loss coefficient of 0.38 has been calculated based on available guidance[2]. – Soffit invert level of the bridge has been assumed based on LiDAR DTM. – Deck level of the bridge has been assumed to be 2m above the soffit of the structure.
Felton New Bridge		– There is no survey data available for this structure. – Represented using a flow constriction (2d_lfcsh) polygon layer over the entire length (52m) and width (12.6m) of the bridge. Bridge width and length are based on MasterMap data. – The bridge includes 2 piers, each with a width of 0.5m (assumed based on pictures). A form loss coefficient of 0.09 has been calculated based on available guidance2. – Soffit invert level of the bridge has been assumed based on LiDAR DTM. – Deck level of the bridge has been assumed to be 2m above the soffit of the structure.

[2] Joseph N. Bradley, Hydraulics of Bridge Waterways, U.S. Bureau of Public Roads, 1960

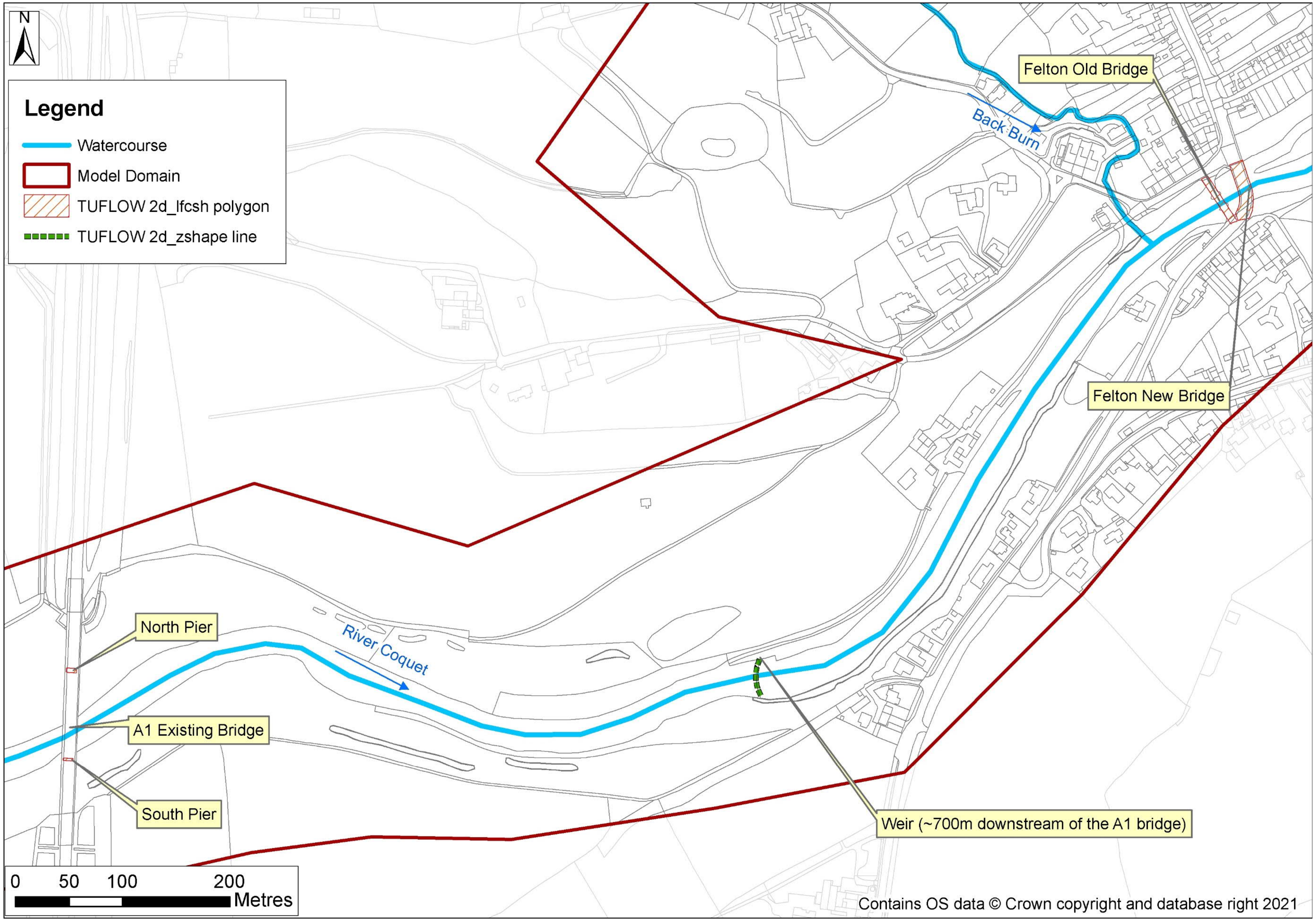

Figure 2-4 - River Coquet Structures

2.6 BOUNDARY CONDITIONS

2.6.1. The following boundary conditions have been implemented in the hydraulic model (see Figure 2-1):

a. Hydrological Inflows for the River Coquet, Fence Burn and Back Burn. These have been included using TUFLOW 2d_bc ST (flow versus time) boundary lines.
b. TUFLOW 2d_bc HQ (water level versus flow) boundary lines have been included at the downstream end the River Coquet, to allow flows to leave the model domain. The boundary is located 1.5km downstream of the A1 Bridge.

3 SCHEME CONSTRUCTION SCENARIO MODEL BUILD

3.1 INTRODUCTION

3.1.1. The Construction Scenario represents the temporary works to be undertaken prior to the Scheme becoming operational. The temporary works constitute modifications of the north and south banks downstream of the existing A1 bridge and the implementation of a temporary bridge. Full details of the works descriptions can be found in section 2.4 of the Environmental Statement Addendum: Stabilisation Works for Change Request [REP4-063] and section 2.4 of the Environmental Statement Addendum: Southern Access Works for Change Request [REP6-064]. These works are described in Schedule 1 (authorised development) of the Development Consent Order (Application Document Reference: TR010041/APP/3.1). The related Works Plans to these works are shown in Works Plans for Change Request Sheet Number 09 of 19 [REP4-036].

3.1.2. The following sections describe the schematisation of this scenario in the hydraulic model.

3.2 TOPOGRAPHIC REPRESENTATION

3.2.1. The temporary works include pilling platforms on both banks set at an elevation of 38m AOD surrounded by legato blocks training walls. The pilling platforms encroach into the existing River Coquet channel up to a maximum of 8m and reflect the reasonable worst case encroachment into the existing River Coquet channel (see Figure 3-1). The works on the north bank and south banks have been represented using separate DTMs. These DTMs have been stamped on top of the Baseline Scenario DTM in the hydraulic model.

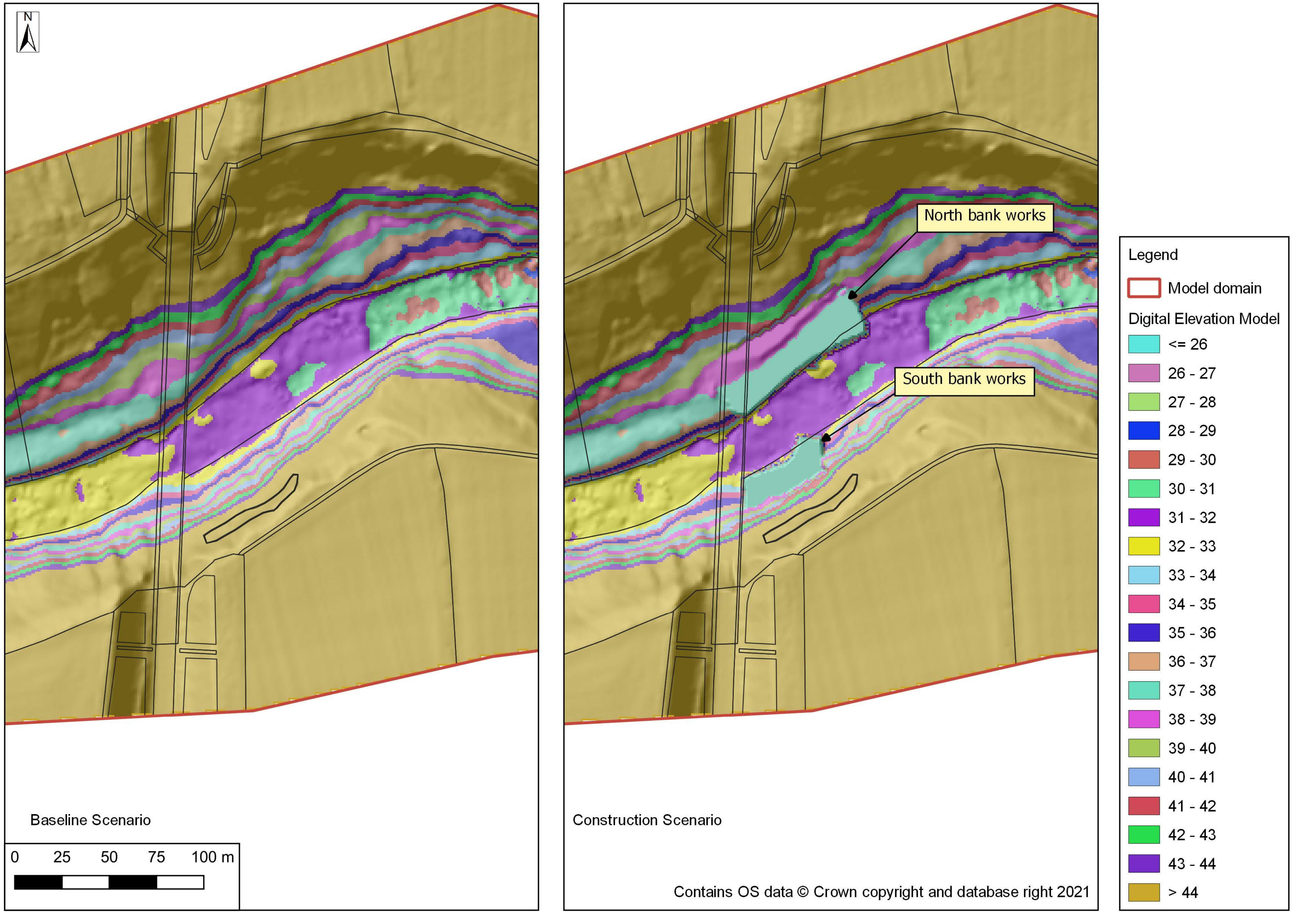

Figure 3-1 - Topographic Modifications – Comparison of the Model Topography Between Baseline and Construction Scenarios.

3.3 HYDRAULIC FRICTION

3.3.1. In order to account for the temporary surfaces on the north and south banks. Manning's 'n' roughness coefficient values were modified as shown in Figure 3-2 and detailed in Table 3-1.

Table 3-1 - Manning's 'n' Roughness Coefficients – Temporary Works

TUFLOW Material Code	Temporary Surface	Manning's 'n' Roughness Coefficient
2001	Piling Platform	0.025
2001	Transition Piling Platform to Legato Blocks	0.025
2001	Temporary bridge abutment	0.025
2002	Legato block training wall	0.015

3.3.2. TUFLOW 2d_mat polygons were created to apply the Manning's n roughness values for the temporary works.

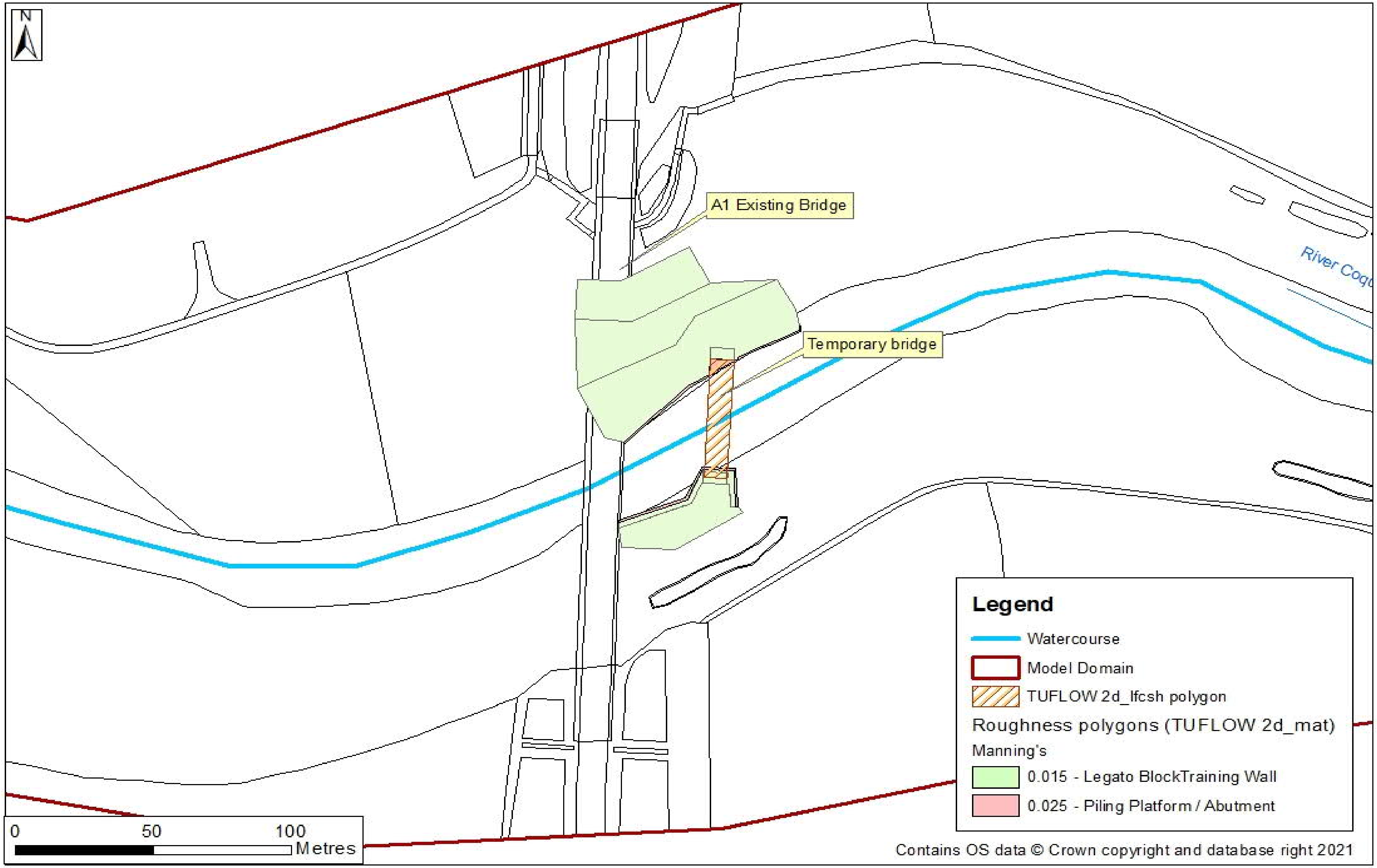

Figure 3-2 - Location of Areas with Modified Roughness Coefficient Values and Temporary Bridge – Construction Scenario

3.4 REPRESENTATION OF CHANNEL STRUCTURES

3.4.1. Table 3-2 provides a description of the representation of the temporary bridge in the Construction Scenario. The location of the temporary bridge is shown in Figure 3-2.

Table 3-2 - Construction scenario – Temporary Bridge Representation

Structure	Representation
Temporary bridge	– Represented using a flow constriction (2d_lfcsh) polygon layer over the entire length (47.8m) and width (8.0m) of the bridge. – The bridge is a clear span bridge; therefore, no blockages has been applied in between the bed of the channel and the soffit of the structure. – The bridge has a soffit level of 37.3m AOD and a deck level of 38m AOD. The area in between the bridge deck and the bridge soffit is assumed to be 100% blocked. A total form loss of 0.4 has been applied as flow conditions are expected to drown out the bridge deck. – The bridge will include a parapet wall with a height of 1.6m above the deck of the structure. The wall includes a series of openings. It has been assumed that 40% of the area of the parapet wall will be obstructed.

4 OPERATIONAL SCENARIO MODEL BUILD

4.1 OPERATIONAL SCENARIO MODEL BUILD

4.1.1. The Operational Scenario represents the Scheme in operation. The relevant works for the purpose of the hydraulic model are the new south and north piers for the new bridge as well as the inclusion of the scour protection on the north and south banks. Full details of the works descriptions can be found in section 2.4 of the Environmental Statement Addendum: Stabilisation Works for Change Request [REP4-063] and section 2.4 of the Environmental Statement Addendum: Southern Access Works for Change Request [REP6-064].The following sections describe the schematisation of this scenario in the hydraulic model.

4.2 TOPOGRAPHIC REPRESENTATION

4.2.1. The proposed south bank works include a concrete pile cap (set at an elevation of 36m AOD) upon which the proposed south pier will sit (see Figure 4-1) which reflects the reasonable worst case for the bridge foundation. Gabion and rock armour revetments will also be included for both north and south banks.

4.2.2. The south bank works have been represented using a separate DTM of the proposed works which has been stamped on top of the Baseline Scenario DTM.

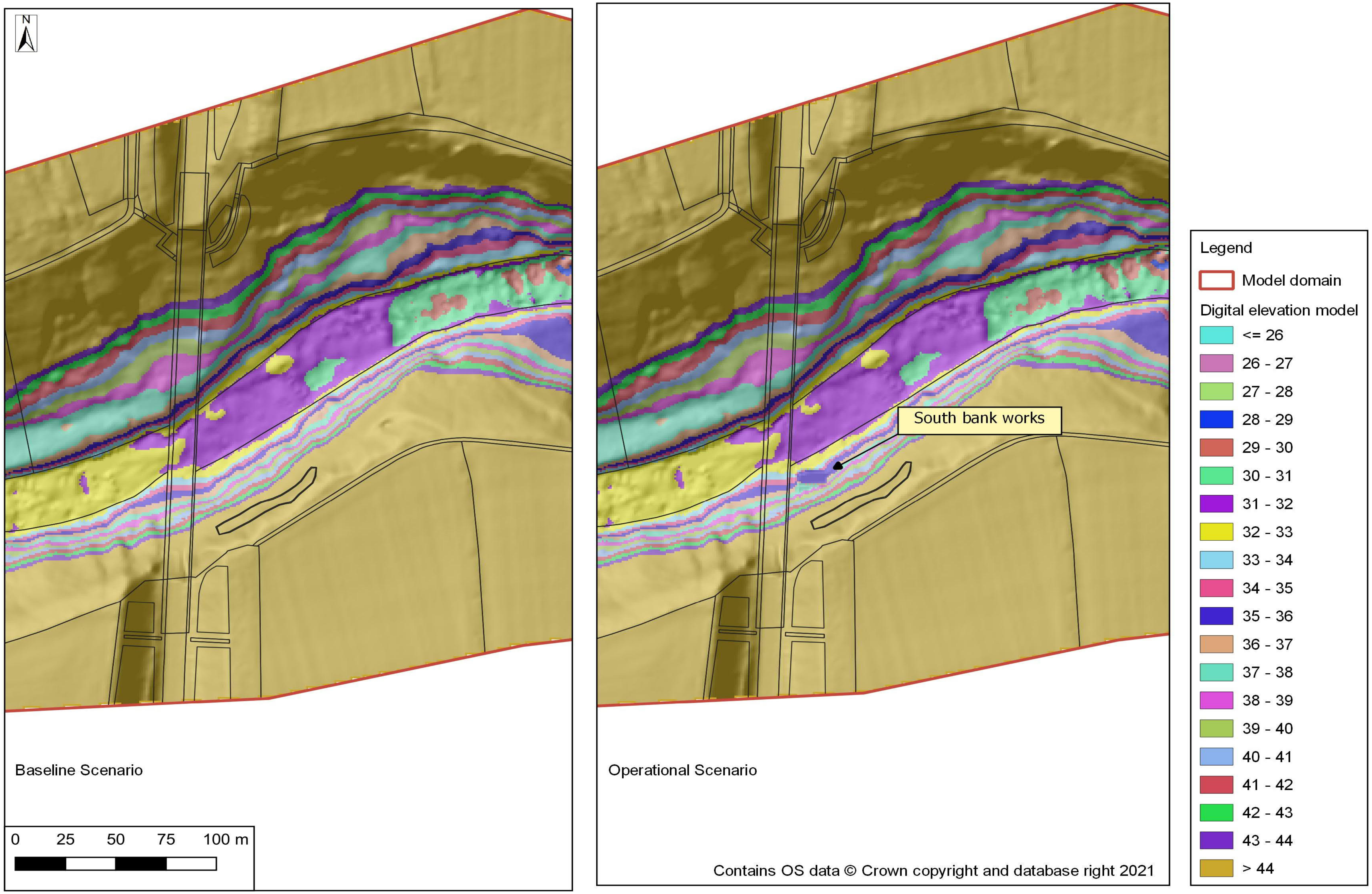

Figure 4-1 - Topographic Modifications – Comparison of the Model Topography Between Baseline and Operational Scenarios

4.3 HYDRAULIC FRICTION

4.3.1. In order to account for the proposed surfaces on the north and south banks, Manning's 'n' roughness coefficient values were modified as shown in Figure 4-2 and detailed in Table 4-1.

Table 4-1 - Manning's 'n' Roughness Coefficients – 2D Model Domain

TUFLOW Material Code	Proposed Scheme Surface	Manning's 'n' Roughness Coefficient
20200	Concrete pile cap	0.015
20210	Gabion revetment	0.040
20220	Rock armour	0.045
20230	Rock armour	0.050

4.3.2. TUFLOW 2d_mat polygons were created to apply the Manning's n roughness values for the proposed Scheme.

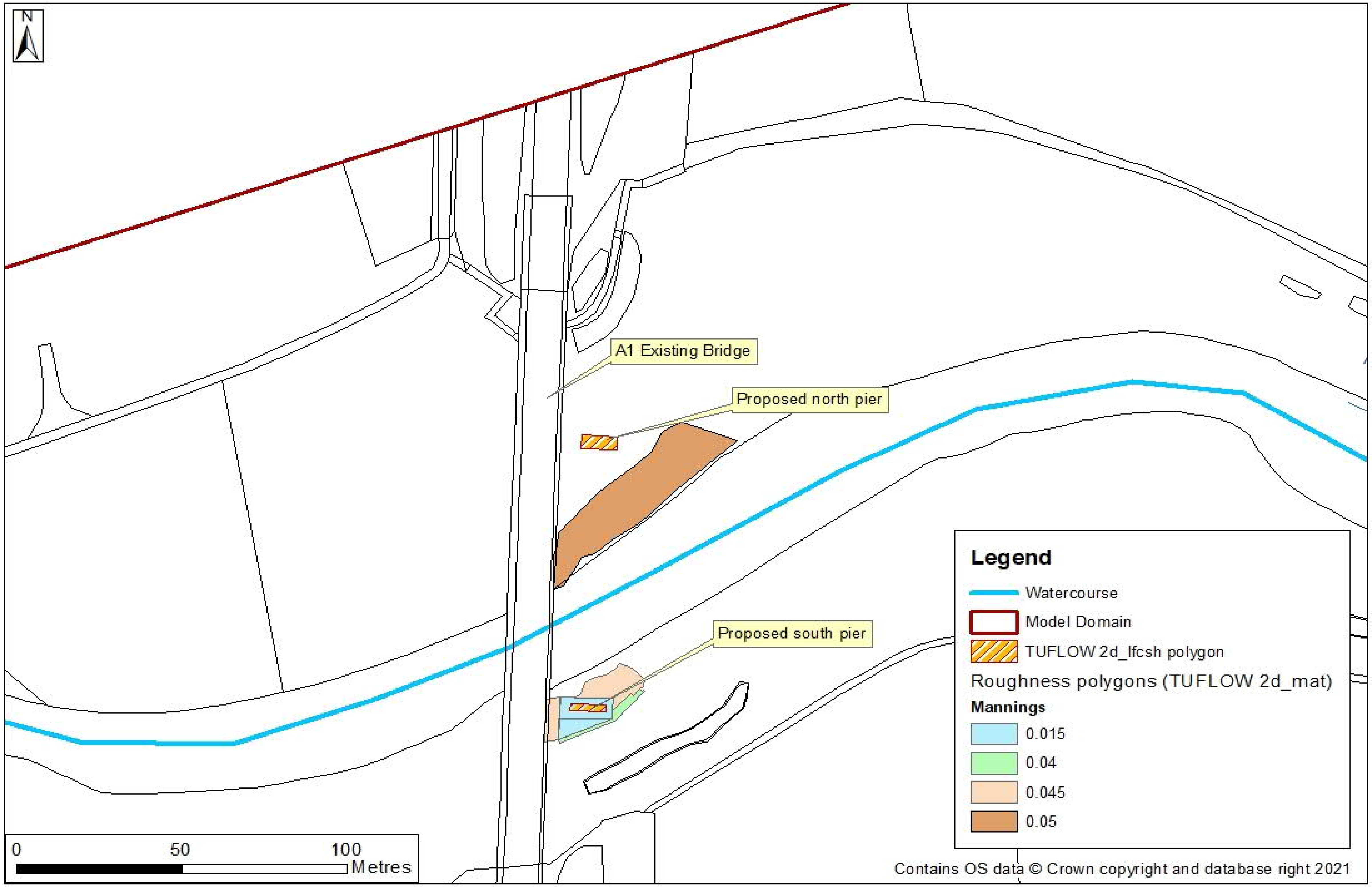

Figure 4-2 - Location of Areas with Modified Roughness Coefficient Values and Proposed Piers – Operational Scenario

4.4 REPRESENTATION OF CHANNEL STRUCTURES

4.4.1. Table 4-2 provides a description of the representation of the north and south piers in the Operational Scenario. The location of the piers is shown in Figure 4-2.

Table 4-2 - Operational Scenario – Pier Representation

Structure	Representation
North pier	– Represented using a flow constriction (2d_lfcsh) polygon layer over two grid cells perpendicular to the direction of flow (due to grid alignment) and along the length of the pier (10.8m). – The pier has a width of 2.0m, therefore, a blockage of 50% was applied to each grid cell within 2d_lfcsh polygon. – No form loss coefficient was applied given the relatively small footprint of the pier (2m wide) over the entire length of the bridge (wet section) ~ 100m.
South Pier	– Represented using a flow constriction (2d_lfcsh) polygon layer over one grid cell perpendicular to the direction of flow (due to grid alignment) and along the length of the pier (10.8m). – The pier has a width of 2.0m, therefore, a blockage of 100% was applied to the grid cell covered by the 2d_lfcsh polygon. – No form loss coefficient was applied given the relatively small footprint of the pier (2m wide) over the entire length of the bridge (wet section) ~ 100m.

5 MODEL EVENTS AND SIMULATIONS

5.1.1. The following Annual Exceedance Probability (AEP) events have been simulated with the Baseline, Construction and Operational models: 50%, 20%, 10%, 2%, 1%, 1% plus 65% climate change flow uplift, 0.5%, 0.5% plus 50% climate change flow uplift, 0.5% plus 65% climate change flow uplift, 0.1% and 0.1% plus 50% climate change flow uplift.

5.1.2. 50% and 65% climate change flow uplifts are based on the Environment Agency guidance[3] for the year 2115. These are applicable for the Northumbria river basin district of which the River Coquet catchment is part of. The 50% flow uplift corresponds to the upper end allowance category and the 65% flow uplift corresponds to the most extreme climate change scenario allowance category.

5.1.3. The Q50 or 50 percentile flow (low flow value expected to be exceeded 50% of the time) has also been simulated.

5.1.4. Sensitivity tests were carried out using the 0.5% AEP plus 65% climate change flow uplift event (see Section 8).

[3] Environment Agency, Flood Risk Assessments: Climate Change Guidance, https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances, Last updated 22nd July 2020

6 FLOW RECONCILIATION

6.1.1. A flow reconciliation exercise has been undertaken by comparing modelled peak flows to hydrological statistical estimates produced using the Flood Estimation (FEH) methodology downstream of Felton New Bridge (Grid reference 418600E 600350N). Hydrology for these events has been derived using the industry standard best practice methods outlined in the Flood Estimation Handbook (FEH). The hydrology audit trail is provided in Appendix A.

6.1.2. The % differences between modelled peak flows and the hydrological estimates are provided in Table 6-1. It is considered that these % differences are acceptable based on accepted modelling tolerances and therefore, no scaling of the hydrological inflows has been deemed necessary.

Table 6-1 - Flow Reconciliation

AEP Event	Hydrological Estimate (m^3/s)	Modelled Peak Flow (m^3/s)	Difference (%)
Q50	4.43	4.41	-0.4
50%	152.1	154.2	+1.4
20%	205.5	208.1	+1.3
10%	243.0	245.9	+1.2
2%	340.8	343.6	+0.8
1%	390.5	393.1	+0.7
0.5%	450.9	453.6	+0.6
0.1%	613.1	614.0	+0.2
1% + 65% CC	644.4	644.8	+0.1
0.5% + 50%CC	676.3	677.1	+0.1
0.5% + 65% CC	743.9	743.3	-0.1
0.1% + 50% CC	919.6	913.2	-0.7

7 BASELINE SCENARIO MODEL RESULTS VERSUS HISTORIC RECORDS OF FLOODING

7.1.1. No model calibration or verification has been undertaken as there are no flow or level gauges within the modelled reach and hence a formal calibration and verification exercise is not possible. However, Baseline Scenario model results have been compared against historic records of flooding.

7.1.2. The Environment Agency has provided eight records of historical flood events in the vicinity of the crossing in the form of recorded flood outlines. These have been reviewed and are listed in Table 7-1.

Table 7-1 - Records of Historical Flood Events in the Vicinity of the Crossing

Date of Event	Source of Flooding	Data Source	Area Flooded	Notes
6th of November 2000	River Coquet	Visual	Felton – Stable Cottage	The outline covers a small area within Felton.
6th of September 2008	River Coquet	Visual	Felton Mill	The outline covers a small area within Felton.
6th of September 2008	River Coquet	Aerial photography	River Coquet valley	The outline covers a large area ~ 300m upstream of the A1 bridge but does not include the A1 bridge area.
6th of September 2008	River Coquet	Aerial photography	River Coquet valley	The outline covers a large area from 300m upstream of the A1 bridge (including the A1 bridge).
7th of September	Back Burn	Visual	Felton	The outline covers a small area within Felton.
17th of July 2009	Unknown	Survey (poor data quality)	Felton	The outline covers a small area within Felton.
31st of March 2010	River Coquet	Visual	Felton Mill	The outline covers a small area within Felton.
28th of June 2012	Back Burn and surface water	Survey (poor data quality)	Felton	The outline covers a small area within Felton.

7.1.3. The recorded flood outlines covering smaller discrete areas offer insufficient flood extents to facilitate a reasonable comparison to modelled results albeit that the locations recorded are shown to flood by the hydraulic model results.

7.1.4. Only the records associated with the event on the 6th of September 2008 have been considered suitable for comparison against the hydraulic model results.

7.1.5. The 6th of September 2008 event corresponds with the highest recorded flows at the upstream gauge station in Rothbury (Grid reference NU 06700 01600) and downstream gauge station at Morwick (Grid reference NU 23400 04400). These stations recorded 416.56m^3/s and 525.72m^3/s respectively. Estimating the corresponding flow at the A1 crossing based on catchment area derives a flow of approximately 485m^3/s. The event modelled with a peak flow closest to this is the 0.5% AEP event with an estimated peak flow of 450.9m^3/s. The next nearest modelled event is the 0.1% AEP with an estimated peak flow of 613.1m^3/s. whilst the 1% AEP event is estimated to have a peak flow of 390.5m^3/s.

7.1.6. The modelled extents for the 1% AEP, 0.5% AEP and 0.1% AEP events accord well with the recorded flood outlines over much of the modelled area (see Figure 7-1, Figure 7-2, Figure 7-3). This is to be expected given the topography of the gorge containing the flow. The notable area of difference is in the vicinity of Shothaugh Farm / Otter House where the modelled extents are significantly greater for each of the modelled events compared to the recorded outlines. In this location and in areas adjacent to the River Coquet channel it is considered that the aerial imagery used to determine the historic flood outlines was unable to define flooding in this area due to dense vegetation.

7.1.7. It is considered that the model results are reasonable based on comparison of modelled flood extents against recorded flood outlines.

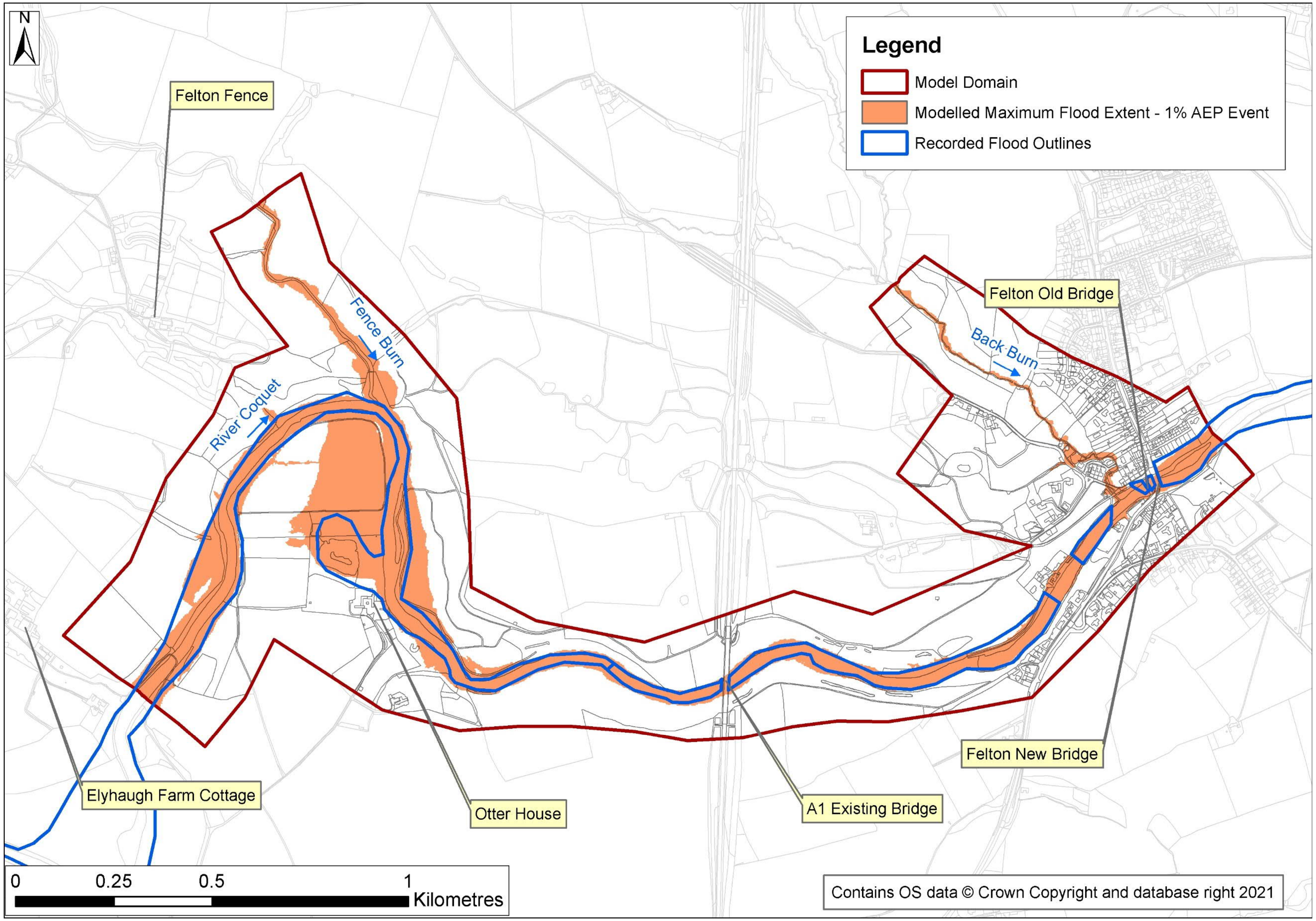

Figure 7-1- Modelled Maximum Flood Extent 1% AEP Event versus Recorded Flood Outlines 6th of September 2008 Flood Event

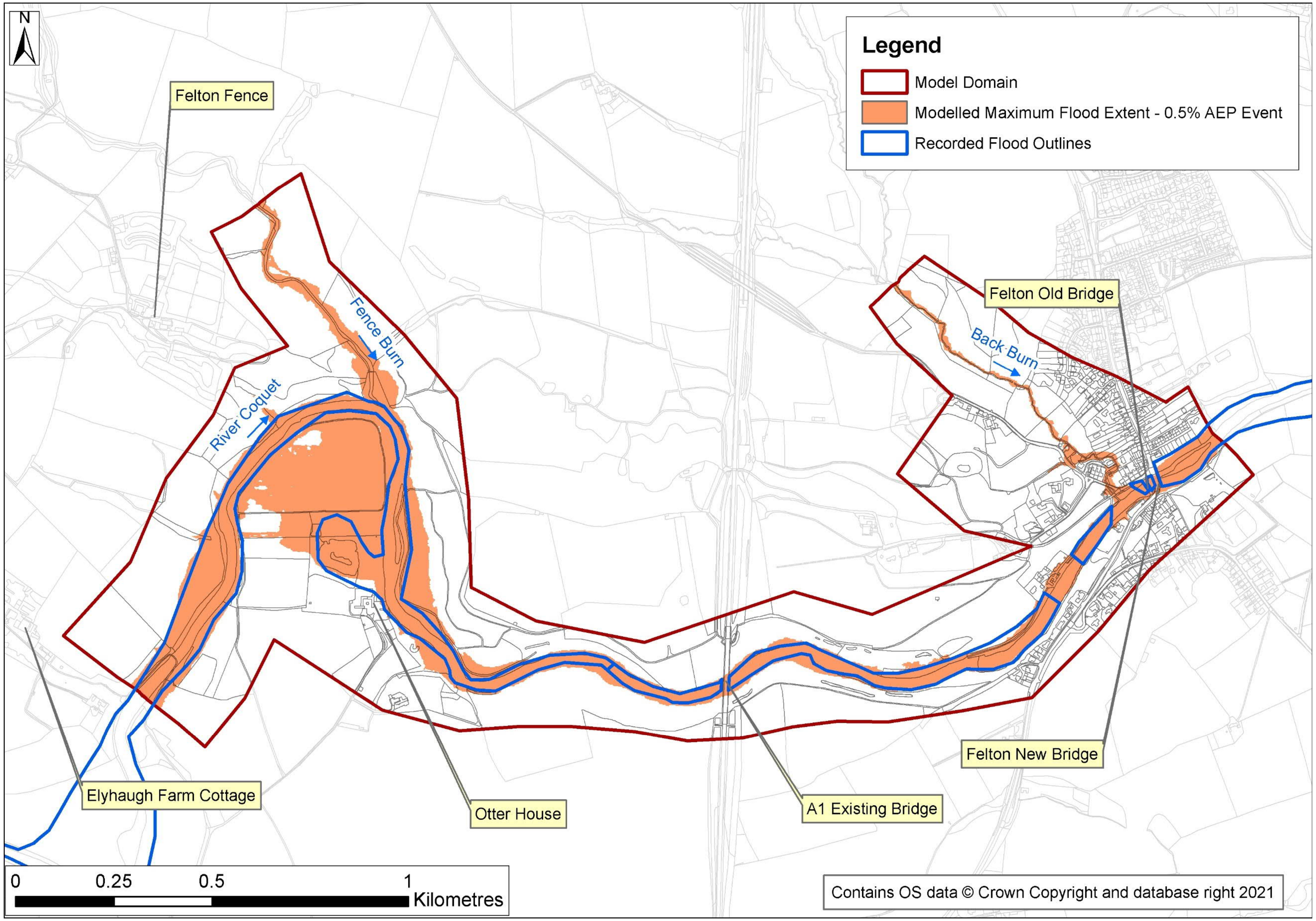

Figure 7-2 - Modelled Maximum Flood Extent 0.5% AEP Event versus Recorded Flood Outlines 6th of September 2008 Flood Event

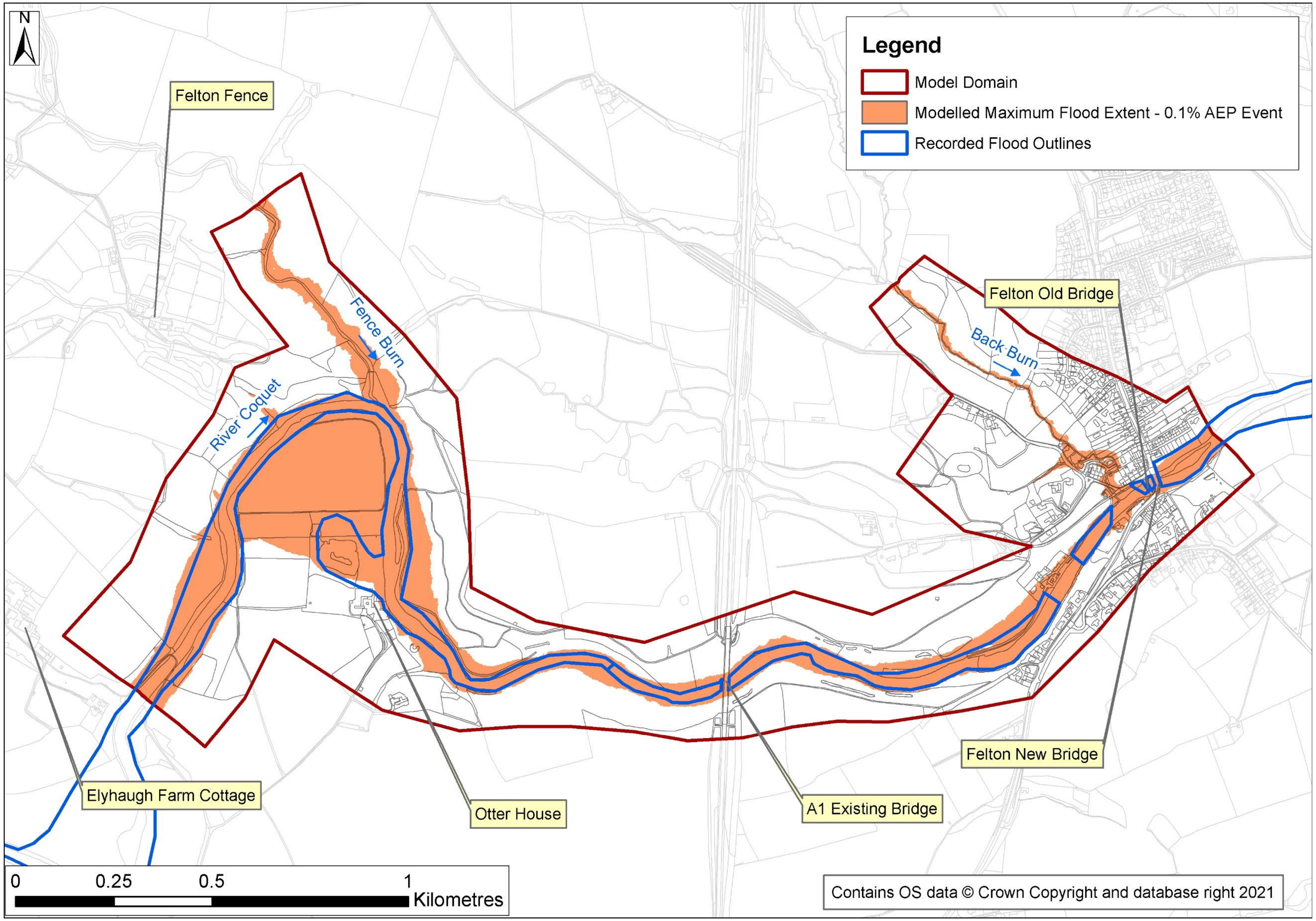

Figure 7-3 - Modelled Maximum Flood Extent 0.1% AEP Event versus Recorded Flood Outlines 6th of September 2008 Flood Event

8 MODEL NUMERICAL PERFORMANCE

8.1.1. TUFLOW HPC hydraulic modelling software provides numerical performance guidance along with acceptable 2D mass error ranges that should be achieved during each model run. The accepted tolerance range recommended by the software manual is +/- 1% mass balance error, furthermore, due to the nature of the TUFLOW HPC solution scheme (explicit solver), it is expected that 2D volume be conserved with a 2D mass error close to 0%.

8.1.2. For all simulations carried out for this study (Baseline, Construction and Operational Scenarios and sensitivity tests), the 2D mass error outputs are all within this tolerance and close to 0%, as shown in the example presented in Figure 8-1 for the Baseline Scenario 0.1% AEP plus 50% event. The change in volume through each model simulation has also been checked and has been found to vary relatively smoothly which is another indicator of good convergence of the 2D model.

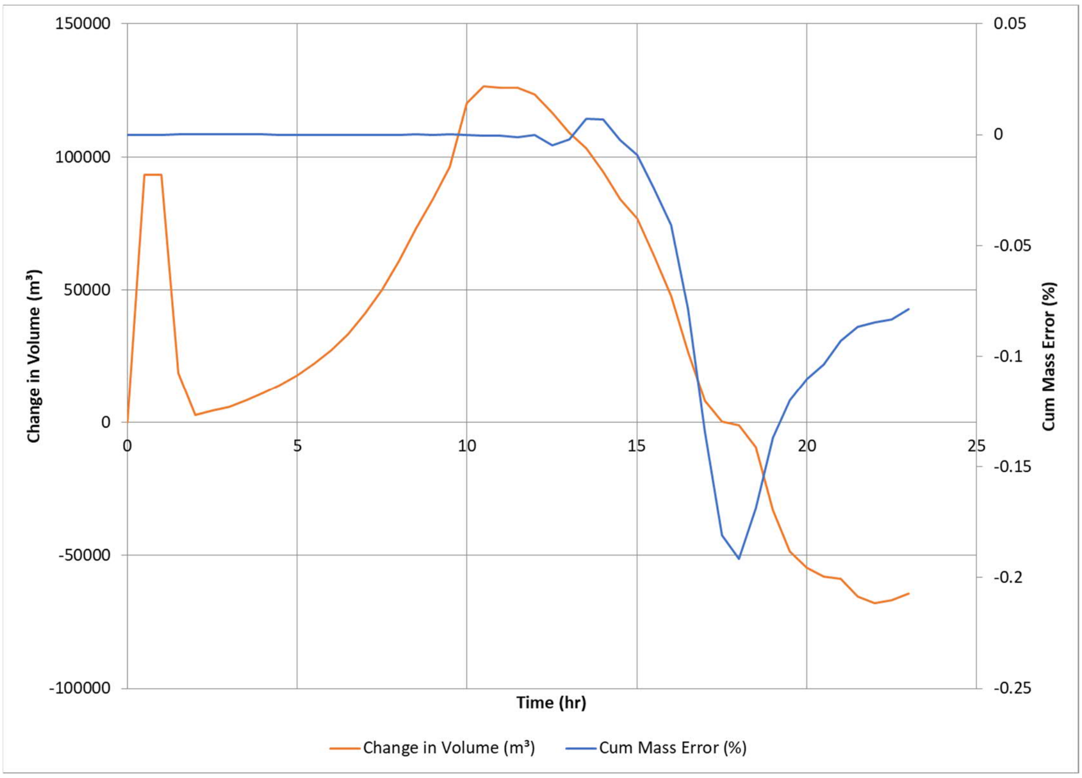

Figure 8-1 - TUFLOW 2D Model Mass Error vs Change in Volume – Baseline Scenario – 0.1% AEP plus 50% Climate Change Flow Uplift Event

8.1.3. As TUFLOW HPC uses adaptive timestepping to maintain numerical stability. There are 3 parameters that help monitor the numerical performance of the model during the computation. These parameters are the Courant Number or Nu, the Wave Speed Number or Nc and the Momentum Diffusion Number of Nd. The software developers recommend these parameters to be Nu≤1, Nc≤1 and Nd≤0.3 and not be exceeded by 20% during the computation. For all simulations carried out in this study, these parameters remain within the recommended tolerance.

8.1.4. Other factors that confirm that the model performance is healthy are the variation in timestep and efficiency. For all simulations, the efficiency is high and close to 100%.

8.1.5. Warnings regarding instability timestep corrections are present for all the simulations. The location of these instabilities is 80m downstream of the River Coquet hydrological inflow. However, the hydraulic model results do not show a significant impact on the model results associated with these instability timestep corrections. The instability is highly localised and shows no evidence of influencing the results of interest in the vicinity of the crossing or other areas of interest.

9 SENSITIVITY TESTING

9.1 INTRODUCTION

9.1.1. Sensitivity tests were undertaken as part of this study on key model parameters using the 0.5% AEP plus 65% climate change flow uplift event. The following sections describe these sensitivity tests and impacts on model results.

9.1.2. Table 9-1 provides Baseline Scenario sensitivity tests results at key locations (see Figure 2-1) including the River Coquet beneath the A1 existing bridge.

9.1.3. Thematically mapped model results for the Baseline Scenario sensitivity tests at the location of the A1 existing bridge are provided in Appendix B.

9.2 BASELINE SCENARIO DOWNSTREAM BOUNDARY CONDITIONS SENSITIVITY TESTS

9.2.1. As mentioned in Section 2.6, TUFLOW 2d_bc HQ (water level versus flow) boundary lines have been included at the downstream end of the River Coquet model to allow flows to leave the model domain. TUFLOW has been allowed to automatically generate the HQ relationship based on input slope values of 0.002 for the main channel and 0.01/0.009 for the floodplain (these values are based on bathymetric survey and LiDAR DTM data).

9.2.2. The downstream boundary slopes have been increased and reduced by +/-20% respectively to test the impact on model results at the location of the A1 River Coquet crossing.

9.2.3. As shown in Table 9-1 increasing or reducing the downstream boundary slope by +/-20% respectively does not have an impact on maximum water levels at any of the 3 key locations inspected. Figure 9-1 shows that the area impacted by the change in downstream boundary slope is limited to the area adjacent to the model boundary. The classification of the magnitude of the water level difference presented in the Figure 9-1 has been done in accordance with the criteria specified within Highways England’s Design Manual for Roads and Bridges[4].

9.2.4. The model results for these tests have not been mapped in Appendix B as model results do not change at the location of the existing A1 bridge when compared against the Baseline Scenario model results.

9.3 BASELINE SCENARIO MANNING’S ‘N’ ROUGHNESS SENSITIVITY TESTS

9.3.1. Hydraulic roughness is expected to change throughout the year during the summer and winter seasons. The model sensitivity towards roughness of the river channel and floodplain

[4] Highways England, Design Manual for Roads and Bridges, LA 113 Road drainage and the water environment, Rev 0

has been tested by increasing and reducing the Manning's 'n' roughness values by +/-20% respectively.

9.3.2. Table 9-1 shows that maximum water levels are sensitive to changes in Manning's 'n' roughness values with maximum water levels increasing or reducing by +/-0.5m along the entirety of the River Coquet.

9.3.3. At the location of the existing A1 bridge, maximum water levels are increased by +0.480m when roughness values are increased by +20%. When roughness values are reduced by -20%, the maximum water level is reduced by -0.543m.

9.4 BASELINE SCENARIO HYDROLOGICAL INFLOWS SENSITIVITY TESTS

9.4.1. Hydrological inflows have been estimated using the FEH methodology. Uncertainties associated with the FEH methodology have been assessed by comparing model result of the 0.5% AEP plus 65% climate change flow uplift event against the following events:

a. 0.1% AEP event which hydrological inflows are nearly -20% lower than those of the 0.5% AEP plus 65% climate change flow uplift event.
b. 0.1% AEP plus 50% climate change flow uplift event which hydrological inflows are nearly +20% higher than those of the 0.5% AEP plus 65% climate change flow uplift event.

9.4.2. Table 9-1 shows that maximum water levels are sensitive to +/-20% variation of the hydrological inflows. For most locations the maximum water levels has been increased or reduced by more than +/-0.5m.

9.4.3. At the location of the existing A1 bridge, maximum water levels are increased by +0.775m or reduced by -0.654m when inflows are +20% greater or -20% lower respectively.

9.4.4. Hydraulic model results show a change in maximum flood extents as expected due to the reduced or increased water levels in the Manning's 'n' roughness values and hydrological inflows sensitivity tests scenarios. Due to the River Coquet valley topography with steep side slopes, most of the flooding is contained within the valley of the river. Otter House (upstream of the existing A1 bridge) and properties within Felton are the only receptors within the study area. Otter House is not expected to flood internally in the baseline scenario or sensitivity tests scenarios. Hydraulic model results predict internal flooding of properties within Felton. The number of properties flooding in Felton increases by 3-5 properties when manning's 'n' values are increased and when a variation of +20% in hydrological inflows is considered, there is not much of change relative to the Baseline scenario in the other sensitivity tests scenarios.

Table 9-1 - Baseline Scenario – Sensitivity Tests Results at Key Locations – 0.5% AEP plus 65% Climate Change Flow Uplift Event

Key Location	Maximum Water Level (m AOD)						
	Baseline	Downstream Boundary Slope - 20%	Downstream Boundary Slope +20%	Manning's 'n' Roughness Values - 20%	Manning's 'n' Roughness Values +20%	Hydrological Inflows -20%	Hydrological Inflows +20%
River Coquet - to the west of Otter House	41.766	41.766	41.766	41.304	42.205	41.047	42.599
River Coquet - beneath A1 existing bridge	38.031	38.031	38.031	37.488	38.511	37.377	38.806
River Coquet - Upstream of Felton Old Bridge	32.933	32.934	32.932	32.634	33.211	32.442	33.551

Difference - Sensitivity Test minus Baseline (m)						
Key Location	Downstream Boundary Slope - 20%	Downstream Boundary Slope +20%	Manning's 'n' Roughness Values - 20%	Manning's 'n' Roughness Values +20%	Hydrological Inflows -20%	Hydrological Inflows +20%
River Coquet - to the west of Otter House	0	0	-0.462	+0.439	-0.719	+0.833
River Coquet - beneath A1 existing bridge	0	0	-0.543	+0.480	-0.654	+0.775
River Coquet - Upstream of Felton Old Bridge	+0.001	-0.001	-0.299	+0.278	-0.491	+0.618

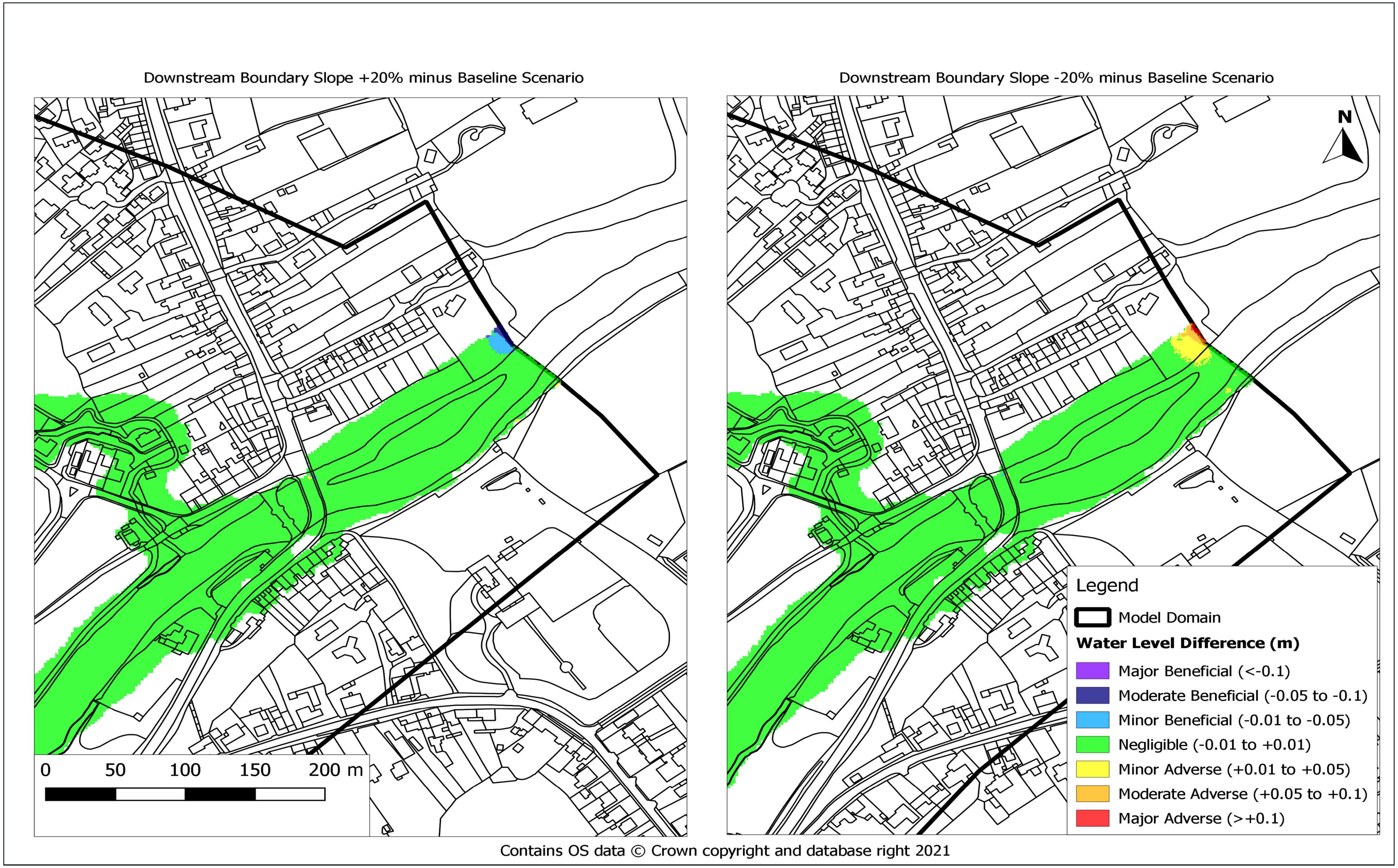

Figure 9-1 - Water Level Difference – Downstream Boundary Slope Sensitivity Tests Minus Baseline

9.5 CONSTRUCTION SCENARIO TEMPORARY BRIDGE FORM LOSS COEFFICIENT SENSITIVITY TESTS

9.5.1. There is no technical guidance on form loss values to use for bridges decks when using TUFLOW 2d_lfcsh polygons. TUFLOW has undertaken model calibration exercises which suggest using a fixed value of 0.4 (used for the temporary bridge deck in the Construction Scenario) where flow conditions are expected to drown out the structure. A sensitivity test has been undertaken by running the model with a form loss value of zero to assess the impact of this variable on model results.

9.5.2. Hydraulic model results show that maximum water levels immediately upstream of the temporary bridge do not change significantly with a form loss of value of 0.4 or zero for the bridge deck. In the 0.5% AEP plus 65% climate change flow uplift event , the maximum water level immediately upstream of the temporary bridge is 38.348m AOD (0.348m above the temporary bridge deck) with a form loss value of 0.4, this maximum water level is reduced by 1mm with a form loss value of zero.

9.5.3. The form loss value for the bridge deck is implemented once the bridge deck has become submerged, therefore, for lower magnitude events, this is not expected to impact on model results.

9.5.4. Based on the above it is considered that the model results have a low sensitivity to the changes of the temporary bridge deck form loss value.

9.6 OPERATIONAL SCENARIO PROPOSED PIERS REPRESENTATION SENSITIVITY TESTS

9.6.1. There are different methods that can be used to represent bridge piers[5]. TUFLOW 2d_lfcsh polygons have been used given the 2D model grid resolution (2m cell size). A sensitivity test has been undertaken by representing the proposed south pier using a TUFLOW 2d_zshape polygon to block out the cells (raising elevations) within the pier footprint.

9.6.2. Figure 9-3 shows the model results for the area in the vicinity of the proposed south pier using TUFLOW 2d_lfcsh and TUFLOW 2d_zshape polygons. As shown in this figure, model results immediately adjacent to the pier experience a change. Table 9-2 shows the difference in model results at point inspections locations (see Figure 9-2) immediately adjacent to the pier. Model results further away from the south pier do not change significantly. It is considered that the model results immediately adjacent to the pier are sensitive to the pier representation method used.

[5] TUFLOW, Modelling Bridge Piers and Afflux in TUFLOW, https://downloads.tuflow.com/_archive/Technical_Memos/Modelling%20Bridge%20Piers%20in%202D%20using%20TUFLOW.pdf

Table 9-2 - Model Results Using TUFLOW 2d_lfcsh and TUFLOW 2d_zshape at Inspection Locations

	TUFLOW 2d_lfcsh Scenario				TUFLOW 2d_zshape Scenario			
Location	Maximum Water Depth (m)	Maximum Flow Velocity (m/s)	Maximum Bed Shear Street (lbf/ft²)	Maximum Froude Number	Maximum Water Depth (m)	Maximum Flow Velocity (m/s)	Maximum Bed Shear Street (lbf/ft²)	Maximum Froude Number
North	1.94	2.92	16.73	1.79	0.80	0.68	3.50	2.71
East	2.12	3.40	23.99	0.85	0.75	2.30	11.52	3.53
South	2.16	1.74	10.95	1.28	2.06	2.21	18.40	1.33
West	2.34	2.84	17.30	0.95	2.32	2.69	18.55	0.71
Difference (TUFLOW 2d_zshape Scenario Minus TUFLOW 2d_lfcsh Scenario)								
North					-1.14	-2.24	-13.23	0.92
East					-1.37	-1.10	-12.47	2.68
South					-0.09	0.47	7.45	0.04
West					-0.01	-0.14	1.26	-0.24

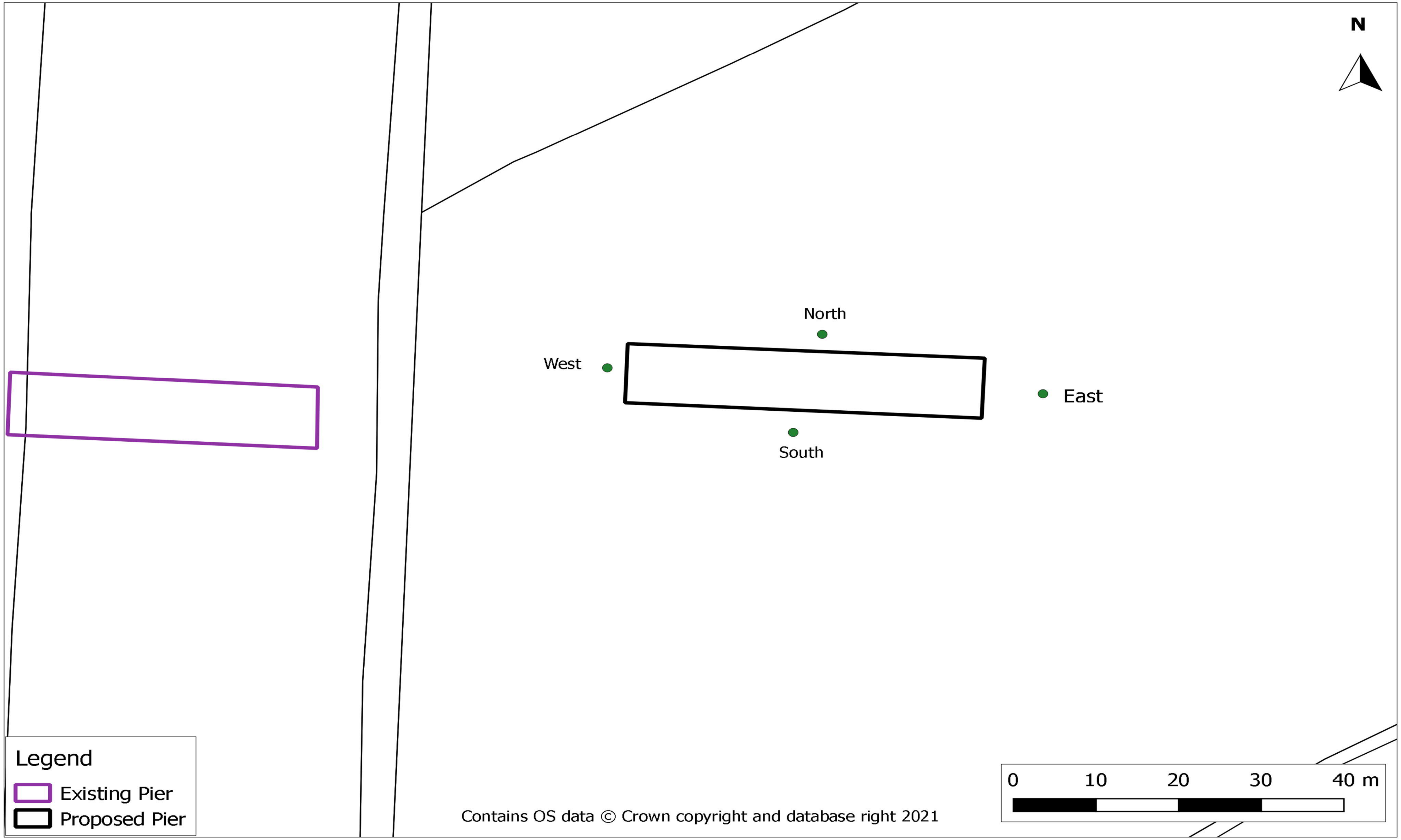

Figure 9-2 - Proposed South Pier Inspection Locations

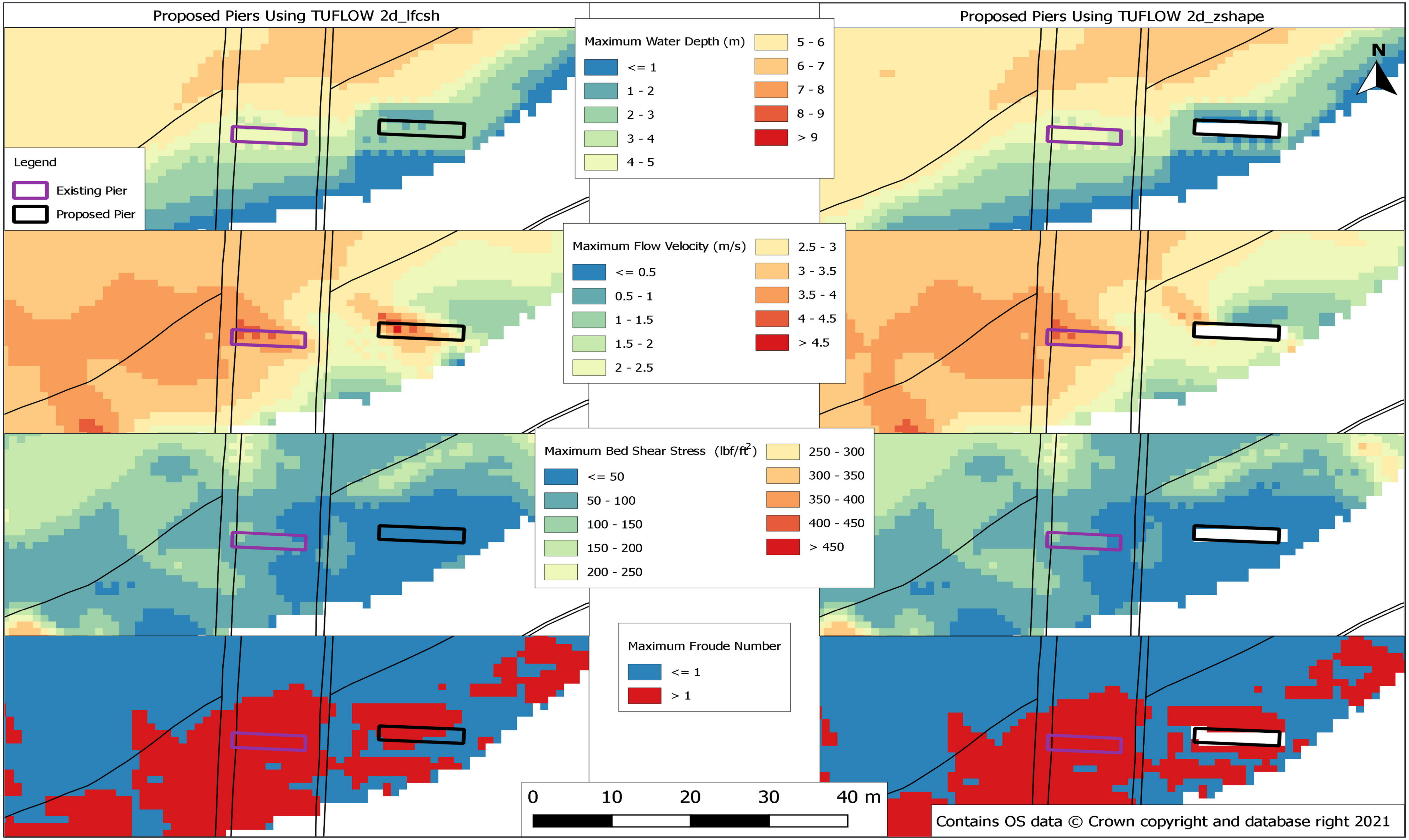

Figure 9-3 - Model Results Using TUFLOW 2d_lfcsh and TUFLOW 2d_zshape

10 ASSUMPTIONS AND LIMITATIONS

10.1.1. The accuracy and validity of the model results is heavily dependent on the accuracy of the hydrological and topographic data included in the model. Whilst the most appropriate available information has been used to construct the model, there are assumptions and limitations associated with 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 generally conservative for modelled water levels at the scheme location. Additionally, the sensitivity analysis has quantified the magnitude of potential uncertainty by examining the response to changes in key variables and that the modelling results are consistent with expected changes in these parameters.

10.1.3. The assumptions and limitations are listed below.

a. The model topography is mostly based on bathymetry data and LiDAR DTM data. Ground levels differ between both datasets where they overlap. Therefore, ground levels have been blended at the edges of the bathymetry survey extent (within a 2.5m buffer) to reconcile bathymetry survey levels with LiDAR DTM data and avoid sudden changes in ground levels.
b. Channel roughness and channel valley sides roughness is based on values agreed between the Environment Agency and the Applicant, as evidence in Appendix B of the River Coquet Geomorphology Modelling Assessment [REP3-009] .
c. There is no survey data available for Felton Old Bridge and Felton New Bridge, therefore, their representation has been assumed using MasterMap data, LiDAR DTM data and photographs. The assumptions made to enable representation of the bridges in Felton have not been sensitivity tested as the results would remain relative (if not accurate). Any sensitivity testing of the bridge parameters is considered unlikely to result in a material change to the assessment of impact at this location (where the 0.1% AEP maximum water level is more than 1m below the deck of the bridge) or elsewhere within the model.
d. Sensitivity tests undertaken on the Baseline Scenario (as presented in Sections 9.2 to 9.4) have shown that the Baseline model result are sensitive to Manning’s ‘n’ roughness values and the hydrological inflows used.
e. The model has not been calibrated or verified against observed events due to lack of measured flood datasets within the study area. A comparison of the model results have been made against recorded flood outlines which show that the model results are reasonable.
f. There are different methods that can be used to represent piers[5]. TUFLOW 2d_lfcsh polygons have been used given the model resolution (2m cell size). Hydraulic model results are expected to change in the vicinity of the pier depending on the method adopted to represent piers as presented in Section 9.6. The model results in the vicinity of the piers (in both Baseline and Operational Scenarios) are considered by the design engineers to be acceptable using professional judgement to be used to undertake an assessment of the scour protection needed for the piers, however, Computational Fluid

Dynamics (CFD) modelling may be undertaken for a detailed design assessment of the piers' scour protection system to refine elements of the design detail.

g. There is no technical guidance on the definition of form loss values for bridges decks when using TUFLOW 2d_lfcsh polygons. TUFLOW has undertaken model calibration exercises which suggest using a fixed value of 0.4 (used for the temporary bridge deck in the Construction Scenario) where flow conditions are expected to drown out the structure. A higher form loss value of 1.5 is suggested by the software manual[6] for orifice flow conditions (use for Felton New Bridge and Felton Old Bridge). For the temporary bridge, the use of a form loss value of 0.4 has been found to have negligible impact on model results as presented in Section 9.5.

[6] BMT, TUFLOW Classic/HPC User Manual Build 2018-03-AD

11 MODEL USER GUIDE

11.1 HYDRAULIC MODEL FILES

11.1.1. A list of the hydraulic model files and associated model results are provided along with references to the input data used to represent the different scenarios (see Table 11-1) to support external review of the hydraulic model should this be required. Sections 11.2 and 11.3 show the model folder structures and description of TUFLOW files.

Table 11-1 - Hydraulic Model Files

Data	Description
RiverCoquetHydraulicModel.zip	This file contains the TUFLOW model files – Baseline Scenario.
RiverCoquetHydraulicModelResults.zip	This file contains the model results for each simulation – Baseline Scenario.
RiverCoquetHydraulicModel_23042021.zip	This file contains the TUFLOW model files – Construction and Operational Scenarios.
RiverCoquetHydraulicModelResults_ConstructionScenario.zip	This file contains the model results for each simulation – Construction Scenario.
RiverCoquetHydraulicModelResults_OperationalScenario.zip	This file contains the model results for each simulation – Operational Scenario.
RiverCoquetHydraulicModelResults_SensitivityTests.zip	This file contains the model results for each simulation undertaken as part of the sensitivity testing exercise.
ModelLog.xlxs	Model log for the Baseline, Construction, Operational and Sensitivity Test Scenarios model simulation for all events simulated to this date.
HE551459-JAC-EWE-M2F_S03_NS39363-RP-GI-0001.pdf	This document which provides details on the model build for the Baseline, Construction and Operational Scenarios.
SurveyData.zip	▪ Bathymetric survey (March 2021) ▪ Riverbank survey data (February 2021) ▪ Costain topographical survey

Data	Description
18122020_dtm_1m_trim.asc	1m horizontal resolution LiDAR DTM downloaded on the 8th of December 2020 from http://environment.data.gov.uk/ds/survey/#/survey
North_bank_temp_DTM_final.tif	DTM representing the temporary works on the north bank in the Construction Scenario.
South_bank_temp_DTM_final.tif	DTM representing the temporary works on the south bank in the Construction Scenario.
South_bank_Proposed_DTM_FINAL.tif	DTM representing the proposed topographic modification to the south bank in the Operational Scenario.

11.2 MODEL FOLDER STRUCTURE

11.2.1. The following figures show the model folder structure. Figure 11-1 shows the _TUFLOW folder (which contains the model files) and simulation folders. The simulation folders shown are for the Baseline Scenario simulations only, similar folders are created for the Construction and Operational Scenarios and Sensitivity Tests.

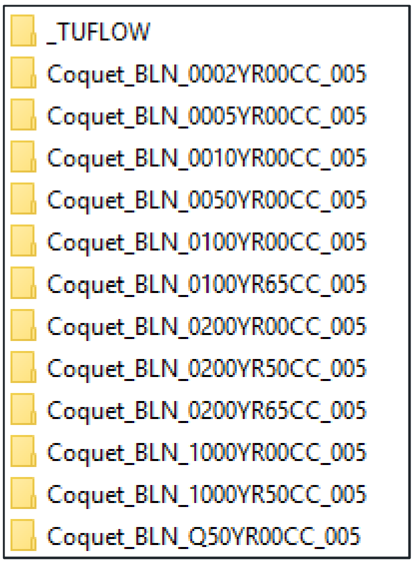

Figure 11-1 - TUFLOW folder and Results Folders

11.2.2. Figure 11-2 shows the _TUFLOW folder which contains the boundary condition database folder and the model files.

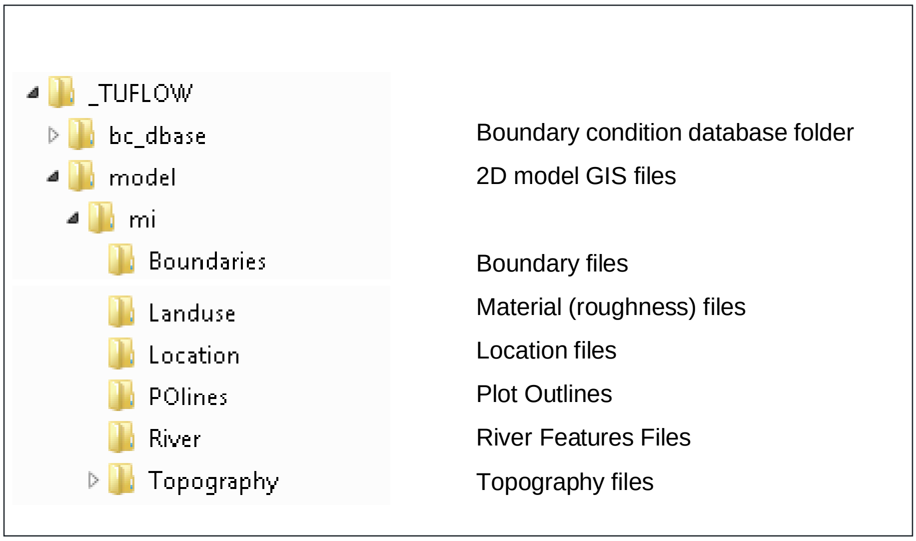

Figure 11-2 - _TUFLOW Folder Structure

11.2.3. Figure 11-3 shows an example simulation folder for the Baseline scenario 0.1% AEP present day event (version 5 model). The TUFLOW control file (.tcf file) is stored within this folder. The same folder structure has been used for all simulations undertaken, including the Construction Scenario and Operational Scenario and Sensitivity Tests simulations.

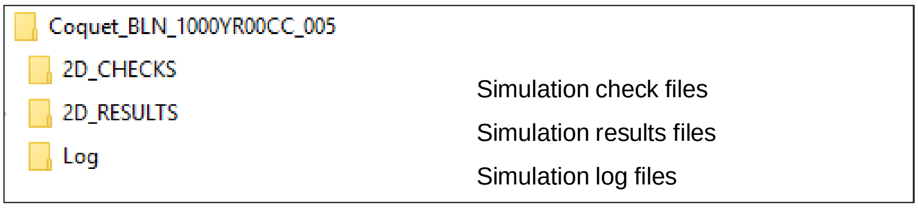

Figure 11-3 - Simulation Folder Structure

11.3 TUFLOW FILE DESCRIPTION

11.3.1. Figure 11-4 shows the description of the TULFOW files for the Baseline scenario 0.1% AEP present day event. It should be noted that all Baseline scenario simulations make use of the same TUFLOW geometry control file (.tgc file) and TUFLOW boundary control file (.tbc file).

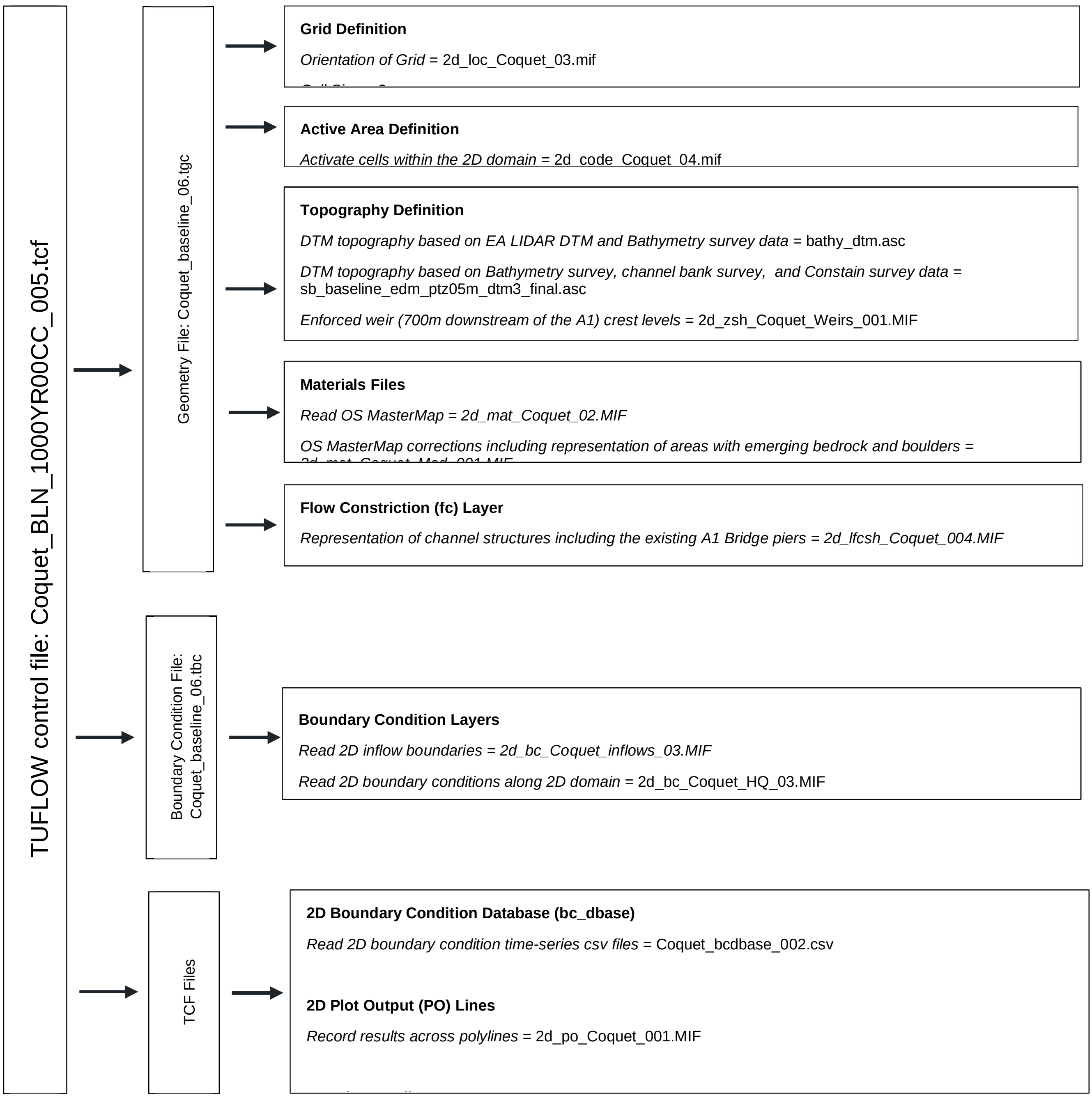

Figure 11-4 - TUFLOW File Description- Baseline Scenario

11.3.2. Figure 11-5 shows the description of the TULFOW files for the Construction scenario 0.1% AEP present day event. It should be noted that all Construction scenario simulations make use of the same TUFLOW geometry control file (.tgc file) and TUFLOW boundary control file (.tbc file).

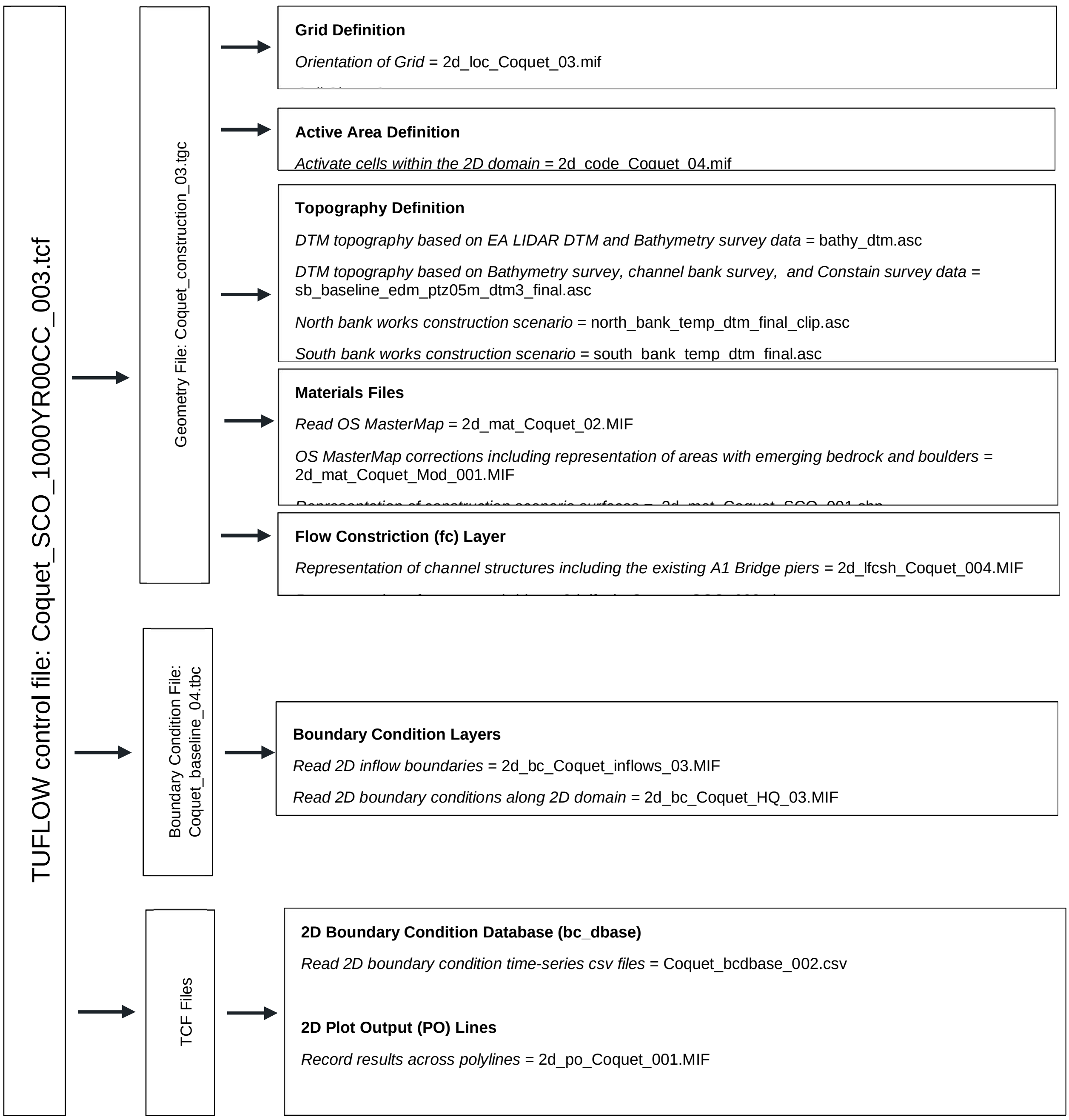

Figure 11-5 - TUFLOW File Description- Construction Scenario

11.3.3. Figure 11-6 shows the description of the TULFOW files for the Operational scenario 0.1% AEP present day event. It should be noted that all Operational scenario simulations make use of the same TUFLOW geometry control file (.tgc file) and TUFLOW boundary control file (.tbc file).

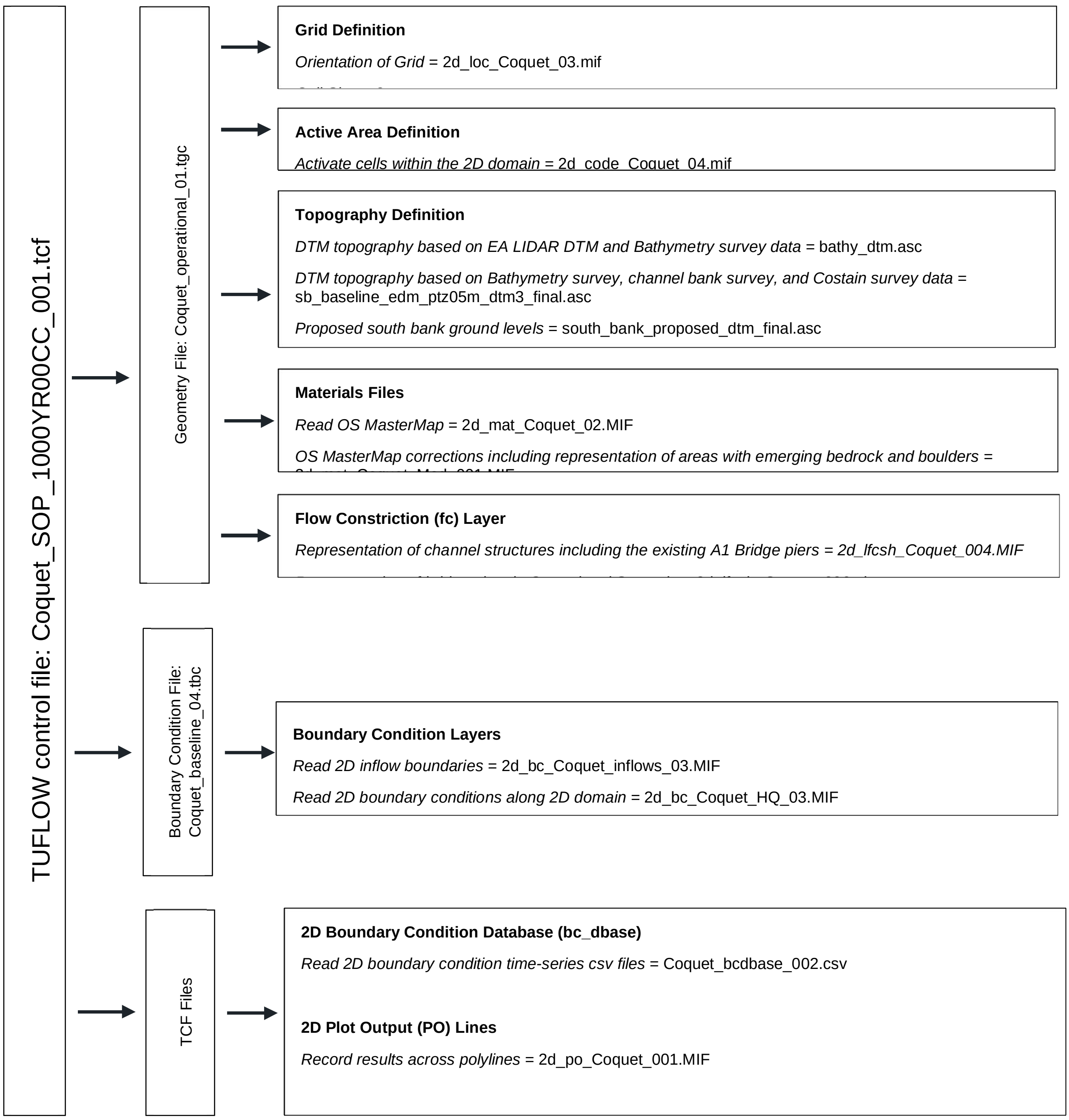

Figure 11-6 - TUFLOW File Description- Operational Scenario

11.3.4. Figure 11-7 shows the description of the TUFLOW files for the roughness +/-20% sensitivity test scenarios. When compared to the Baseline Scenario TUFLOW files, the only files changed are the .tmf files.

TUFLOW control file: Coquet_M20R_0200YR65CC_001.tcf or Coquet_P20R_0200YR65CC_001.tcf

Geometry File: Coquet_baseline_06.tgc

Grid Definition

Orientation of Grid = 2d_loc_Coquet_03.mif

Cell Size = 2m

Active Area Definition

Activate cells within the 2D domain = 2d_code_Coquet_04.mif

Topography Definition

DTM topography based on EA LIDAR DTM and Bathymetry survey data = bathy_dtm.asc

DTM topography based on Bathymetry survey, channel bank survey, and Constain survey data = sb_baseline_edm_ptz05m_dtm3_final.asc

Enforced weir (700m downstream of the A1) crest levels = 2d_zsh_Coquet_Weirs_001.MIF

Materials Files

Read OS MasterMap = 2d_mat_Coquet_02.MIF

OS MasterMap corrections including representation of areas with emerging bedrock and boulders = 2d_mat_Coquet_Mod_001.MIF

Flow Constriction (fc) Layer

Representation of channel structures including the existing A1 Bridge piers = 2d_lfcsh_Coquet_004.MIF

Boundary Condition File: Coquet_baseline_06.tbc

Boundary Condition Layers

Read 2D inflow boundaries = 2d_bc_Coquet_inflows_03.MIF

Read 2D boundary conditions along 2D domain = 2d_bc_Coquet_HQ_03.MIF

TCF Files

2D Boundary Condition Database (bc_dbase)

Read 2D boundary condition time-series csv files = Coquet_bcdbase_002.csv

2D Plot Output (PO) Lines

Record results across polylines = 2d_po_Coquet_001.MIF

Roughness File

Roughness definition = Coquet_03_M20R.tmf (for roughness reduced by -20%) or

Roughness definition = Coquet_03_P20R.tmf (for roughness increased by +20%)

Figure 11-7 - TUFLOW File Description – Roughness +/-20% Sensitivity Tests

11.3.5. Figure 11-8 shows the description of the TUFLOW files for the downstream boundary slope +/-20% sensitivity test scenarios. When compared to the Baseline Scenario TUFLOW files, the .tbc files and 2d_bc_Coquet_HQ files have been changed.

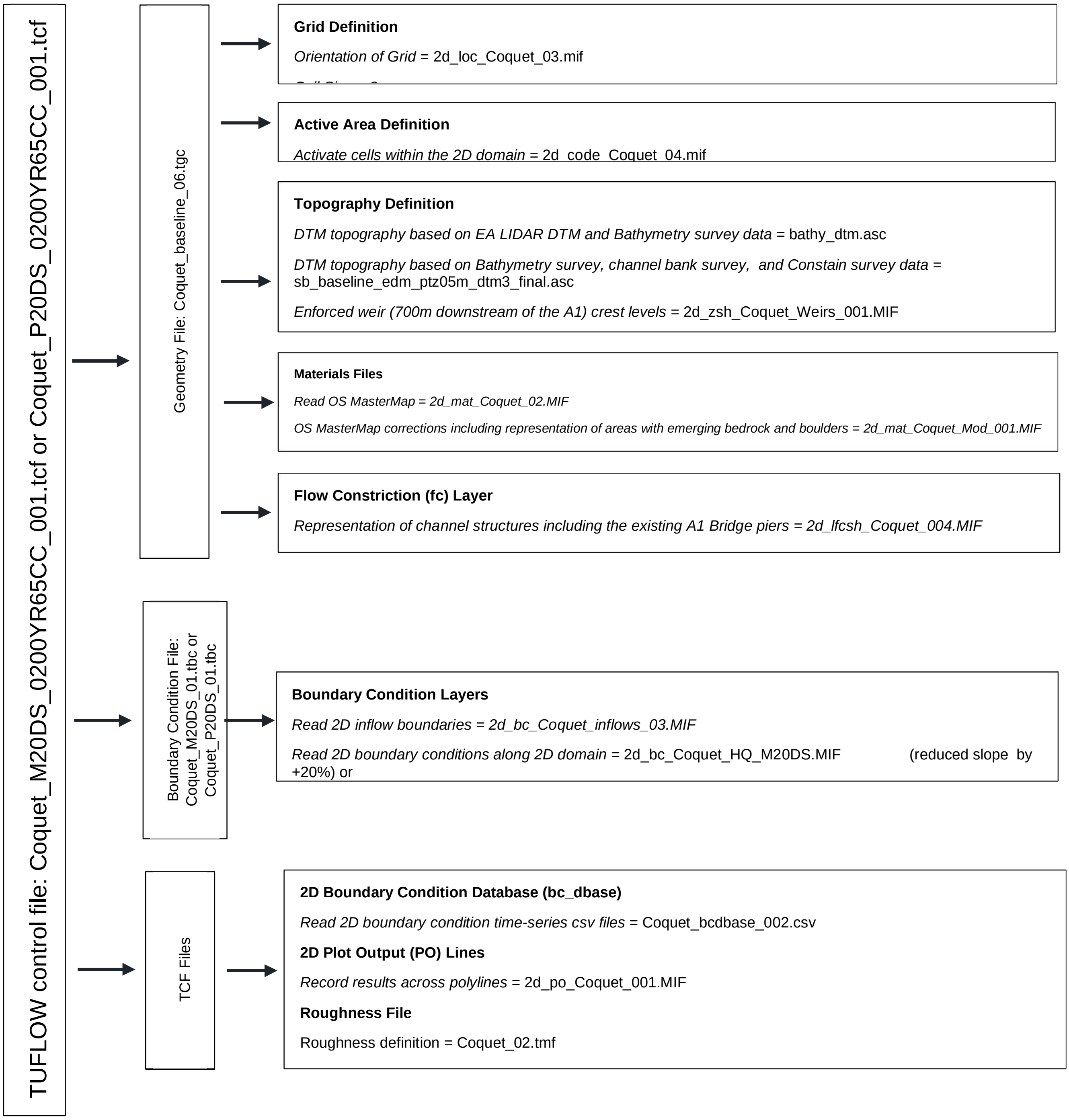

Figure 11-8 - TUFLOW File Description – Downstream Boundary Slope +/-20% Sensitivity Tests

11.3.6. Figure 11-9 shows the description of the TUFLOW files for the Construction Scenario sensitivity test scenario with a form loss of zero for the bridge deck.

TUFLOW control file: Coquet_SCO_1000YR00CC_002.tcf

→ Geometry File: Coquet_construction_02.tgc

→ **Grid Definition**

Orientation of Grid = 2d_loc_Coquet_03.mif

→ **Active Area Definition**

Activate cells within the 2D domain = 2d_code_Coquet_04.mif

→ **Topography Definition**

DTM topography based on EA LIDAR DTM and Bathymetry survey data = bathy_dtm.asc

DTM topography based on Bathymetry survey, channel bank survey, and Constain survey data = sb_baseline_edm_ptz05m_dtm3_final.asc

North bank works construction scenario = north_bank_temp_dtm_final_clip.asc

South bank works construction scenario = south_bank_temp_dtm_final.asc

→ **Materials Files**

Read OS MasterMap = 2d_mat_Coquet_02.MIF

OS MasterMap corrections including representation of areas with emerging bedrock and boulders = 2d_mat_Coquet_Mod_001.MIF

→ **Flow Constriction (fc) Layer**

Representation of channel structures including the existing A1 Bridge piers = 2d_lfcsh_Coquet_004.MIF

→ Boundary Condition File: Coquet_baseline_04.tbc

→ **Boundary Condition Layers**

Read 2D inflow boundaries = 2d_bc_Coquet_inflows_03.MIF

Read 2D boundary conditions along 2D domain = 2d_bc_Coquet_HQ_03.MIF

→ TCF Files

→ **2D Boundary Condition Database (bc_dbase)**

Read 2D boundary condition time-series csv files = Coquet_bcdbase_002.csv

2D Plot Output (PO) Lines

Record results across polylines = 2d_po_Coquet_001.MIF

Figure 11-9 - TUFLOW File Description- Construction Scenario Sensitivity Test – Bridge Deck Form Loss

11.3.7. Figure 11-10 shows the description of the TUFLOW files for the Operational Scenario pier sensitivity test (using a TUFLOW 2d_zhape polygon to block out cells within the pier footprint).

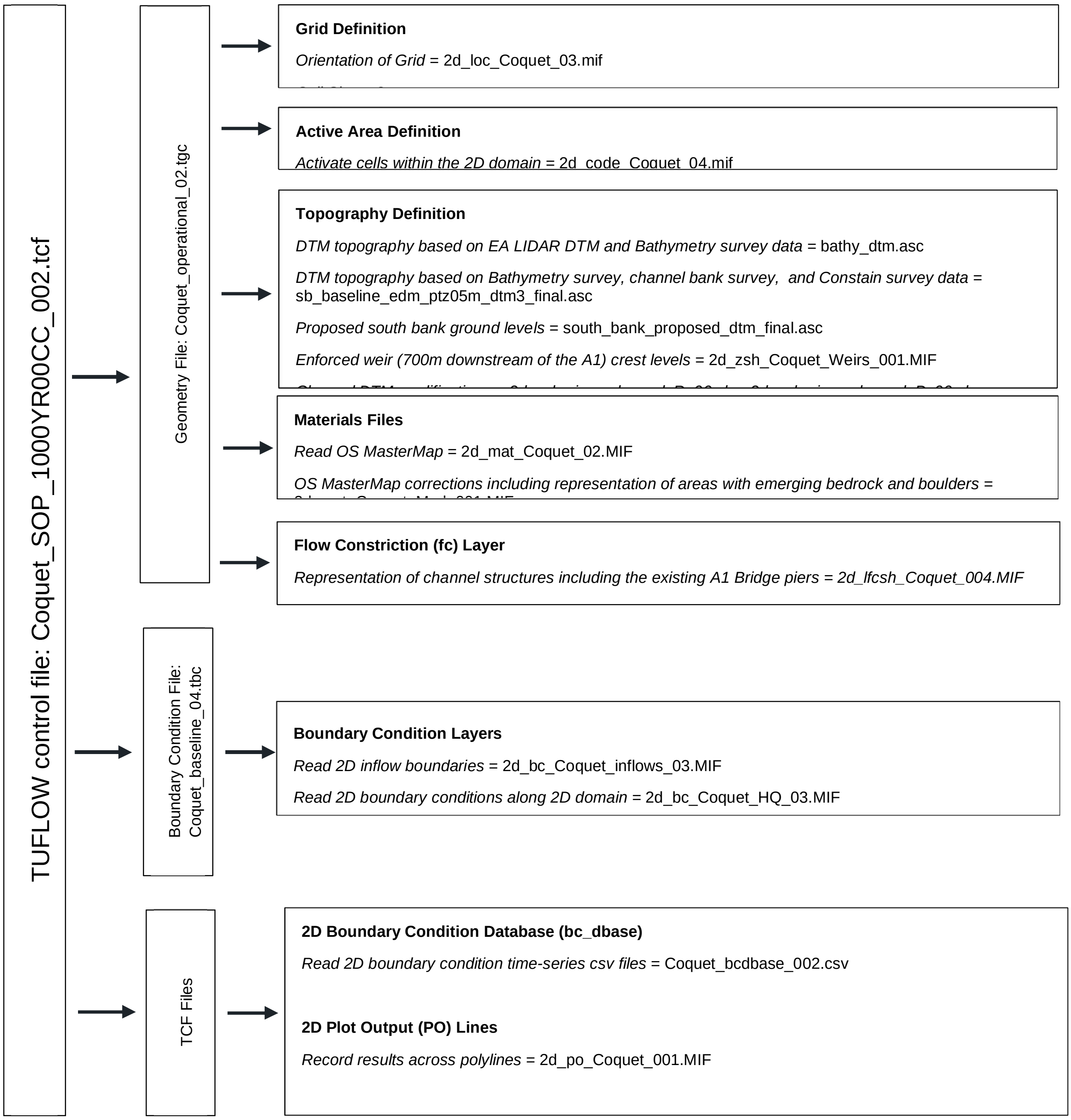

Figure 11-10 - TUFLOW File Description- Operational Scenario – Pier Representation Sensitivity Test

Appendix A

FLOOD ESTIMATION REPORT

Flood estimation report: A1 in Northumberland, River Coquet

Introduction

This report template is a supporting document to the Environment Agency's Flood Estimation Guidelines. It provides a record of the hydrological context, the method statement, the calculations and decisions made during flood estimation and the results. This document has been used for one site.

Where relevant, references to specific sections of the Flood Estimation Guidelines document are included to indicate where further useful information can be found.

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Approval

Revision stage	Analyst / Reviewer name & qualifications	Amendments	Date
Method statement originator	SR		Jan 2021
Method statement checker	CM	Recommendations on Flow Estimation Points	Jan 2021
Method statement reviewer	CS	Minor typographical amendments and clarifications	Jan 2021
Initial calculations originator	CM		Jan 2021
Initial calculations checker	SH	Minor recommendations to amend pooling group	Feb 2021
Initial calculations reviewer	KF	Minor typographical amendments and clarifications	Mar 2021

Abbreviations

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

1 SUMMARY OF ASSESSMENT

1.1 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections. The aim of the table is to enable quick and easy identification of the type of assessment undertaken.

Catchment location	The subject site is a new (additional) bridge over the River Coquet, upstream of Felton Village in Northumberland (approximate National Grid Reference NZ 17437 99795).
Purpose of study and scope	Highways England are developing a project to dual the A1 between Morpeth and Ellingham in Northumberland. To facilitate this, a new (additional) bridge over the River Coquet (approximate National Grid Reference NZ 17437 99795) is required. The new bridge will be located immediately downstream (east) of the existing bridge. The River Coquet, in the vicinity of the bridge, has not been modelled previously, consequently to support the design of the bridge and associated river engineering (erosion protection measures) a new 2D hydraulic model of a reach of the River Coquet is to be developed. The model is to be used to; • inform the fluvial geomorphological assessment required as part of the design of a new bridge and associated erosion protection measures for the operational and construction phases of the development; and, • confirm the flood levels during the operation and construction phase for sensitive receptors. The aim of this study is to produce hydrological estimates for a range of Annual Exceedance Probability (AEP) events; 50% (QMED), 20%, 10%, 2%, 1%, 0.5% (and 0.1%. Climate change uplifts to flows will be applied for the 0.5% AEP event) for application within the hydraulic model. The work is estimated to be a routine hydrological assessment.
Key catchment features	The catchment to the bridge site (NGR: NZ 17437 99795), is a predominantly rural (URBEXT2000 = 0.002) and impermeable catchment (BFIHOST = 0.394). There are no notable impacts from reservoirs (FARL = 0.99).
Flooding mechanisms	The primary source of flooding to the subject site is fluvial from the River Coquet. The site lies within Flood Zone 3 as shown on the gov.uk fluvial flood map ('Flood Map for Planning').
Gauged / ungauged	The bridge lies between two flow gauging sites on the River Coquet; Coquet at Rothbury (National River Flow Archive (NRFA) No: 22009) and Coquet at Morwick (NRFA No: 22001). These sites lie 5.4km upstream and 3.3km downstream, respectively, of the bridge therefore not within the vicinity of the subject site.
Final choice of method	The FEH Statistical method and ReFH2.3 will be used to produce hydrological estimates.
Key limitations / uncertainties in results	There are two flow gauging sites upstream and downstream of the subject site as discussed above however there are no flow or level gauge data within the vicinity of the bridge to increase confidence in flow estimates.

1.2 Note on flood frequencies

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. Results tables in this document contain both return period and AEP titles;

The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

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
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

2 METHOD STATEMENT

2.1 Requirements for flood estimates

Overview	Highways England are undertaking a project to dual the A1 between Morpeth and Ellingham in Northumberland. To facilitate this a new (additional) bridge over the River Coquet (approximate National Grid Reference NZ 17437 99795) is required. The new bridge will be located immediately downstream (east) of the existing bridge. The River Coquet is an Environment Agency (EA) designated main river. The Fence Burn flows into the River Coquet at the upstream end of the model extent and the Back Burn flows into the River Coquet just upstream of Felton and the downstream end of the model extent. **Error! Reference source not found.** shows the hydraulic model domain with the upstream model extent upstream of the confluence with Fence Burn and the downstream model extent at Felton. The River Coquet which flows under the bridge has not been modelled previously, consequently, to support the design of the bridge and associated river engineering (erosion protection measures) a new 2D hydraulic model of the River Coquet, as shown in Figure 1, is to be developed. The aim of this study is to produce hydrological estimates for a range of events for application within the hydraulic model. Hydrographs are required for the following Annual Exceedance Probability (AEP) events; 50% (QMED), 20%, 10%, 2%, 1%, 0.5% and 0.1%. Climate change uplifts to flows will be applied for the 0.5% AEP event. In line with current Environment Agency Guidance (EA, June 2020) an uplift of 50% will be applied to flows to assess the upper end climate change allowance in the Northumbria river basin district, as well as an uplift of 65% to assess the H++ climate change allowance for climate change for the '2080s'. Flow estimates will be made at three Flow Estimation Points (FEP) and hydrographs will be created for three Inflow Points to account for two tributaries entering the River Coquet (Fence Burn and Back Burn). The flow estimate for the lumped catchment to the bridge (subject site) will be used for flow reconciliation within the hydraulic model. See Figure 2 and Figure 3 for locations of these FEPs and Inflow Points. Flow estimates will also be developed for the largest recorded flow on the River Coquet through a review of the upstream and downstream gauges at Coquet at Rothbury (NRFA No: 22009) and Coquet at Morwick (NRFA No: 22001), respectively (Figure 4).
Project scope	There is no flow or level gauging within the vicinity of the subject site (bridge) however the bridge location is approximately equidistant from two flow gauging sites on the River Coquet at; Rothbury (NRFA No: 22009) and Morwick (NRFA No: 22001) upstream and downstream (respectively) of the bridge (Figure 4). A review of the flood history in Felton village which is within the model reach will be undertaken to assess whether flood events could be related to the annual maximum (AMAX) datasets at the upstream (Coquet at Rothbury) and downstream (Coquet at Morwick) gauges for use in a hydraulic model calibration exercise. As part of work supporting a Development Consent Order for the project, WSP has previously undertaken a Manning's equation assessment of flood levels and scour potential at the site using publicly available flow data. This has included calculation of flood levels for the maximum flow recorded at Morwick (525 m^3/s on 07/09/2008).

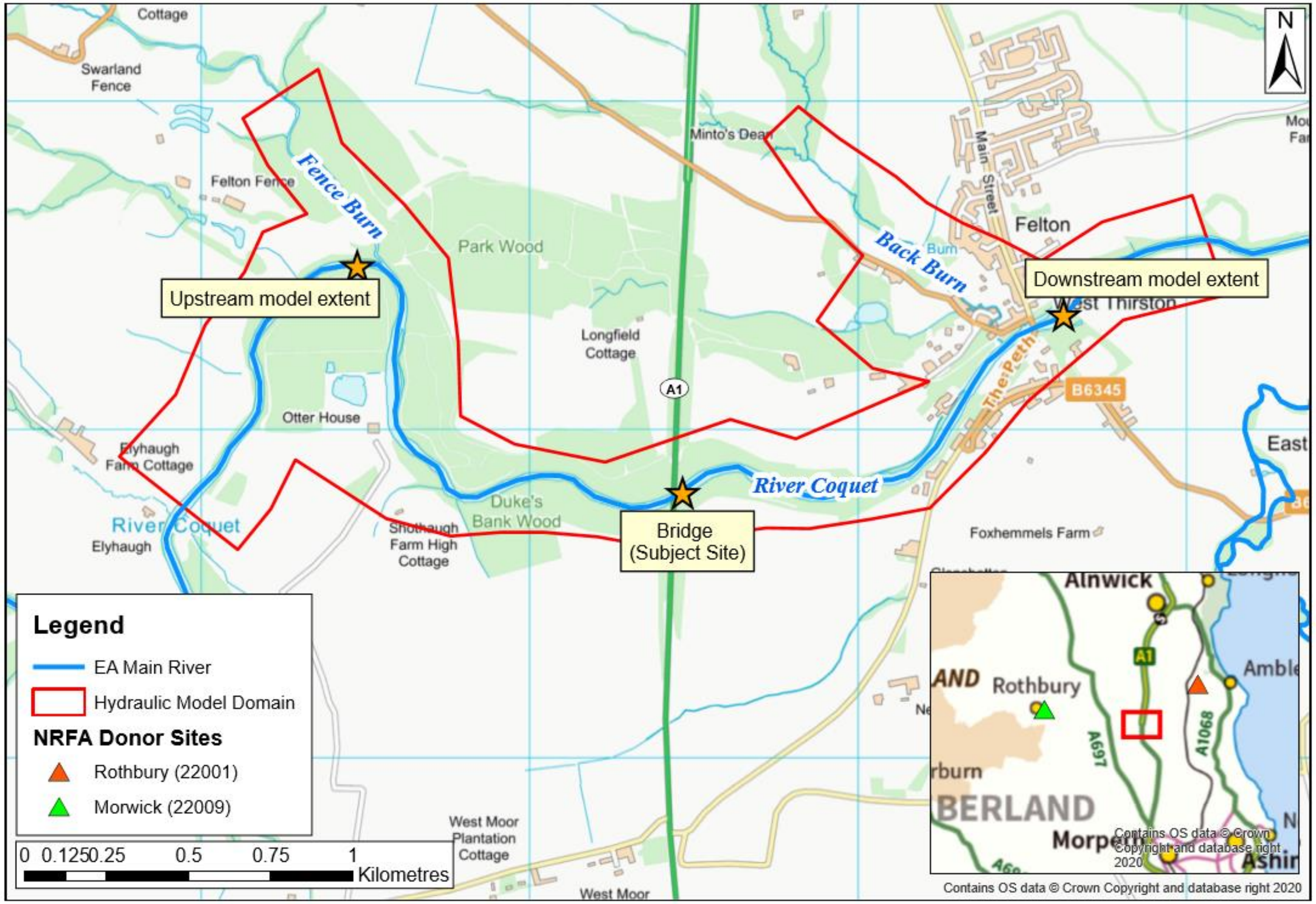


Figure 1: Location Plan

2.2 The catchment

The subject site lies on the River Coquet, a main river upstream of Felton Village in Northumberland (approximate National Grid Reference NZ 17437 99795). The Fence Burn flows into the River Coquet at the upstream end of the model extent and the Back Burn flows into the River Coquet just upstream of Felton and the downstream end of the model extent.

The bridge (subject site) lies approximately 11km west of the coast (direct path) and approximately 20km upstream of the coast (along the River Coquet watercourse). The bridge lies approximately 17km upstream from the Normal Tidal Limit on the River Coquet therefore the subject site is not considered to be tidally influenced.

The total catchment (FEH catchment) to the downstream study extent has an area of 491 km² and a total catchment area of 486 km² to the bridge. The catchment is predominantly rural (URBEXT2000 = 0.002 for the catchment to the bridge) with the only urban area being the small village of Felton just downstream of the bridge.

The catchment is impermeable with a BFIHOST value of 0.393 to the bridge. The bedrock is dominated by Stainmore Formation (Mudstone, Siltstone And Sandstone) which is typical of rivers in a coastal setting. Bedrock in the vicinity of the River Coquet is overlain by a mix of superficial deposits including; Glaciofluvial Deposits (Sand And Gravel), River Terrace Deposits, Silt, Sand And Gravel and Alluvium (Clay, Silt, Sand And Gravel). The remaining catchment further from the watercourse is overlain by Till deposits (Diamicton) (BGS, 2021).
Soils within the catchment to the bridge are a combination of freely draining slightly acid loamy soils and slowly permeable seasonally wet soils (SoilScapes, 2021).

There is little attenuation due to reservoirs and lakes within the catchment. A small lake lies 1.1km upstream of the bridge at Shothaugh which shows some flooding along the River Coquet (EA Long Term Flood Risk Mapping, accessed 2021) however this is confined to the watercourse itself. There are no nearby reservoirs and the FARL value is over 0.90 (FARL = 0.992) therefore it is concluded that the risk from reservoirs and lakes are minimal and likely not to have a significant impact on flooding mechanisms at the subject site.

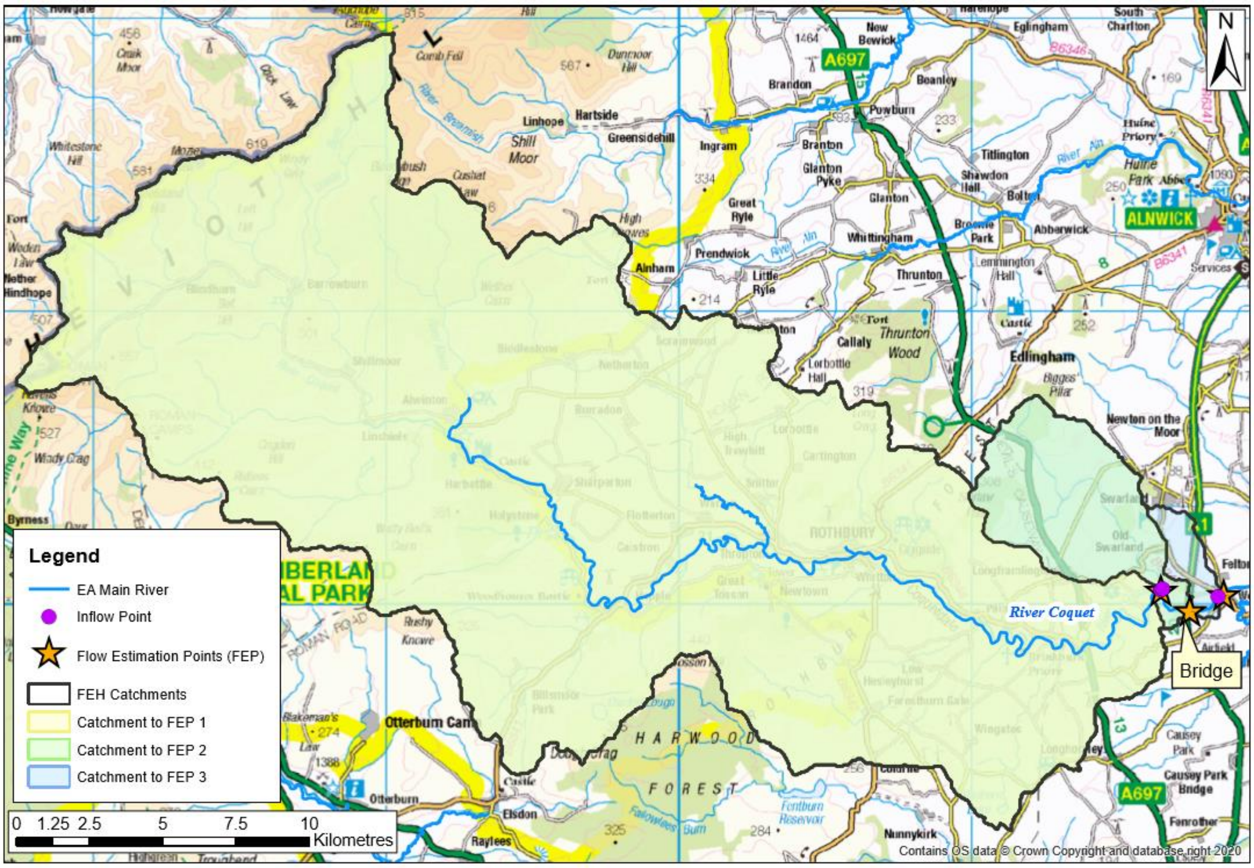


Figure 2: Hydrological Catchments

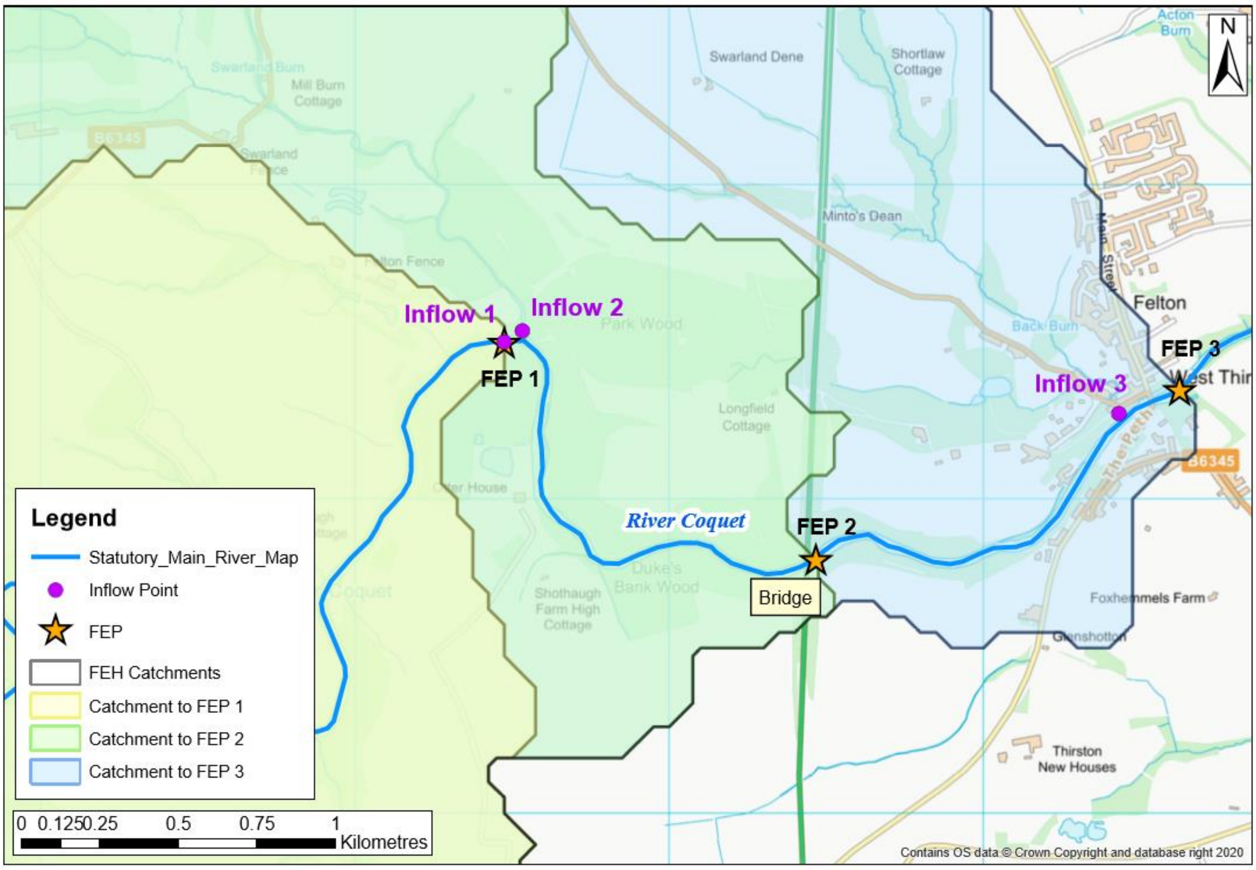


Figure 3: Hydrological Catchments (zoomed to subject site)

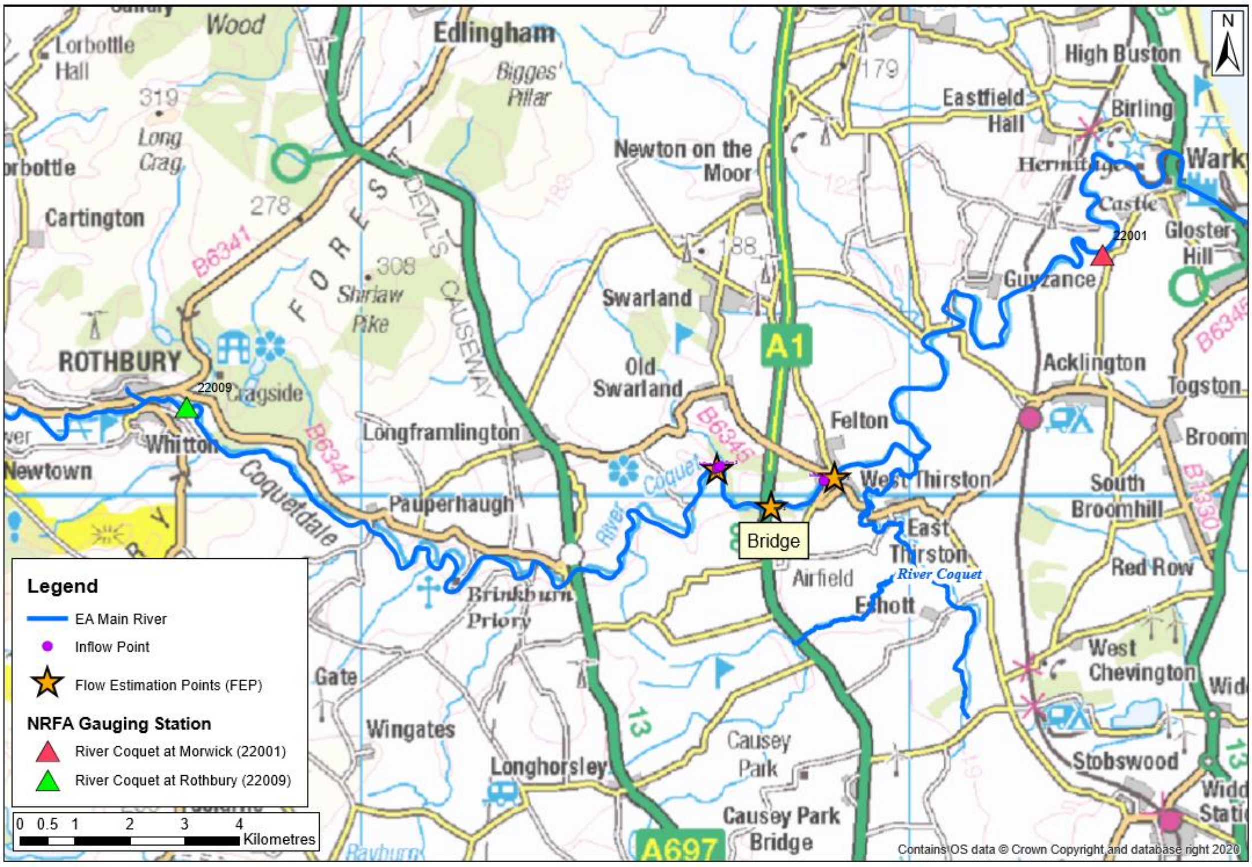


Figure 4: Nearby NRFA Gauge Stations

2.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released September 2020. This contains data up to water year 2018-19 plus provisional data for 87 stations for water year 2019/20 where exception flood events were experienced in November 2019 or February 2020.

2.4 Gauging stations (flow or level)

Water-course	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
River Coquet	Morwick	22	22001	569.8	Velocity area station	01/01/1963
River Coquet	Rothbury	22	22009	346	Velocity area station with cableway	01/04/1972

2.5 Data available at each flow gauging station in Table 2.4

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
Morwick	01/01/1963 - 01/10/2019	No	Yes	Yes	No	Data missing 18/02/2006 - 14/06/2006 due to station rebuild. Full period of record peak flow data reviewed and released in September 2019 Gauged above QMED Few high flow gaugings, however rating extrapolation has been informed by hydraulic model and thought to be reasonable.
Rothbury	01/04/1972 - 01/10/2019	No	Yes	Yes	No	Full period of record peak flow data reviewed and released in September 2019 Gauged above QMED Gauged beyond AMAX3. Well-confined section with no expected change in control or cross-section beyond QMED due to steep banks

2.6 Rating equations

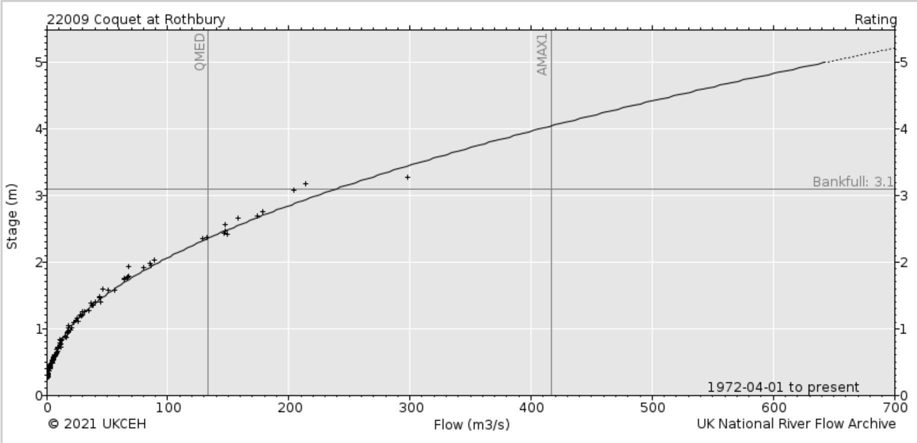
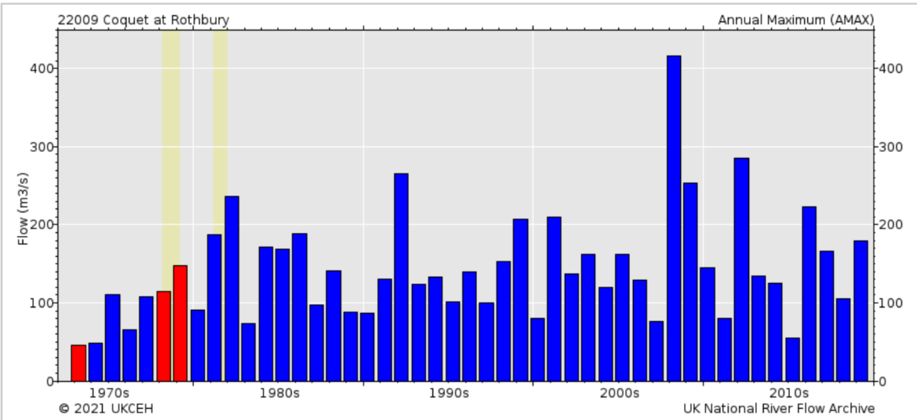
Station name	Type of rating	Rating review needed?	Comments and link to any rating reviews
Morwick	Rating extrapolation has been informed by hydraulic model and thought to be reasonable.	No	See Section 2.8 below for peak flow rating information.
Rothbury	One rating curve for entire dataset. Gauged beyond AMAX3 with little extrapolation required.	No	See Section 2.8 below for peak flow rating information.

2.7 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	No	No	N/A	N/A
Historical flood data	Yes	Yes	Northumbria Local Resilience Forum September 2008 (Northumberland County Council, 2008)	Flood history at Felton Village (within the model reach) has been reviewed to assess whether it could be related to an estimated / interpolated annual maximum (AMAX) record for Felton through interrogation of the annual maxima records at Coquet at Rothbury (22009) and Coquet at Morwick (22001). Only one record of the history of flooding at Felton Village was found on the Northumbria Local Resilience Forum September 2008. This stated that on 6 September 2008 'Approximately 7 residential properties in Felton were reported as flooded. No further historical flood data was available. This event corresponds with the highest recorded flows at Rothbury and Morwick gauging stations.
Flow or river level data for events	Yes	Yes	NRFA web service (last accessed February 2021)	There are no flow gaugings within the vicinity of the subject site however there are two gauging stations situated upstream and downstream of the site. Peak flow data at Rothbury and Morwick gauging stations upstream and downstream of the subject site were assessed. it was found that the highest recorded flow at Rothbury was on 7 September 2008 with a peak flow of 525.72m³/s and the highest recorded flow at Morwick was on 6 September 2008 with a peak flow of 416.56m³/s.
Rainfall data for events	No	No	N/A	N/A
Potential evaporation data	No	No	N/A	N/A

Results from previous studies	No	No	N/A	The River Coquet in the vicinity of the bridge has not been modelled previously.
Other data or information	No	No	N/A	N/A

2.8 Hydrological understanding of catchment

Plots of flow data and interpretation	N/A
Plots of flood peak data and interpretation	**22009 – Coquet at Rothbury** Good fit to gaugings, gauged close to AMAX2. The highest recorded flow occurred on 7 September 2008 with a peak flow of 525.72m³/s.  **Figure 5: Coquet at Rothbury (22009) Peak Flow Rating Information**  **Figure 6: Coquet at Rothbury (22009) Annual Maximum (AMAX) Data** **22001 – Coquet at Morwick** Data missing 18/02/2006 - 14/06/2006 due to station rebuild. Two peak flow ratings applied across period of record, the most recent is valid from 1976 and based upon an earlier version of rating C but with an additional upper limb informed by hydraulic modelling. Good fit to gaugings. Flood Studies Report grade A1. The highest recorded flow occurred on 6 September 2008 with a peak flow of 416.56m³/s.

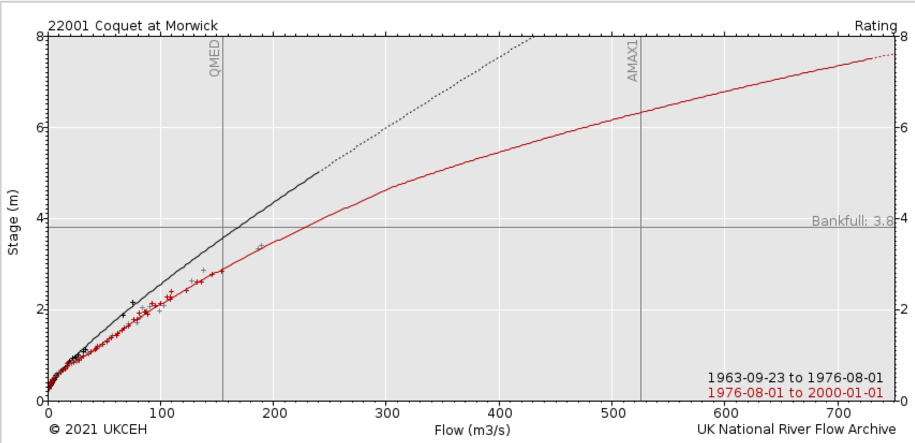


Figure 7: Coquet at Morwick (22001) Peak Flow Rating Information

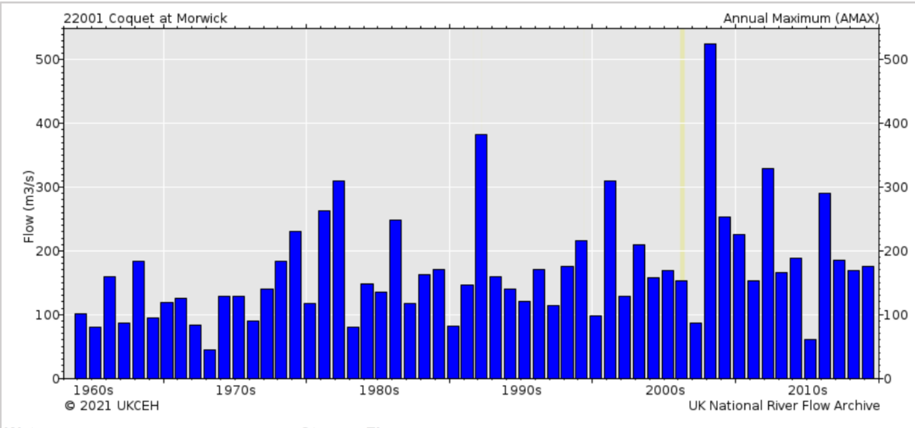


Figure 8: Coquet at Morwick (22001) Annual Maximum (AMAX) Data

Conceptual model	The subject site is the new bridge proposed over the River Coquet (NGR NZ 17437 99795). This will be located immediately downstream (east) of the existing bridge. The likely cause of flooding at the bridge site is fluvial flood risk from the River Coquet. Peak flows rather than volume are of interest.
Unusual catchment features	There are no unusual catchment features. The catchment is predominantly rural (URBEXT2000 = 0.002) and impermeable. There is little influence from reservoirs (FARL = 0.99).

2.9 Initial choice of approach

Is FEH appropriate?	The catchment is predominantly rural with no unusual features so FEH is appropriate. The EA Flood Estimation Guidance will be followed and the FEH statistical method and the ReFH2 will be used to derive hydrographs.
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?	The study aims to determine fluvial flood risk (mainly peak flows and levels) from the River Coquet and its tributaries to a proposed bridge on the A1 over the River Coquet. There are two gauging stations on the River Coquet upstream and downstream of the subject site however there are no flow or level gauging stations directly within the vicinity of the subject site.

The FEH statistical method will be used to derive peak flows and ReFH2.3 will be used to derive peak flows and hydrographs. The hydraulic model will then be reconciled so that design peak flows match the preferred hydrology estimates (FEH statistical or ReFH2.3).

Flow estimates will be made at three Flow Estimation Points (FEP). The catchments will be defined based on location of tributaries and catchment features (such as changes in permeability and urbanisation). Peak flows will be estimated at these FEPs using the FEH statistical method.

A total of three inflows are required to appropriately capture tributaries and the catchment draining to the River Coquet within the modelled reach. Flows to these locations will be created using ReFH2.3.

Table 1 identifies the Flow Estimation Points and Table 2 shows the catchments to Inflow Points.

Table 1: River Coquet Lumped Catchments to FEPs

Location	FEP	NGR	Purpose	Lumped Area (km²)
Lumped catchment (Catchment to Inflow 1) to upstream model extent on the River Coquet	FEP 1	NU 16450 00500	Inflow to 2D model	457
Lumped catchment to bridge	FEP 2	NZ 17437 99795	Flow reconciliation of hydraulic model runs	486
Lumped catchment to downstream model extent	FEP 3	NU 18600 00350	Check of flow estimates at the downstream point	491

Table 2: River Coquet Catchments to Inflow Points

Location	Inflow	NGR	Purpose	Lumped Area (km²)	Distributed Area (km²)
Lumped catchment to upstream model extent on the River Coquet	Catchment 1	NU 16450 00500	Inflow to 2D model	457	-
Fence Burn tributary (Inflow 2) lateral inflow to bridge	Catchment 2	NZ 17437 99795	Inflow to hydraulic model	27.46	28.94
Back Burn tributary (Inflow 3) lateral inflow from bridge to downstream model extent	Catchment 3	NU 18600 00350	Inflow to hydraulic model	3.90	4.62

See Figure 2 and Figure 3 for these catchment areas to FEP and Inflow Points.

Total catchments to each flow estimation point (FEP) will be purchased from the FEH web service as well as for the tributary inflow catchments (Fence Burn and Back Burn). For the intervening smaller inflow catchments, the catchment descriptors will be adjusted accordingly based on those for the respective tributary catchments t (as shown in Figure 2 and Figure 3). A statistical peak flow will be estimated at each FEP and compared against a ReFH2.3 estimate to then make a decision on the best estimate for use in the hydraulic model.

The methodology to complete this work is outlined below.

Produce catchment boundaries and best-estimates of catchment descriptors

The FEH web service will be used to purchase catchment descriptors for three catchments to the FEPs and two catchments to Inflow Points (as outlined in Table 1 and Table 2 above). Once downloaded FEH catchment boundaries will be checked against LiDAR Digital Terrain Model (DTM) data, OS 1:25,000 scale Explorer mapping and aerial photography. If FEH catchments are found to require adjustments there will also be some adjustment of catchments descriptors where appropriate.

The catchment area for Catchment 2 will include the upstream tributary (Fence Burn) and adjusted (increased) in area to account for the intervening area of the River Coquet to the bridge. Catchment 3 will include the downstream tributary (Back Burn) and adjusted (increased) in area to account for the intervening area downstream of the bridge. This will ensure the total area of the River Coquet upstream and downstream of the bridge site is captured (see Table 2 above for this adjustment to area).

The catchment descriptors for Catchment 1 (FEP1), Catchment 2 and Catchment 3 will be used to estimate the three point inflows to the hydraulic model.

QMED estimates and adjustment

The catchment does not have flow or level gauging in the vicinity of the subject site therefore QMED will be estimated from catchment descriptors at each FEP. Urban adjustment will be applied using the PRUAF equation based on BFIHOST. The application of a donor adjustment will be considered and applied if suitable.

The catchment does however have two gauging stations upstream and downstream of the modelled reach therefore the QMED estimates for these sites will be taken as a comparison to the subject site.

Growth curve definition

The FEH statistical analysis will be applied to develop a pooled growth curve for the lumped catchment to FEP 2 (catchment to the bridge and subject site).

As the catchment does not have flow or level gauging in

	the vicinity of the subject site, the growth curve will be estimated using pooling group analysis to express design flows for other return periods as a ratio over QMED. Growth curves will also be produced, using single site analysis, for the two gaugings stations upstream and downstream of the modelled reach for comparison with the subject site and to compare against the historical flood history at Felton Village. **Design inflows for 6 events** ReFH2.3 will be used to estimate design flood hydrographs for seven Annual Exceedance Probability (AEP%) events (50% (QMED), 20%, 10%, 2%, 1%, 0.5 and 0.1%). Climate change uplifts to flows will be applied for the 0.5% AEP event) to compare with the statistical peak flow estimate at FEP 2 (catchment to the bridge). A decision will be made on which peak flow (statistical method or ReFH2.3) will be used. The critical storm duration with be determined for a single target return period by identifying the duration producing the highest water level at FEP2 (the bridge). The critical duration found will be applied to all return periods. **Calibration / Validation** The peak flow at the bridge (FEP2) will be used to reconcile flows generated by the hydraulic model against those derived by the methods outlined within this document.
Software to be used (with version numbers)	FEH Web Service[1] / WINFAP 4[2] / ReFH2.3

[1] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

[2] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

3 LOCATIONS WHERE FLOOD ESTIMATES REQUIRED

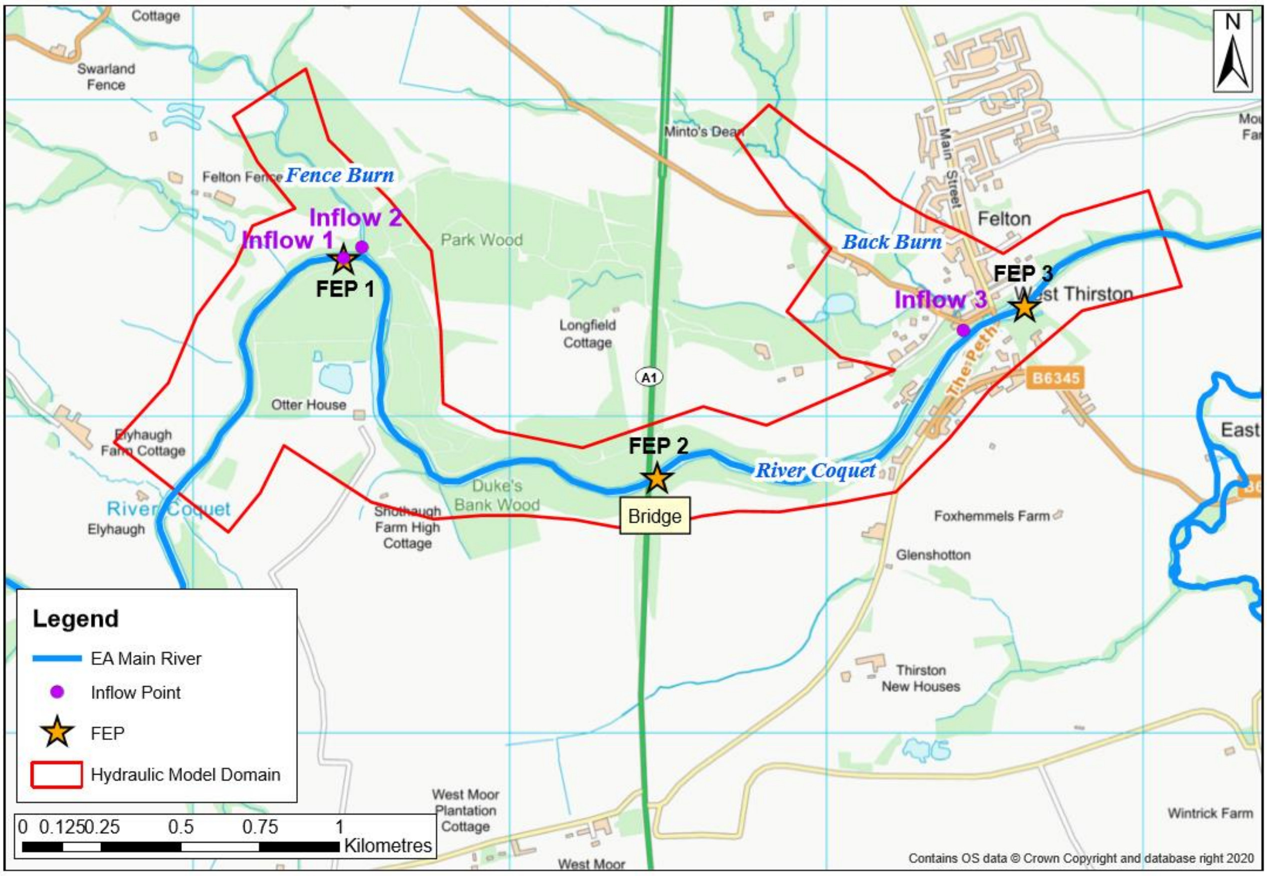


Figure 9: Flood Estimation Points (FEP) and Inflow Locations

The table below lists the locations of subject sites as shown in Figure 9 above. The site codes listed below are used in all subsequent tables to save space.

3.1 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 (km^2)
FEP 1	L	River Coquet	Lumped catchment and Catchment to Inflow 1 to upstream study area	416450	600500	457.1	-
FEP 2	L	River Coquet	Lumped catchment to the bridge site	417450	599800	486.0	-
FEP 3	L	River Coquet	Lumped catchment to downstream study area	418600	600350	490.6	-
Catchment 2	S (intervening)	Fence Burn Tributary	Fence Burn Tributary (Inflow 2) between Fence Burn Tributary and bridge (FEP2)	416500	600550	27.5	28.9

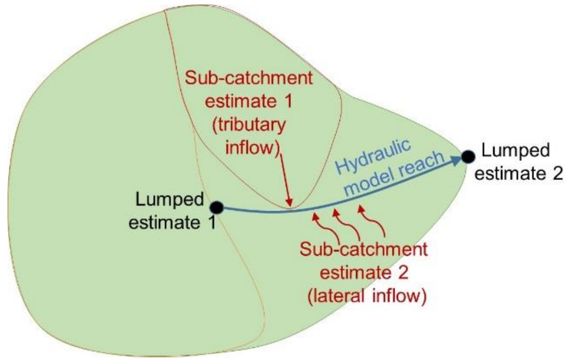
Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered (km^2)
Catchment 3	S (intervening)	Back Burn Tributary	Back Burn Tributary (Inflow 3) from bridge to downstream study area	418350	600300	3.9	4.6

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.

*FEP = Flow Estimation Point



3.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPLBAR Adjusted (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	URBEXT 2000 Adjusted	FPEXT
FEP 1 (Catchment 1)	0.99	0.450	0.40	36.36	-	125.70	879	0.002	0.002	0.036
FEP 2 (lumped)	0.99	0.450	0.39	36.29	-	122.20	873	0.002	0.002	0.035
FEP 3 (lumped)	0.99	0.450	0.39	37.49	-	121.70	872	0.002	0.002	0.035
Catchment 2	1.00	0.450	0.33	6.84	-	67.30	784	0.000	0.000	0.032
Catchment 3	1.00	0.450	0.44	2.22	2.31	49.20	716	0.020	0.021	0.035

3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	Catchment boundaries were adopted from the FEH derived catchments. A check was made to FEH catchment boundaries against OS mapping and LiDAR Digital Terrain Model (DTM) data to ensure catchments were suitable. Catchment boundaries were further updated for Catchment 2 (Fence Burn Tributary = 27.5 km^2) and Catchment 3 (Back Burn Tributary = 3.90 km^2) to account for the intervening catchment area along the River Coquet upstream and downstream of the bridge to ensure no area was excluded. Catchment 2 was therefore adjusted to 28.94 km^2 and Catchment 3 adjusted to 4.62 km^2. A check was made for Catchment 2 and Catchment 3 against intervening catchments (using the INTER spreadsheet from John Packman (CEH)) using Lumped FEP 2 – Lumped FEP 1 for Catchment 2 and Lumped FEP 3 – Lumped FEP 2 for Catchment 3. The results from this intervening tool did not produce realistic values for FARL and the intervening catchments therefore it was decided to use the catchments for Fence Burn Tributary and Back Burn and adjust catchment descriptors accordingly. This included adjustment of area as discussed above and DPLBAR for Catchment 3 which was adjusted based on power term 0.548 (FEH volume 5, equation 7.1). A check of the remaining catchment descriptors were found to be suitable for the study.
Record how other catchment descriptors were checked and describe any changes.	Geology mapping on the BGS online viewer (http://mapapps.bgs.ac.uk, accessed January 2021) at 1:50,000 scale and the Cranfield Soilscape web viewer (http://www.landis.org.uk/soilscapes accessed January 2021) were used to get an appreciation of the permeability of the overall catchment. Values of BFIHOST were found not to contradict the geology and soil maps. URBEXT2000 was adjusted to updated to the present year (2021) however the catchment is overall rural. This was checked against OS mapping and aerial photography.
Source of URBEXT	URBEXT2000 was taken from the FEH Web Service.
Method for updating of URBEXT	URBEXT2000 was updated to the present year (2021) using the CPRE formula from 2006 CEH report on URBEXT2000. This only changed URBEXT2000 value for Catchment 3 from an URBEXT2000 value of 0.020 to 0.021. This is expected as it is the most urbanised catchment including the western edge of Felton village. All other catchments remained the same due to the rural nature of the catchment.

4 STATISTICAL METHOD

4.1 Application of Statistical method

What is the purpose of applying this method?	The FEH statistical method will be applied to estimate peak flows for the catchments at FEP1, FEP2 and FEP3. A decision will then be made whether the statistical estimates should be used to reconcile ReFH2.3 hydrographs for each catchment against the FEPs. The catchment does not have any flow or level gauging stations in the vicinity of the subject site - the proposed bridge (FEP 2) - therefore the statistical method will be applied to estimate QMED from catchment descriptors initially for the lumped catchment to FEP 2. The lumped estimate to FEP 2 will be used to apply growth curves to FEP 1, FEP 3 and Catchments 2 and 3. The two gauging stations upstream and downstream of the modelled reach will be considered as donor stations and used to inform the catchment descriptor derived values at the subject site.

4.2 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		
FEP 1	108.20	DT	22001	-	-	0.62	0.79	1.00	144.41
			22009	-	-	0.38	0.55		
FEP 2	113.56	DT	22001	-	-	0.62	0.79	1.00	151.55
			22009	-	-	0.38	0.55		
FEP 3	113.92	DT	22001	-	-	0.62	0.79	1.00	152.07
			22009	-	-	0.38	0.55		
Catchment 2	9.56	DT	22001	-	-	0.62	0.79	1.02	12.74
			22009	-	-	0.38	0.55		
Catchment 3	1.23	DT	22001	-	-	0.62	0.79	1.00	1.67
			22009	-	-	0.38	0.55		

Are the values of QMED spatially consistent?	QMED values increase in a downstream direction and are therefore spatially consistent. However, it is noted there is little increase in flow (0.5m³/s) between the bridge (FEP 2) and the downstream extent (FEP 3). As the subject site is at FEP 2 the results are suitable for purpose of this study, however it has been strongly recommended later in the report that if future studies are required at this site this should be investigated further.

Method used for urban adjustment for subject and donor sites		The Kjeldsen (2010)[3] / WINFAP v4[4]
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)[3] 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)[4].		

4.3 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	Site 22001 at Morwick is a large rural catchment with similar characteristics as the subject site (84 km^2 larger than subject site) and lies on the same watercourse, approximately 3.3 km downstream. Site 22009 at Rothbury is also a large rural catchment with similar characteristics as the subject site (140 km^2 smaller than subject site) and lies on the same watercourse approximately 5.4 km upstream. Both sites have a long reliable flow record, gauged above QMED. A weighted average based on area was taken for both donor sites and used for the QMED adjustment factor at the subject site. When QMED is adjusted based on distance from the two donors (Kjeldsen, 2019), the resulting peak flow estimates are less than the peak flows at Rothbury (22009), despite being 5.4 km upstream of the subject site. This reduction in flow is not representative of the flow at the subject site between the two gaugings stations. A decision was therefore made to override the recommended use of the moderation term and assume an adjustment factor of 1.33 through an area weighted average between the two donor sites. Single site analysis at the two donor sites will also be carried out to compare against the subject site.

4.4 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)	Area Weighted Average between 2 Donor Sites
22001	AM	No	156	123.35	1.26	1.33

[3] Kjeldsen, T. R. (2010). Modelling the impact of urbanization on flood frequency relationships in the UK. Hydrol. Res. **41**. 391-405.
[4] Wallingford HydroSolutions (2016). WINFAP 4 Urban adjustment procedures.

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)	Area Weighted Average between 2 Donor Sites
22009	AM	No	133	92.12	1.44	

4.5 Derivation of pooling groups

One group was used to the subject site at the bridge (catchment to FEP 2) as all catchments being assessed have similar SAAR (767 – 879), FARL (0.99 – 1.00) and FPEXT (0.032 – 0.036) values.

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
FEP 2 (Lumped)	FEP 2	No	The unreviewed pooling group was possibly heterogeneous and a review of the pooling was optional. Sites removed: - 24001 (Wear @ Sunderland Bridge) – Same watercourse as station 24008 (Wear @ Witton Park) with an overlapping record period Sites added with similar catchment characteristics to subject site to increase data years: - 23008 (Rede @ Rede Bridge) The reviewed pooling group was acceptably homogeneous and a review of the pooling group was optional	L-CV – 0.22 L–Skew – 0.17
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

4.6 Derivation of flood growth curves at subject sites

Growth curves were derived at the subject site (lumped to FEP 2) through pooled analysis and also for the gauging station sites at Rothbury and Morwick (through single site analysis) to compare against the subject site (Figure 12). The flow estimates at the subject site derived using the adjusted QMED and pooling method showed a representative estimate as they lie between both upstream and downstream gauging station sites as would be expected (Figure 13).

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
FEP 2	P	Lumped to FEP 2	GL was chosen as this distribution presented the best z value (z = 1.06)	Rural	Location: 1.00 Scale: 0.224 Shape: -0.172	2.57
Rothbury	SS	Rothbury	GL was chosen as this distribution presented the best z value and was consistent with FEP 2 (z = 0.98)	Rural	Location: 1.00 Scale: 0.244 Shape: -0.169	2.70

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
Morwick	SS	Morwick	GL was chosen as this distribution presented the best z value and was consistent with FEP 2 (z = 1.57)	Rural	Location: 1.00 Scale: 0.256 Shape: -0.213	3.00

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).

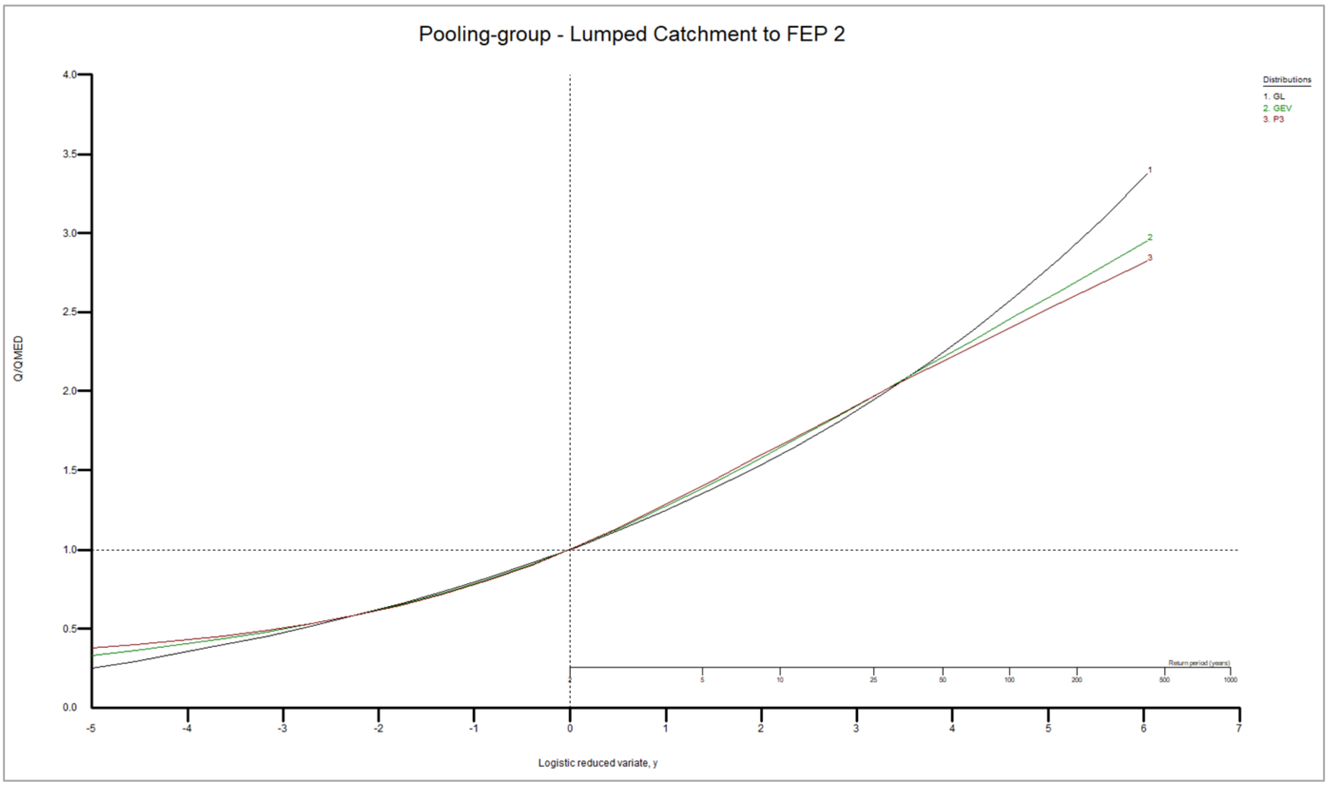


Figure 10: Pooling Group for Lumped Catchment to FEP 2

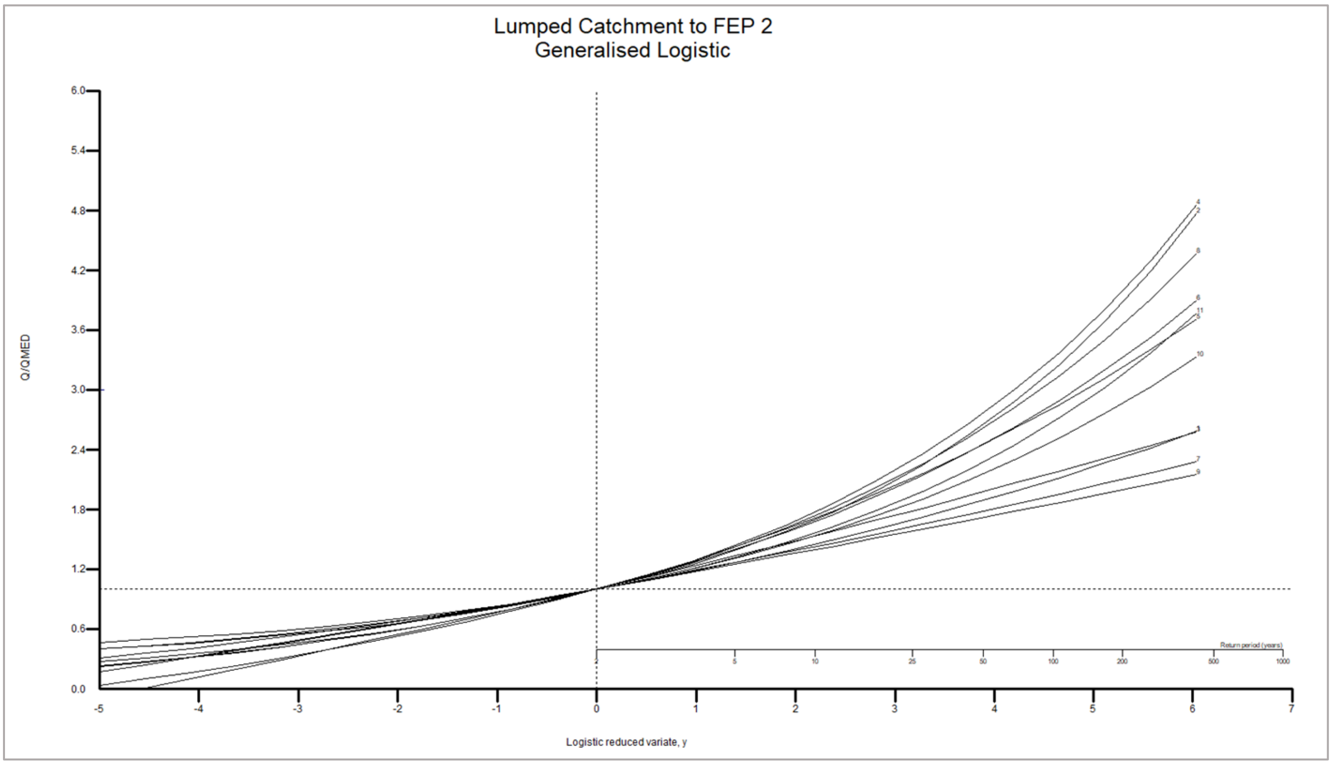


Figure 11: Generalised Logistic Graph for Lumped Catchment to FEP 2

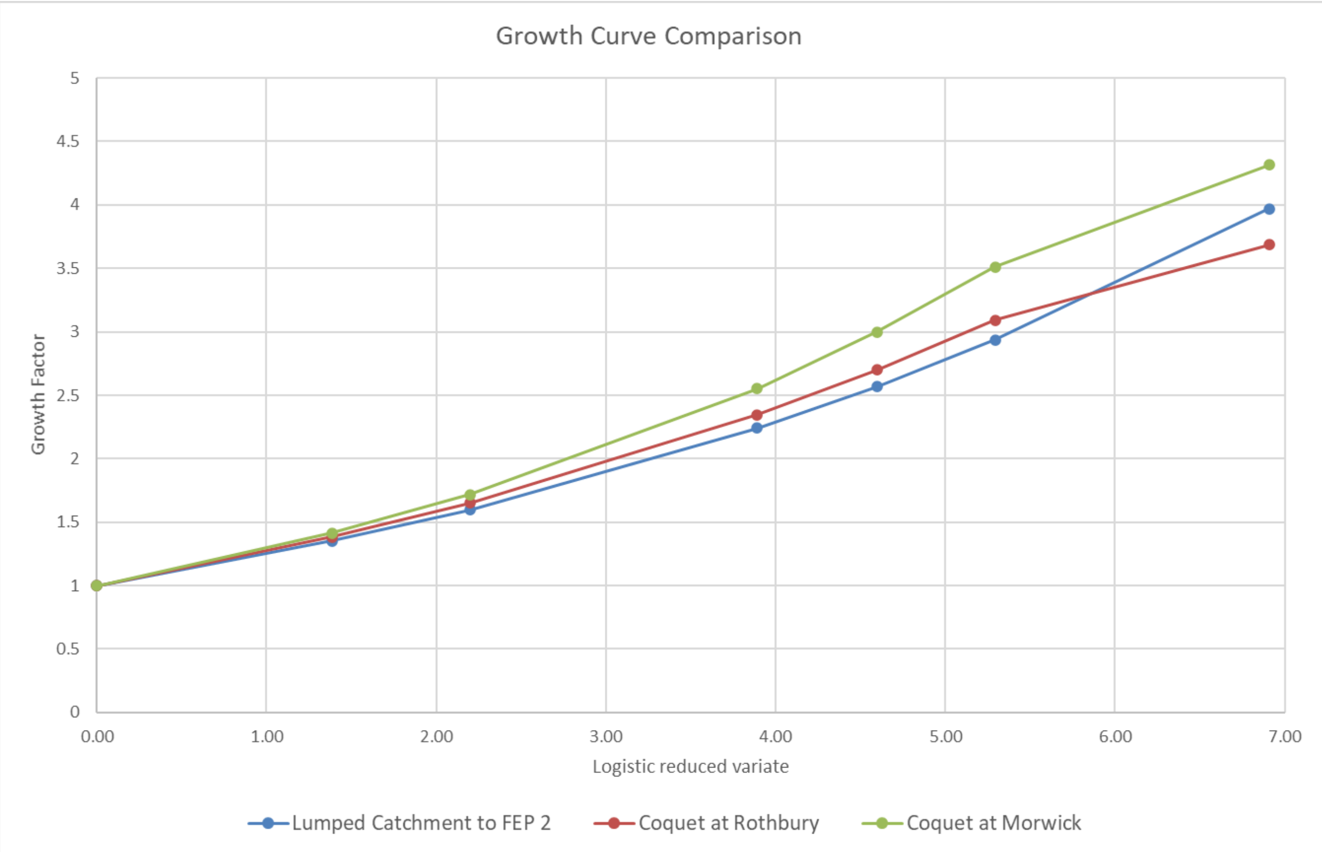


Figure 12: Growth Curve Comparison at Subject Site and Gauging Stations

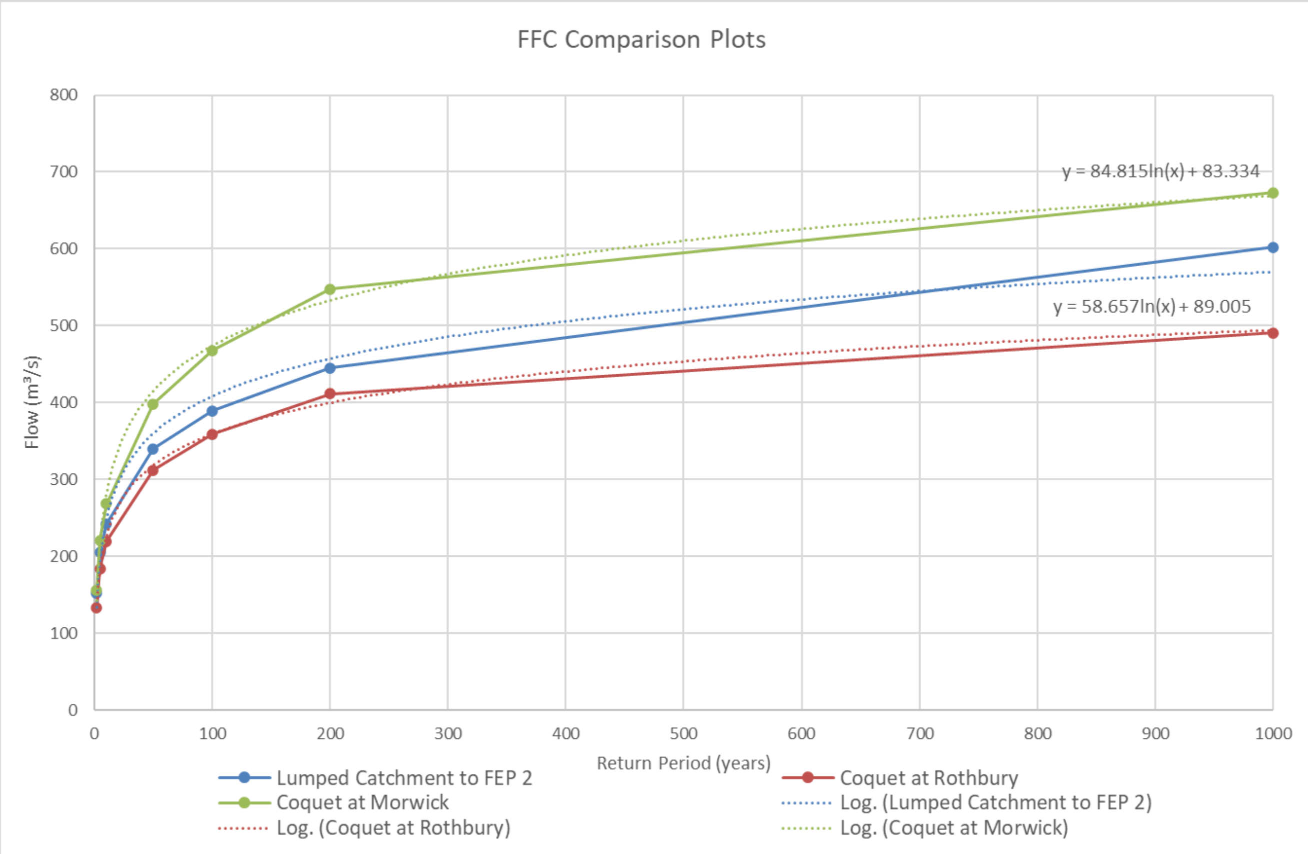


Figure 13: Flow Frequency Curves at Subject Site and Gauging Stations

4.7 Flood estimates from the statistical method

Growth factors derived using the lumped catchment to FEP 2 were applied to all catchments to ensure consistency. Results are shown in the table below.

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	50	100	200	1000	200 + 50% CC	200 + 65% CC
	Flood peak (m^3/s) for the following AEP (%) events								
	50	20	10	2	1	0.5	0.1	0.5+50% CC	0.5+65 %CC
FEP 1 (Catchment 1)	144.4	195.1	230.8	323.6	370.8	423.8	573.5	635.8	699.3
FEP 2	151.5	204.7	242.2	339.6	389.2	444.8	601.8	667.2	733.9
FEP 3	152.1	205.5	243.0	340.8	390.5	446.3	603.9	669.5	736.5
Catchment 2	12.7	17.2	20.4	28.6	32.7	37.4	50.6	56.1	61.7
Catchment 3	1.7	2.3	2.7	3.7	4.3	4.9	6.6	7.4	8.1

6 REVITALISED FLOOD HYDROGRAPH 2 (REFH2) METHOD

6.1 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 will be applied to derive hydrographs to input into the hydraulic model. If the decision is made to use statistical estimates as the best preferred method then these ReFH2.3 hydrographs will be reconciled within the model to match the statistical peak flow estimates.

6.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	C_{max} (mm)	PR_{imp} % runoff for impermeable surfaces	BL (hours)	BR
FEP 1 (Catchment 1)	CD	8.02	269.74	70	52.96	0.88
FEP 2	CD	8.09	269.74	70	52.93	0.89
FEP 3	CD	8.25	271.14	70	53.49	0.90
Catchment 2	CD	3.88	256.08	70	35.95	0.86
Catchment 3	CD	2.41	351.57	70	34.70	1.83
Brief description of any flood event analysis carried out			The two donor sites at Rothbury and Morwick, which lie upstream and downstream of the modelled reach, were considered however this study is predominantly interested in the peak flow and levels at the bridge site and flood volume isn't considered to be a primary concern and as the gauging sites aren't based directly in the modelled area, flood event analysis was not considered necessary for this study.			
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)
FEP 1 (Catchment 1)	Rural	Winter	15.5
FEP 2	Rural	Winter	15.5
FEP 3	Rural	Winter	15.5

6.4 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Default Storm duration (hours)	Design Event Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
Catchment 2	Winter	6.5	15.5	491 km²	Consistent with FEP 3

Site code	Season of design event	Default Storm duration (hours)	Design Event Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
				(catchment to FEP 3)	
Catchment 3	Winter	4.5	15.5	491 km² (catchment to FEP 3)	Consistent with FEP 3
Results of storm duration testing.			The individual storm durations above have been used to generate peak flow estimates for comparison with the FEH Statistical method estimates. For application to the hydraulic model a uniform storm duration and areal reduction factor (ARF) will be used. This will be a 15.5hr storm duration and an ARF set to the value of 0.896 for all inflow hydrographs.		

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	50	100	200	1000	200 + 50% CC	200 + 65% CC
	Flood peak (m^3/s) for the following AEP (%) events								
	50	20	10	2	1	0.5	0.1	0.5+50 %CC	0.5+65 %CC
FEP 1 (Catchment 1)	117.0	151.9	179.4	258.4	300.2	347.0	470.5	520.4	572.5
FEP 2	123.3	159.7	188.6	270.8	314.4	363.3	494.3	545.0	599.5
FEP 3	121.9	158.0	186.3	267.7	310.8	358.8	487.9	538.2	592.1
Catchment 2	11.0	14.9	17.7	24.9	28.7	33.2	47.6	49.8	54.8
Catchment 3	1.5	2.2	2.7	3.9	4.5	5.2	7.5	7.9	8.6

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 2 years / 50% AEP			Return period 100 years / 1% AEP		
	FEH	ReFH2	Ratio	FEH	ReFH2	Ratio
FEP 1 (Catchment 1)	144.4	117.0	0.81	370.8	300.2	0.81
FEP 2	151.5	123.3	0.81	389.2	314.4	0.81
FEP 3	152.1	121.9	0.80	390.5	310.8	0.80
Catchment 2	12.7	11.0	0.87	32.7	28.7	0.88
Catchment 3	1.7	1.5	0.92	4.3	4.5	1.05

7.2 Final choice of method

Choice of method and reasons	The FEH Statistical method is considered to be most appropriate for determining flow frequency at the FEPs. The design peak flow estimates and hydrographs were derived for the purposes of this modelling study which is to; – inform the fluvial geomorphological assessment required as part of the design of a new bridge and associated erosion protection measures for the operational and construction phases of the development; and, – confirm the flood levels during the operation and construction phase of the bridge. Therefore, peak flow estimates are the key outputs required for this study and not flood flow volumes. The statistical is in general the UK preferred method for deriving peak flows, furthermore two suitable donor stations with a good AMAX data record are available to help inform the QMED values. As shown in Figure 13 there is good single site analysis and very good FFC comparisons when we place the pooled FFC at the bridge site (FEP 2) with the single site for the lower return periods where we can place confidence. The flows presented from the ReFH2 method are based on catchment descriptors only. For this reason, there is lower confidence in the flow estimates derived from the ReFH2 model. The ReFH2 peak flow estimates are overall lower than those generated by the FEH Statistical method (with donor adjustment), with the exception of Catchment 3 which presents slightly higher peak flow values from ReFH2. A check of this has been undertaken and it was found that the catchment descriptor derived QMED estimates using the FEH Statistical method, without donor adjustment, are similar to the ReFH2 outputs for QMED. From the reasons discussed above, the ReFH2.3 hydrographs will be applied to the model and scaled to the statistical peak flow estimates at each FEP.
How will the flows be applied to a hydraulic model?	The ReFH2-generated hydrographs, using the 15.5 hour storm for the whole catchment will be provided to run through the hydraulic model. The hydrograph for the catchment to Inflow 1 (FEP 1 / Catchment 1) will be applied as a point inflow at the upstream extent of the hydraulic model and routed. Inflows

	representing the tributaries; Fence Burn (Inflow 2) and Back Burn (Inflow 3) will be applied as lateral flows by distributing them upstream and downstream of the bridge. The modelled flows will be checked to ensure that they approximate the lumped estimates from the statistical hydrological assessment. If necessary, the intervening area hydrographs will be scaled to better approximate the lumped estimate. Hydrographs have been derived for the 50%, 20%, 10%, 2%, 1%, 0.5% and 0.1% AEP events (plus climate change for the 0.5% AEP event) using a 15.5 hour storm duration. It should be noted that the hydrographs will be run through the model however if results do not compare well with the historical flood event at Felton after reconciliation with the statistical peak flow estimates then a reassessment of the flows may be required.

7.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are: - The stations used to define the growth curve from the FEH Statistical method are representative of the catchments to FEP 1, FEP 2 and FEP 3. These may not be as representative of the intervening catchments including the tributaries as they are smaller than catchments within the pooling group. - A donor adjustment factor through an area weighted average of the flow gauges upstream and downstream of the subject site was most appropriate for this study. This was compared to the method of adjusting QMED using 2 donor sites and the distance measure (Kjeldsen, 2019). When the adjustment factor for 2 donor sites is applied to the subject site, the resulting flow estimates are less than the flow estimates at Rothbury (22009), despite being 5.4 km upstream of Rothbury. This reduction in flow estimates are not representative of the subject site between the two gauging stations and is merely a product of the moderation of the adjustment factor by distance. Therefore, an adjustment factor of 1.33 was used, based on an area weighted average between the two donor sites. - Peak flow estimates are the primary concern for this study and not flood volumes at the bridge site. - ReFH2.3 model outputs provide a representative hydrograph shape to reconcile the flows within the hydraulic model.
Discuss any particular limitations	The main limitations are: - There are no flow records available within the modelled reach for calibration and verification of the hydrological models - The FEH Statistical method has been applied beyond the 1% AEP event and the reliability of estimates is reduced above this AEP. However, to overcome this the ReFH ratio method has been applied to adjust ReFH2 peak flow for AEP events above 1%.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	The uncertainty in the ReFH2 method cannot be directly quantified. An average measure of uncertainty is presented in Environment Agency Technical Guidance document on '*Using local data to reduce uncertainty in flood frequency estimation*' (EA, 2017). The report presents results for rural catchments (URBEXT2000 < 0.15).

	The 95% confidence limits for a rural catchment with one donor adjustment of QMED were used. Although this study uses two donor sites for the adjustment of QMED the difference between one donor site and six donor sites within the above guidance is minimal therefore uncertainty values for one donor site is appropriate. The 95% confidence limits for a 1% AEP flood estimate to the subject site (FEP 2) is 182.9 - 825.1m³/s. The comparison which was carried out through single site analysis of the two gauging stations presents a level of uncertainty above the 50 year return period event.
Comment on the suitability of the results for future studies	The design peak flow estimates and hydrographs were derived for the purposes of this modelling study which is to; – inform the fluvial geomorphological assessment required as part of the design of a new bridge and associated erosion protection measures for the operational and construction phases of the development; and, – confirm the flood levels during the operation and construction phase of the bridge If peak flow estimates and hydrographs are required for a different purpose it is recommended that, at a minimum, a review of the results is carried out. It has been noted that there is a very small increase in the QMED estimates between FEP 2 and FEP 3 therefore it is strongly recommended that further investigation is carried out into the reason for this. If the purpose of future studies is to determine flood volumes then it is recommended that further flood event analysis through use of the two gaugings stations is carried out to provide further confidence in the hydrographs.
Give any other comments on the study.	There are two continuous flow gauges; one upstream and one downstream of the study area. Future studies could assess this gauge data to produce hydrograph profiles for both gauged sites. However, the gauge sites are not within the vicinity of the subject site and as the aim of this study is to determine peak levels at the bridge site it was not deemed necessary to use the gauged data.

7.4 Checks

Are the results consistent, for example at confluences?	The FEH Statistical method is the preferred method for flow estimates for this study. Flows produced from this method increase in a downstream direction. The runoff from the lumped catchments to the FEPs is consistent per unit area and is also consistent between the two tributary catchments to Inflow 2 and Inflow 3. Results from the ReFH2 method show a small decrease in flow between FEP 2 and FEP 3. This is likely due to an increased urban area (Felton Village) within the catchment to FEP 3 creating differing peak flow timings. However, the FEH Statistical results show an increase downstream. As flows will be reconciled to the statistical estimates within the hydraulic model results are considered consistent.
What do the results imply regarding the return periods / frequency of floods during the period of record?	There is no flow gauge within the vicinity of the subject site / modelled reach against which to compare the design flow estimates. However, results were compared against the two gauging stations upstream and downstream of the modelled reach and the results at the subject site lie in the middle of both sites which would be expected.
What is the range of 100-year / 1%	The growth factor for the FEH Statistical method 1% AEP event 2.6

AEP growth factors? Is this realistic?	for the catchment to FEP2. The normal range of values is 2.1 - 4.0.
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 for all lumped catchments (FEP 1 – FEP 3) is 1.6 and the ratio for tributaries/ intervening catchments is 1.7.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	There have been no other studies on this reach of the River Coquet.
Are the results compatible with the longer-term flood history?	Peak flow data was available at the upstream (Rothbury) and downstream (Morwick) gauging stations to compare against the flows at the subject site. Flood results for this study show that the flows at the bridge are between the peak flow estimates upstream and downstream of the subject site. There is only one record of flood history that has been identified through this assessment which is a flood event at Felton Village on 6 September 2008. Through interpolation of results at Morwick and Rothbury this is thought to be between a 184 – 266 year event.
Describe any other checks on the results	Modelled flood levels and extents will be sensibility-checked to ensure that flow inputs result in realistic outputs.

7.5 Final results

The final results were taken from the FEH Statistical estimates as shown in the table below.

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	50	100	200	1000	200 + 50% CC	200 + 65% CC
	Flood peak (m^3/s) for the following AEP (%) events								
	50	20	10	2	1	0.5	0.1	0.5+50 %CC	0.5+65 %CC
FEP 1 (Catchment 1)	144.4	195.1	230.8	323.6	370.8	428.6	581.2	642.8	707.1
FEP 2	151.5	204.7	242.2	339.6	389.2	449.8	611.9	674.7	742.2
FEP 3	152.1	205.5	243.0	340.8	390.5	450.9	613.1	676.3	743.9
Catchment 2	12.7	17.2	20.4	28.6	32.7	37.9	54.4	56.9	62.6
Catchment 3	1.7	2.3	2.7	3.7	4.3	5.0	7.2	7.5	8.2

7.6 Uncertainty bounds

This table reports the flows derived from the uncertainty analysis detailed in Section 7.3. The 'true' value is more likely to be near the estimate reported in Section 7.5 than the bounds. However, it is possible that the 'true' value could still lie outside these bounds.

Site code	Flood peak (m^3/s) for the following return periods (in years)							
	2		10		100		1,000	
	Flood peak (m^3/s) for the following AEP (%) events							
	50		10		1		0.1	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
FEP 1 (Catchment 1)	72.2	291.7	113.1	473.1	174.3	786.1	261.5	1296.1

Site code	Flood peak (m^3/s) for the following return periods (in years)							
	2		10		100		1,000	
	Flood peak (m^3/s) for the following AEP (%) events							
	50		10		1		0.1	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
FEP 2	75.8	306.1	118.7	496.5	182.9	825.1	275.4	1364.5
FEP 3	76.0	307.2	119.1	498.2	183.6	827.9	275.9	1367.2

If flood hydrographs are needed for the next stage of the study, where are they provided?	Flood hydrographs will be provided to the hydraulic modellers. These are saved in the project folder on the server within the spreadsheet: OneDrive - Jacobs\Documents\00 Projects\B3660114 A1 in Northumberland\River Coquet Hydrology\01 Hydrological Analysis\ReFH2\Inflows for Modellers - Winter 15hr.csv

8 ANNEX

8.1 Flood History

There is one instance of flooding at Felton Village noted within the Northumbria Local Resilience Forum September 2008 (Northumberland County Council, 2008) on 6 September 2008. Approximately 7 residential properties in Felton were reported as flooded on this date (see photograph below).

Figure 14: Flooding at Felton on 6 September 2008 (Northumbria County Council, 2008)

There are no flow or level estimates detailed within this report however flood history was mapped against the two gaugings stations; Rothbury and Morwick, upstream and downstream of the subject site respectively. The highest recorded flow at Rothbury was on the 7 September 2008 showing a flow of 525.72m³/s and at Morwick the highest recorded flow was on 6 September 2008 with a flow of 416.56m³/s,

Through interpolation of the FDC plots for each gauging station the peak flow event at Rothbury on 7 September 2008 is estimated to be the 266 year return period event and at Morwick on 6 September 2008 is estimated to be the 184 year return period event. A check will be made within the hydraulic model to determine the flood extents for the higher return periods as, based on the historical flow estimates at the two gauging stations and the flood history above, this should show flooding at Felton Village.

8.2 Pooling Analysis

Unreviewed Pooling Group at Subject Site (Lumped Catchment to FEP 2)

Table 3: Unreviewed Pooling Group for Lumped Catchment to FEP 2

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA
10001 (Ythan @ Ardlethen)	0.156	46	50.18	0.179	0.116	1.439	457.12
11003 (Don @ Bridge of Alford)	0.208	46	95.811	0.234	0.294	0.895	509.94
10003 (Ythan @ Ellon)	0.21	23	57.695	0.232	0.035	1.532	532.29
22001 (Coquet @ Morwick)	0.254	56	155.581	0.261	0.263	0.815	578.25
43008 (Wylye @ South Newton)	0.257	47	12.8	0.266	0.151	0.987	448.17
9001 (Deveron @ Avochie)	0.27	60	128.516	0.242	0.201	0.548	444.91
24008 (Wear @ Witton Park)	0.388	46	201.916	0.183	0.045	0.82	454.63
24001 (Wear @ Sunderland Bridge)	0.457	62	185.838	0.192	0.236	1.556	661.17
22009 (Coquet @ Rothbury)	0.48	44	133.493	0.254	0.23	0.42	345.98
27090 (Swale @ Catterick Bridge)	0.485	27	306.276	0.175	0.026	1.321	497.56
8004 (Avon @ Delnashaugh)	0.492	62	210.551	0.194	0.208	0.667	540.75
Total		**519**					
Weighted means				**0.22**	**0.171**		

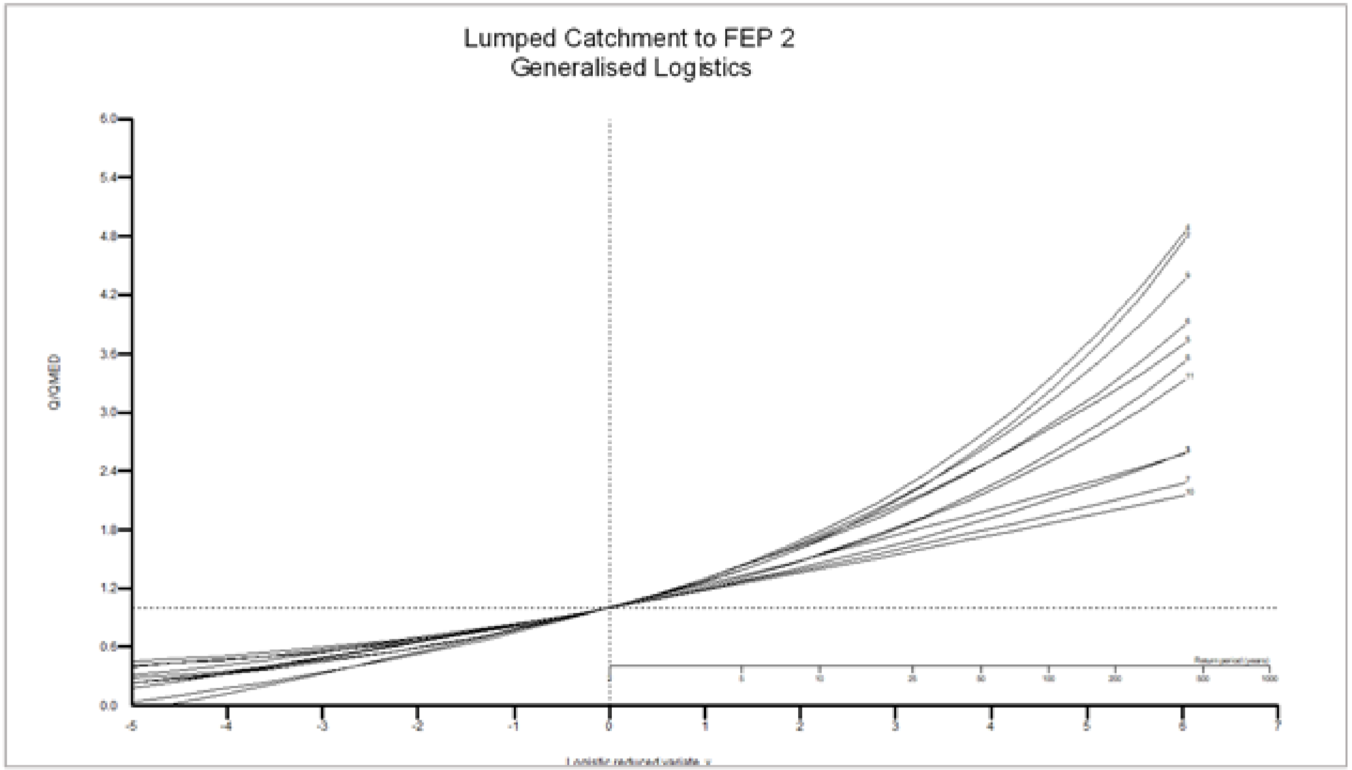


Figure 15: Generalised Logistic Graph for Lumped Catchment to FEP 2 (Unreviewed Pooling Group)

Pooling Group Review:

- No sites with less than 8 years data
- No sites influenced by reservoirs or lakes (all FARL > 0.9)
- All sites have similar flood seasonality around 1 Jan
- Stations were found to be on the same watercourse with overlapping time periods. These were removed to avoid any bias towards the pooling group for that record i.e. double-counting.
    - Two stations on the River Wear (24008 Wear @ Witton Park and 24001(Wear @ Sunderland Bridge). Site 24001 (Wear @ Sunderland Bridge removed as lower-ranking site.
    - Two stations on the River Ythan (10001 Ythan @ Ardlethen and 10003 Ythan @ Ellon) with a total record period of 69 years. Only just over one year over overlapping data out of the 69 years therefore remained in the pooling group.

Revised Pooling Group

Table 4: Revised Pooling Group for Lumped Catchment to FEP 2

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA
10001 (Ythan @ Ardlethen)	0.156	46	50.18	0.179	0.116	1.137	457.12
11003 (Don @ Bridge of Alford)	0.208	46	95.811	0.234	0.294	0.433	509.94
10003 (Ythan @ Ellon)	0.21	23	57.695	0.232	0.035	1.689	532.29
22001 (Coquet @ Morwick)	0.254	56	155.581	0.261	0.263	0.533	578.25
43008 (Wylye @ South Newton)	0.257	47	12.8	0.266	0.151	0.987	448.17
9001 (Deveron @ Avochie)	0.27	60	128.516	0.242	0.201	0.796	444.91
24008 (Wear @ Witton Park)	0.388	46	201.916	0.183	0.045	0.715	454.63
22009 (Coquet @ Rothbury)	0.48	44	133.493	0.254	0.23	0.396	345.98
27090 (Swale @ Catterick Bridge)	0.485	27	306.276	0.175	0.026	1.358	497.56
8004 (Avon @ Delnashaugh)	0.492	62	210.551	0.194	0.208	1.045	540.75
23008 (Rede @ Rede Bridge)	0.511	50	126.774	0.195	0.259	1.127	343.80
Total		507					
Weighted means				0.221	0.172		

Final Pooling Group

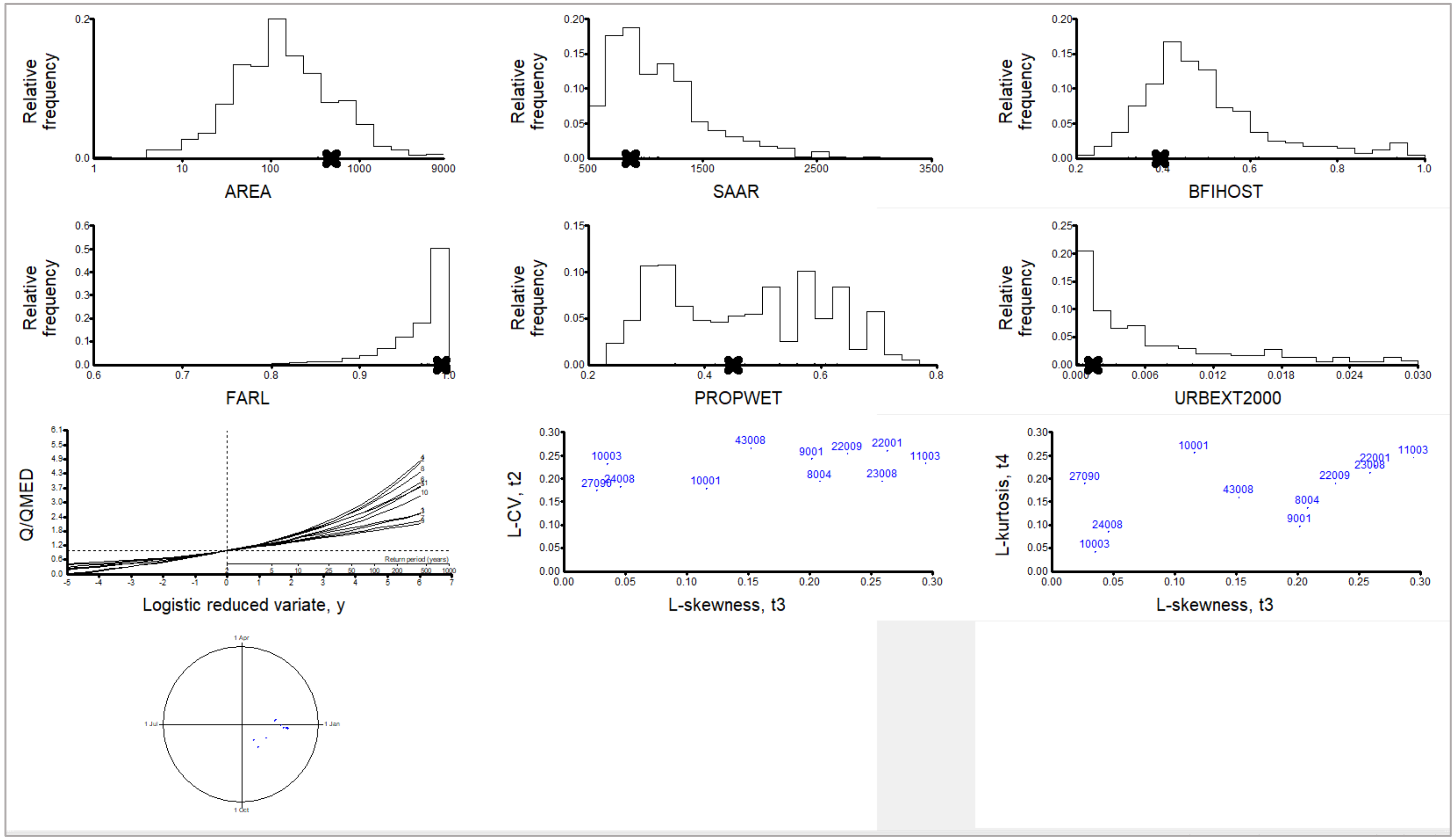


Figure 16: Catchment Descriptors Graphs for Pooling Group Lumped Catchment to FEP 2

Growth Factors

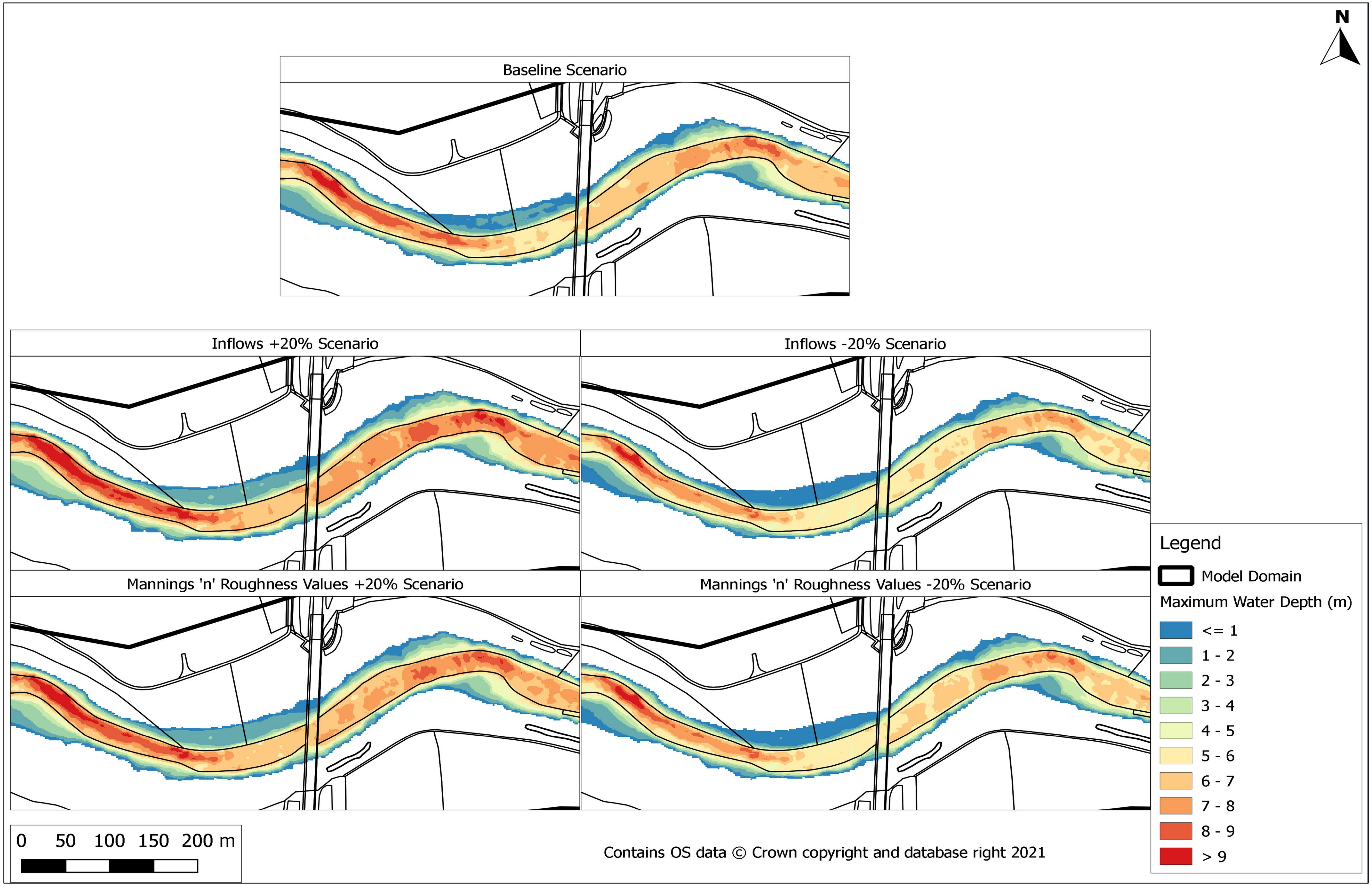
Table 5: Growth Factors for Pooling Group Lumped Catchment to FEP 2

RP	GL	GEV
2	1	1
5	1.351	1.387
10	1.598	1.644
50	2.241	2.213
100	2.568	2.454
200	2.935	2.695
1000	3.971	3.256

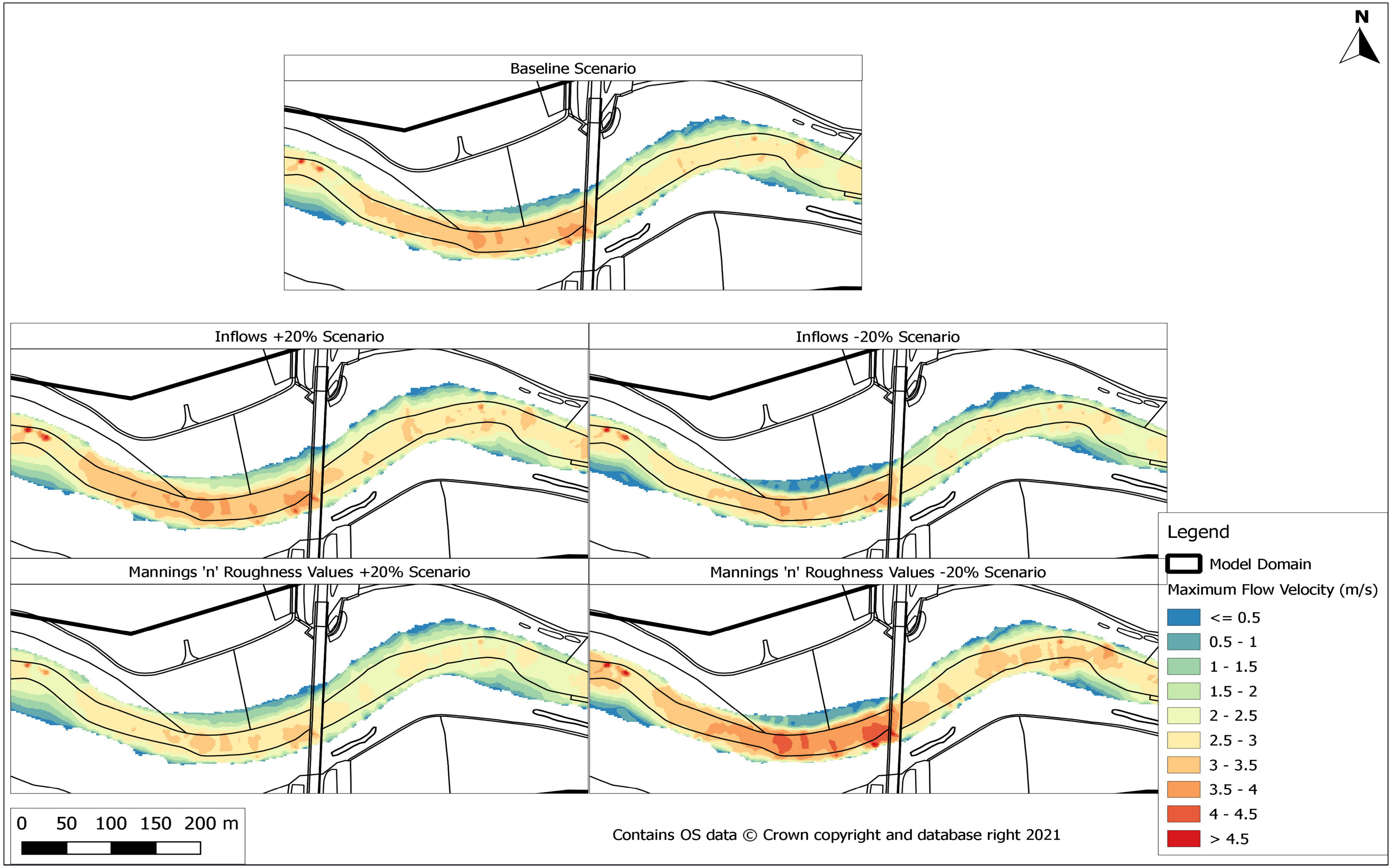
Appendix B

BASELINE SENSITIVITY TESTS RESULTS

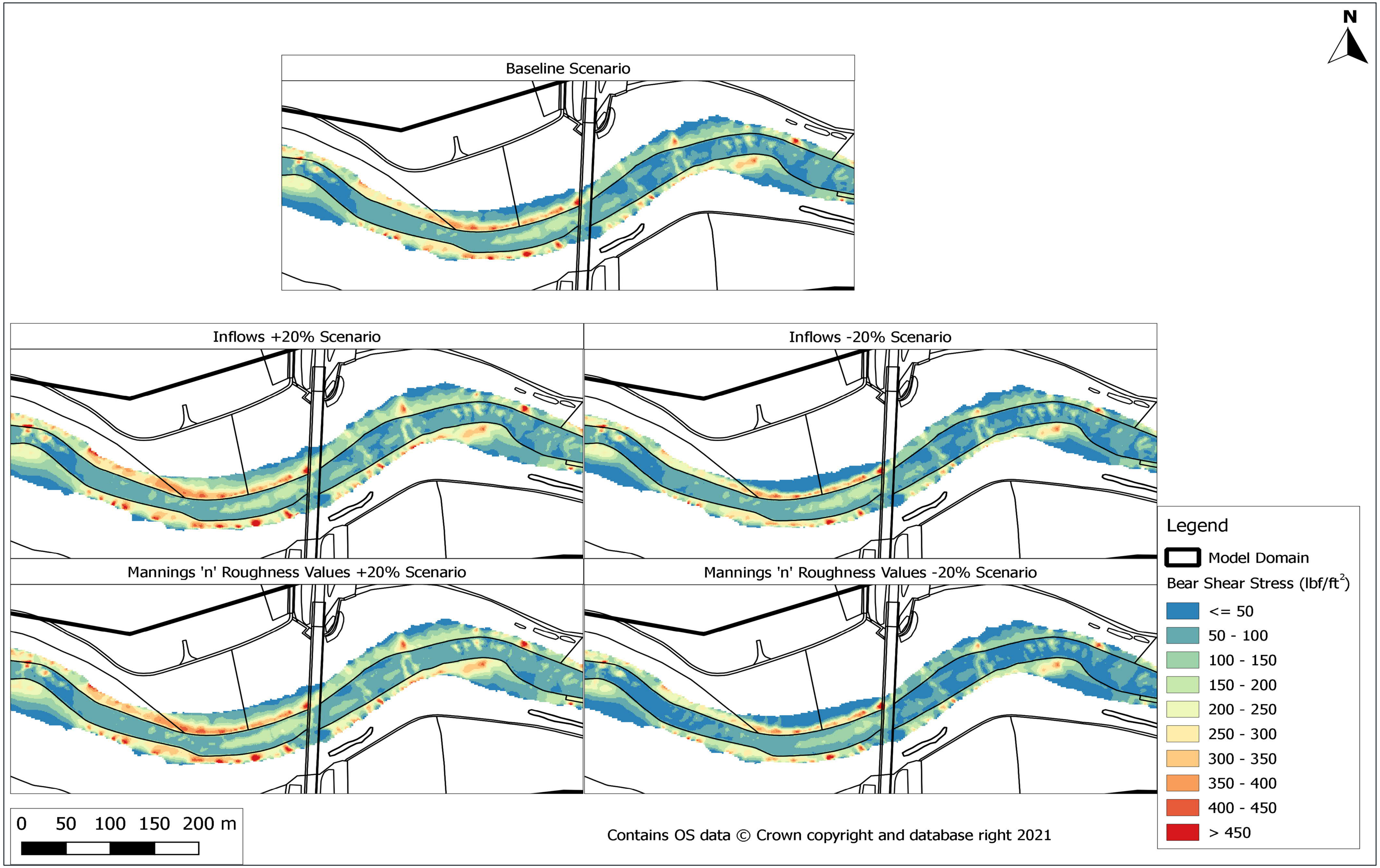
Maximum Flood Depth



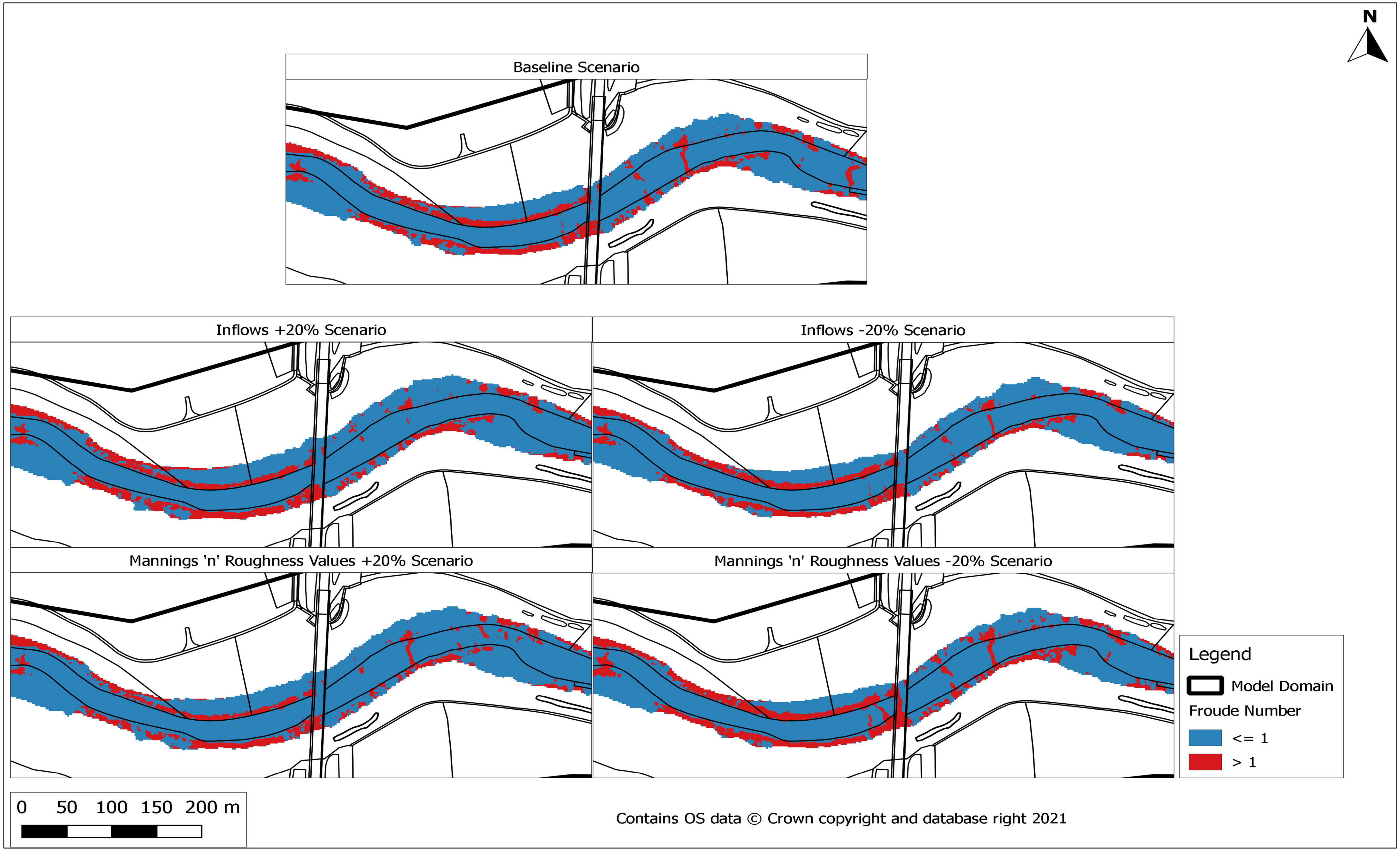
Maximum Flow Velocity



Maximum Bed Shear Stress



Maximum Froude Number



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