

Sixmilebridge Flood Map Update

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Contract

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This report describes work commissioned by JBA Ireland, on behalf of the Office of Public Works (OPW), by an instruction dated 08/03/2023. The Client's representative for the contract was Ross Bryant of JBA Ireland. Patrick Parkinson of JBA Consulting carried out this work.

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The methodology adopted and the sources of information used by JBA in providing its services are outlined in this Report. The work described in this Report was undertaken between March and December 2023 and is based on the conditions encountered and the information available during the said period. The scope of this Report and the services are accordingly factually limited by these circumstances.

Where field investigations are carried out, these have been restricted to a level of detail required to meet the stated objectives of the services. The results of any measurements taken may vary spatially or with time and further confirmatory measurements should be made after any significant delay in issuing this Report.

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Abbreviations

1D	One Dimensional (modelling)
2D	Two Dimensional (modelling)
AEP	Annual Exceedance Probability
CFRAM	Catchment Flood Risk Assessment and Management
DTM	Digital Terrain Model
FRA	Flood Risk Assessment
FSU	Flood Studies Update
GEV	General Extreme Value Distribution
HEC-RAS	Hydrologic Engineering Center – River Analysis System (developed by the US Army)
HEFS	High End Future Scenario
HEP	Hydrological Estimation Point
ID	Identifier
LiDAR	Light Detection And Ranging
mAOD	metres Above Ordnance Datum
MRFS	Medium Range Future Scenario
OPW	Office of Public Works
QMED	Median Annual Flood (with return period 2 years)
SFRA	Strategic Flood Risk Assessment
SI	International System of Units

1 Introduction

1.1 Overview

JBA was commissioned by the Office of Public Works (OPW) to carry out an update of existing CFRAM (Catchment Flood Risk Assessment and Management) hydraulic model data for Sixmilebridge, County Clare. The requirement for this flood modelling and mapping update is driven by the OPW's Map Review Programme to ensure that community level flood map data in the National Flood Information Portal continues to be as up to date and accurate as possible.

JBA carried out a hydrological analysis of the Shannon catchment and constructed a detailed hydraulic model for Sixmilebridge according to the scope of works defined by the OPW. This report provides a detailed summary of the hydrological analysis and hydraulic modelling carried out in this study.

1.2 Study area

Sixmilebridge (Figure 1-1) is a village in County Clare, Ireland, approximately 9km upstream of the Shannon estuary and approximately 13km northwest of Limerick. Sixmilebridge has a population of approximately 2,832 according to 2022 census data, and is predominantly made up of residential and commercial land uses with agricultural land making up much of the surrounding areas.

The main watercourse running through Sixmilebridge is the Owenogarney River, which flows south through Sixmilebridge before discharging into the Shannon Estuary between Limerick and Shannon. The Owenogarney River originates from Castle Lake Approximately 3km upstream of Sixmilebridge.

An unnamed tributary converges with the Owenogarney River in Sixmilebridge, originating from the east to its confluence opposite the Gort Na Núll estate in the southern area of Sixmilebridge.

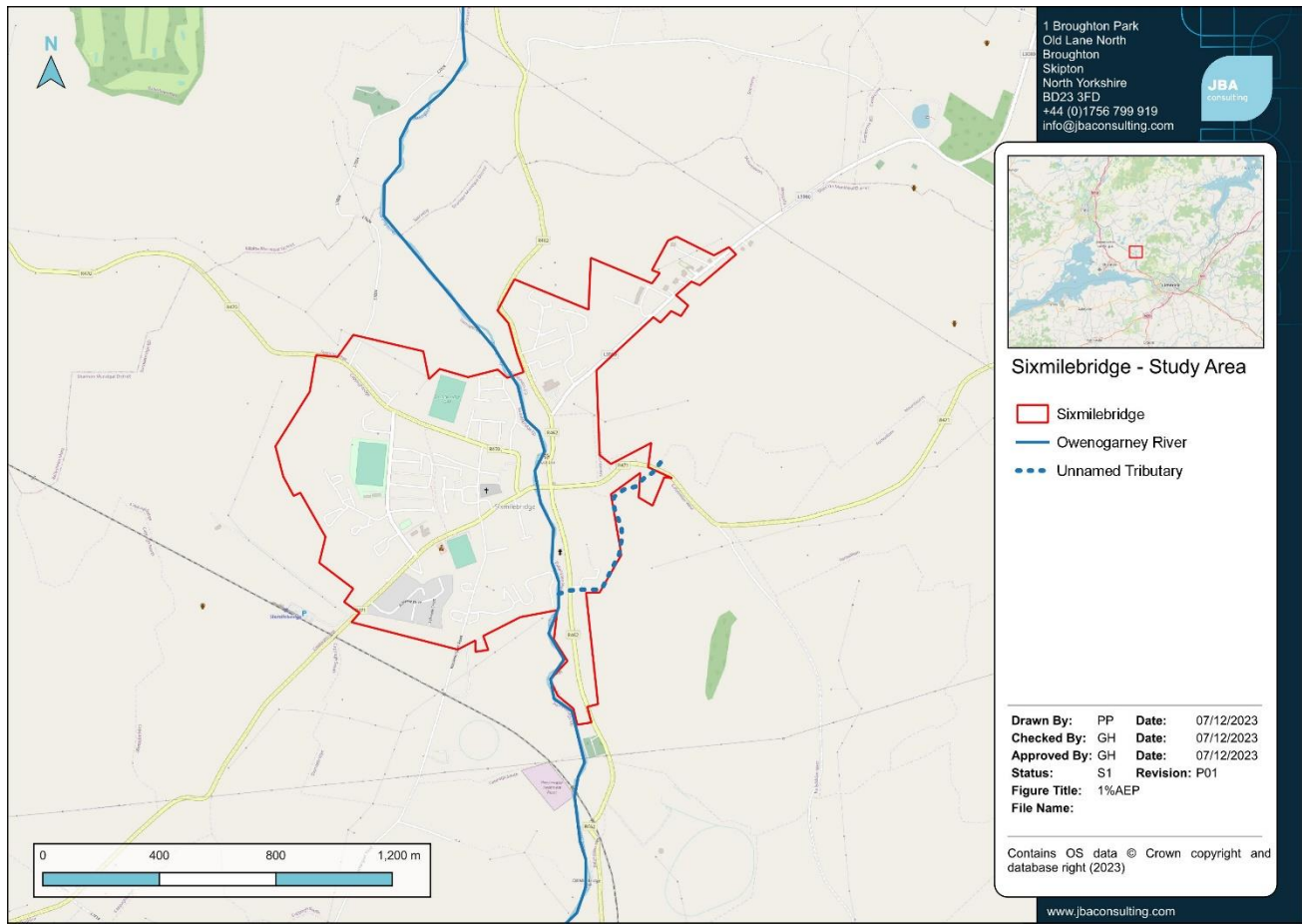


Figure 1-1: Sixmilebridge Location Plan

1.3 Context

1.3.1 OPW Flood Map Update

Since the completion of the National CFRAM programme (2010 - 2018) the OPW have continued to carry out a Map Review Programme to ensure that flood map data at a community level is up to date and accurate. There are various factors that may influence the requirement for flood map updates, including:

- A change in the physical conditions since the Flood Map was produced. Examples of potential changes include new infrastructure, a new development, a culvert or bridge upgrade, land raising, channel realignment, etc.
- An error has been identified in the survey. Errors in the survey may include a channel missing or a channel misaligned, or an error in the surveyed levels used.
- Completion of an OPW-funded Flood Relief Scheme or Minor Works Scheme.
- Evidence presented by a recent flood event that calls into question the published Flood Map.
- The completion of a Strategic Flood Risk Assessment (SFRA), supported by hydraulic modelling, which produces flood maps that are materially different to the OPW published Flood Maps.

1.3.2 Sixmilebridge Flood Maps

Two main drivers have necessitated a flood map update for Sixmilebridge:

- Clare County Council have indicated that Sixmilebridge is an area where flood risk is underpredicted in CFRAM data which has been reported in a recent SFRA (Strategic Flood Risk Assessment, Draft Clare County Development plan 2023-2029).
- A flood defence embankment has been constructed in Sixmilebridge since the completion of the CFRAM study.

A resulting preliminary study was carried out by the OPW. One of the findings from this study was that a Flood Risk Assessment (FRA), which was carried out by JBA for a proposed development in Sixmilebridge for Clare County Council, showed a large disparity in estimated flows to the equivalent flows from the CFRAM data.

The FRA estimated the maximum 1%AEP flow as 43m³/s, which is significantly higher than the equivalent CFRAM estimation (27m³/s). This resulted in significant differences in flooding extents in Sixmilebridge, which was also compounded by the fact the eastward tributary in Sixmilebridge was included in the JBA modelling and not in the CFRAM modelling.

1.4 Hydraulic modelling report scope and methodology

This report will detail the works undertaken by JBA in this hydraulic model update study. project. This report will also detail any additional data collected as part of this study, such as survey data and hydrological data for Sixmilebridge.

This report will set out the following details as outlined in the OPW scope of services for this project:

- Details of the surveys undertaken.
- A detailed description of the hydrological analysis.
- A detailed description of the hydraulic modelling methodology undertaken.
- A detailed model sensitivity analysis.
- Details of model calibration methods.
- Details of model assumptions and limitations.

1.5 Report structure

Section 1 - Introduction

Provides an overview of the project background and study area.

Section 2 - Hydrological Analysis

A separate report has been produced for this study, which provides details of the hydrological analysis methodology undertaken. This section provides a short summary of the methodology and results of the hydrological analysis.

Section 3 - Fluvial Hydraulic Model - Summary of Methodology

Provides a brief summary of the key elements of the hydraulic model build methodology.

Section 4 - Data Availability

A detailed summary of the data used in this study.

Section 5 - Fluvial Hydraulic Model Construction and Schematisation

A detailed overview of the model schematisation and hydraulic parameters of the model.

Section 6 - Model Calibration and Verification to Historical Events

A detailed analysis of model outputs against historic flood event data.

Section 7 - Model Sensitivity

A detailed analysis of a range of sensitivity tests used to demonstrate the model's sensitivity to changing variables.

Section 8 - Key Model Assumptions and Model Limitations

A detailed overview of the assumptions made during the hydraulic modelling process, as well the limitations of the model.

Section 9 - Model File Naming

An overview of the file naming structure for model file deliverables.

2 Hydrological Analysis

A detailed report outlining the hydrology analysis methodology and results is included in Appendix A.

2.1 Catchment description

The main hydrological feature in Sixmilebridge is the Owenogarney River. The river rises in Moylussa hills with its highest point at 531.6mAOD. The catchment stretches to Rogmount Hill and is wider in the upper reach with more tributaries feeding the Owenogarney River upstream. The downstream part of the study catchment is narrower. The total area of the Owenogarney River study catchment downstream of Sixmilebridge is 162km². The river then continues for another 9km before discharging to the Shannon Estuary.

Castle Lake is upstream of the study area on the Owenogarney River and may attenuate flood discharge. The unnamed tributary is located east of the Owenogarney River on the left bank and has a catchment area of 1.67km² which discharges just downstream of Sixmilebridge. The tributary catchment is narrow and steep with the highest point 85mAOD.

The study area is predominantly rural, with Sixmilebridge comprising the only urban area. The upstream part and the south edge of the catchment consists of various forest types, peat bogs, moors, and woodland. A number of small waterbodies are within the catchment.

Figure 2-1, taken from the hydrological analysis report, shows the study area catchment with associated watercourses.

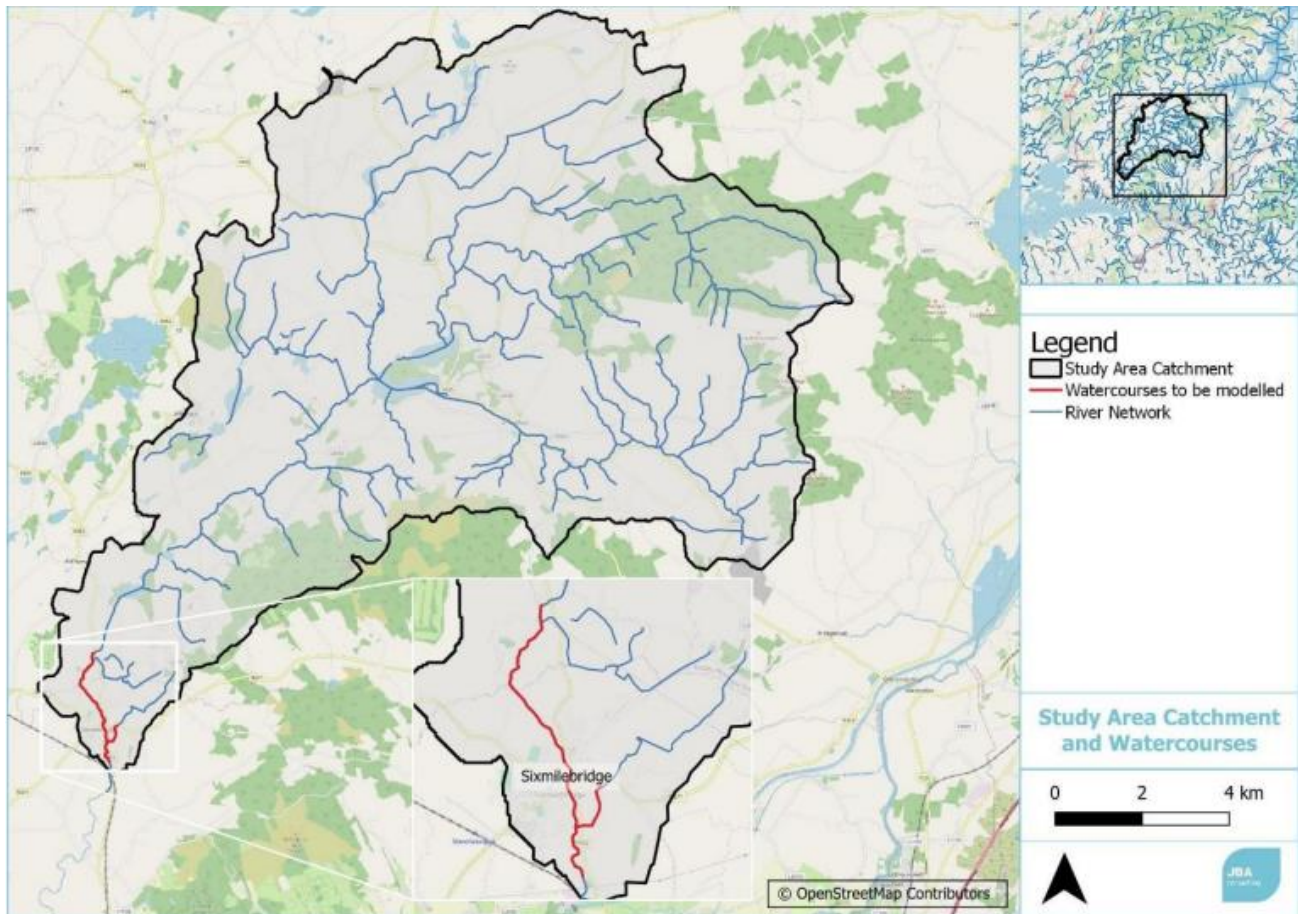


Figure 2-1: Study Area Catchment and Watercourses to be Modelled

2.2 Design flows

The final peak flows for the Owenogarney river were calculated using an unadjusted QMED catchment descriptor FSU method with a GEV growth curve from a pooled analysis for the Owenogarney River. Flows were calculated for 8 design events, the calculated peak flows for which are displayed in Table 2-1.

Table 2-1: Calculated fluvial Peak Flow Values

Annual Exceedance Probability (AEP)	Owenogarney Peak Flow (m ³ s)	Tributary Peak Flow (m ³ s)
50%	21.81	1.04
20%	27.27	1.3
10%	30.98	1.48
5%	34.47	1.64
2%	38.61	1.84

Annual Exceedance Probability (AEP)	Owenogarney Peak Flow (m ³ s)	Tributary Peak Flow (m ³ s)
1%	41.88	2.0
0.5%	45.15	2.15
0.1%	52.14	2.49

2.3 Tidal level estimates

A fluvial-tidal joint probability has been considered in this study due to the tidal influence from the Shannon estuary, therefore calculated tidal levels have been applied at the downstream boundary of the model. Estimated peak tidal levels were derived from the ICWWS for use in this study which is outlined in the hydrological analysis report. It should be noted, however, that for this study updated peak tidal levels have been derived since the hydrological analysis was undertaken. Details of the justification for this decision are outlined in Section 8.1 of this report. The estimated peak tidal levels used in this study are listed in Table 2-2.

Table 2-2: Estimated Peak Tidal Levels

Annual Exceedance Probability (AEP)	ICWWS S25 updated peak levels (mAOD)
50%	3.78
20%	3.99
10%	4.15
5%	4.31
2%	4.52
1%	4.67
0.5%	4.83
0.1%	5.19

2.4 Climate change

The climate change factors listed in Table 2-3 for the Mid-Range and High-End Future Scenarios (MRFS and HEFS, respectively) will be applied to all design flow hydrographs for a separate suite of model runs. This is to account for the impacts of climate change as specified in the OPW scope of works.

Table 2-3: Climate Change Uplift Factors

	MRFS	HEFS
Fluvial flows	+20%	+30%
Sea Level Rise	+500mm	+1000mm

3 Fluvial Hydraulic Model - Summary of Methodology

3.1 Overview

A 1D-2D HEC-RAS hydraulic model was produced for the OPW to provide a more detailed estimate of current and potential future fluvial levels, depths, velocities and extents. Where possible the model output data has been verified against historic flood data and river gauge data.

3.2 Modelled design events

The model has been run for the following range of Annual Exceedance Probability (AEP) events:

- 50%AEP (1 in 2 year event)
- 20%AEP (1 in 5 year event)
- 10%AEP (1 in 10 year event)
- 5%AEP (1 in 20 year event)
- 2%AEP (1 in 50 year event)
- 1%AEP (1 in 100 year event)
- 0.5%AEP (1 in 200 year event)
- 0.1% AEP (1 in 1000 year event)

3.3 Climate change allowances

Allowances were applied to all design events to represent the impacts of future climate change on fluvial flows. Climate change allowances of 20% and 30% were applied to all modelled design events, which represents the Mid-Range Future Scenario (MRFS) and the High-End Future Scenario (HEFS) respectively.

3.4 Model sensitivity analysis

Sensitivity testing was carried out for a range of hydraulic parameters to assess their impact on model results. The sensitivity tests listed below have been carried out for this modelling study:

- Weir Coefficient sensitivity.
- 1D and 2D hydraulic roughness increased by 20%.
- 1D and 2D hydraulic roughness decreased by 20%.
- Sixmilebridge road bridge modelled with one arch.
- Sixmilebridge road bridge modelled with ineffective flow areas.
- 2D model utilising a 5m grid size for the entire 2D domain.
- Fluvial-Tidal joint probability analysis.

Details of the results of each sensitivity test are detailed in Section 7 of this report.

4 Data Availability

4.1 Survey data

4.1.1 Original survey data

Existing CFRAM survey data captured in 2012 was provided by the OPW in support of this project. Surveyed cross sections for the Owenogarney river and the unnamed tributary were provided. Figure 4-1, Figure 4-2 and Figure 4-3 show the available cross section data along the Owenogarney watercourse at Sixmilebridge and the areas upstream and downstream of Sixmilebridge.

The survey data includes channel cross section data including bank levels and channel bed levels. Culvert and weir structures were also surveyed with cross sections of the up and downstream faces of each structure. The locations of hydraulic structures in Sixmilebridge detailed in Section 5.4.

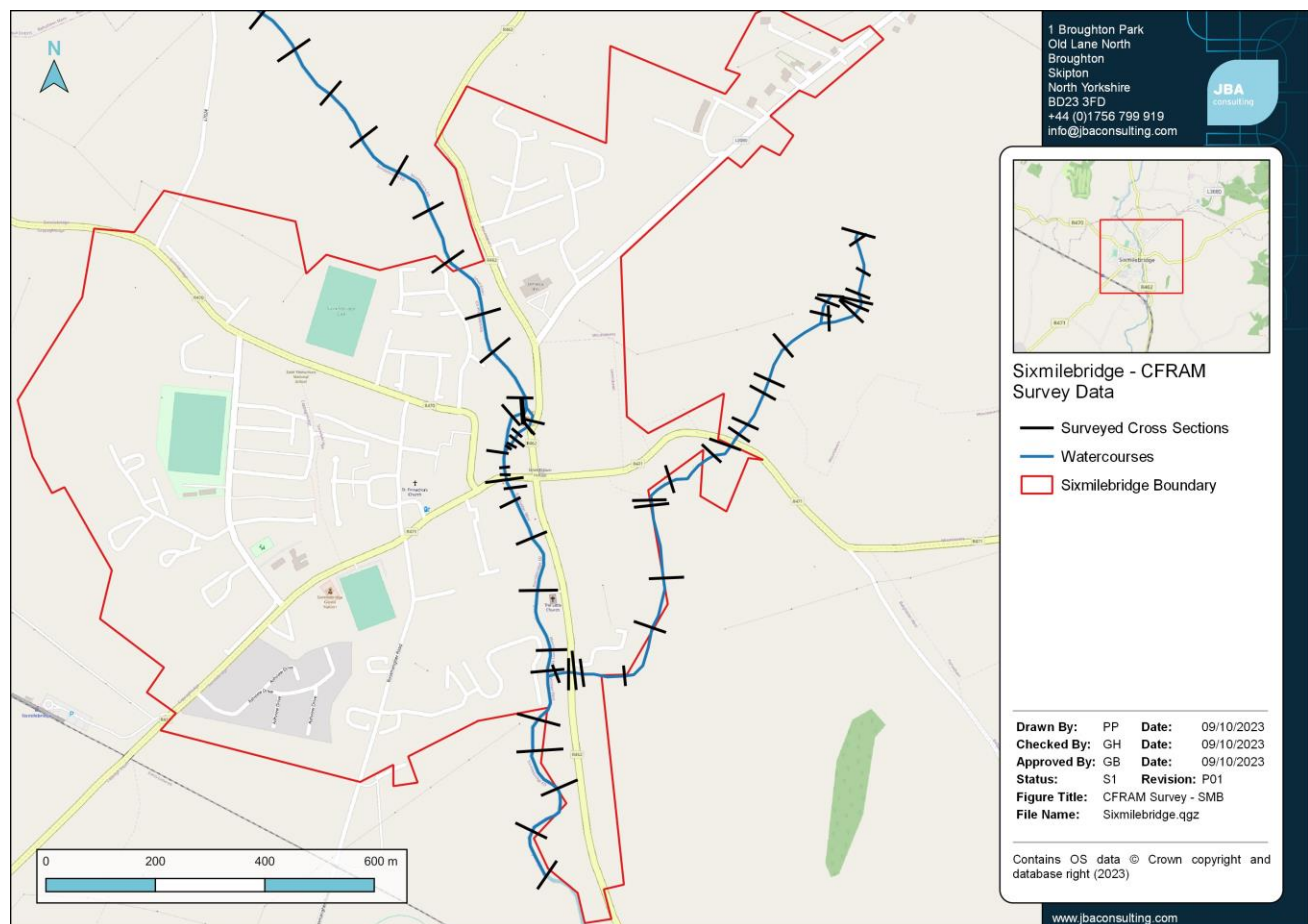


Figure 4-1: CFRAM Survey Data - Sixmilebridge

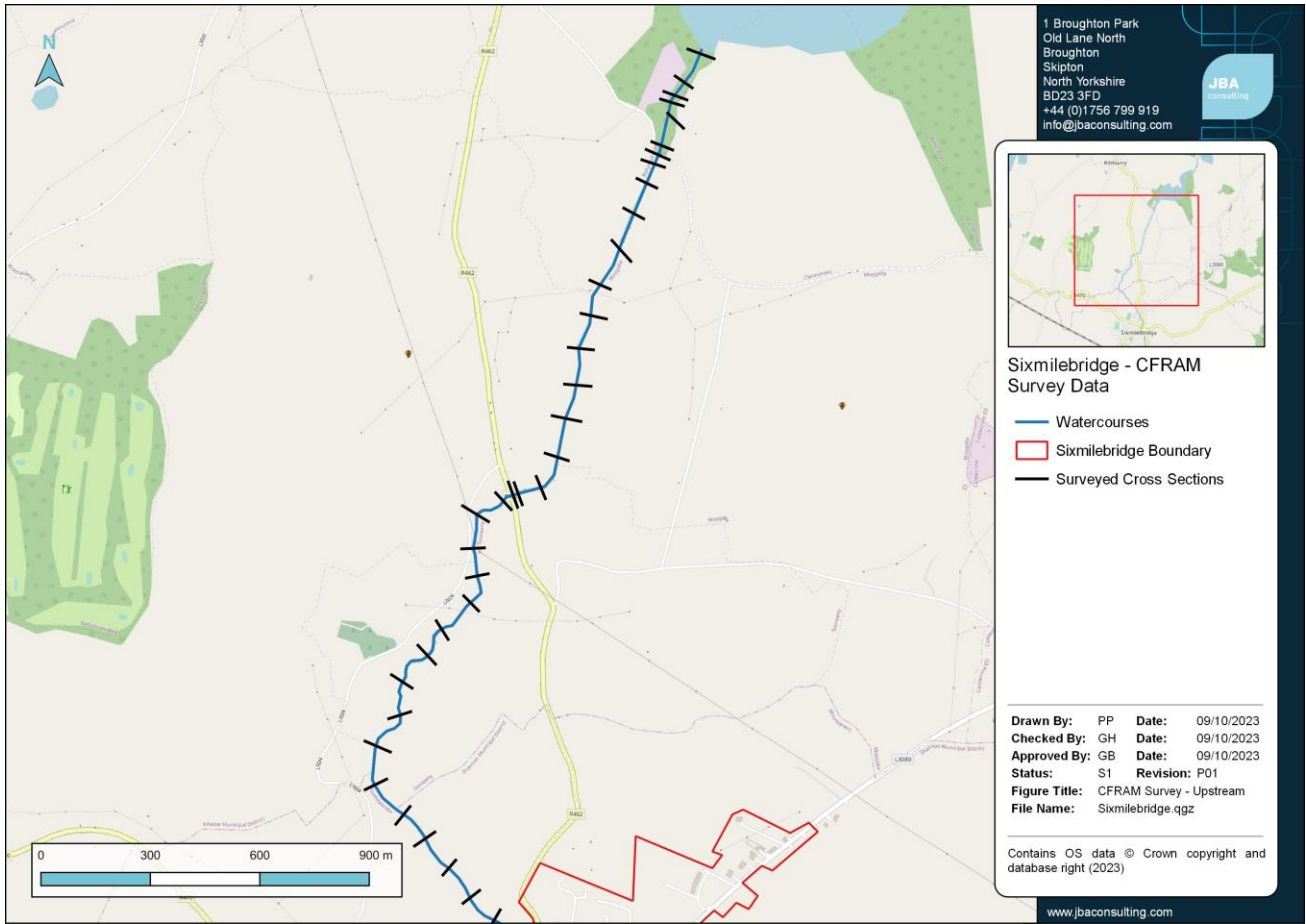


Figure 4-2: CFRAM Survey Data - Upstream

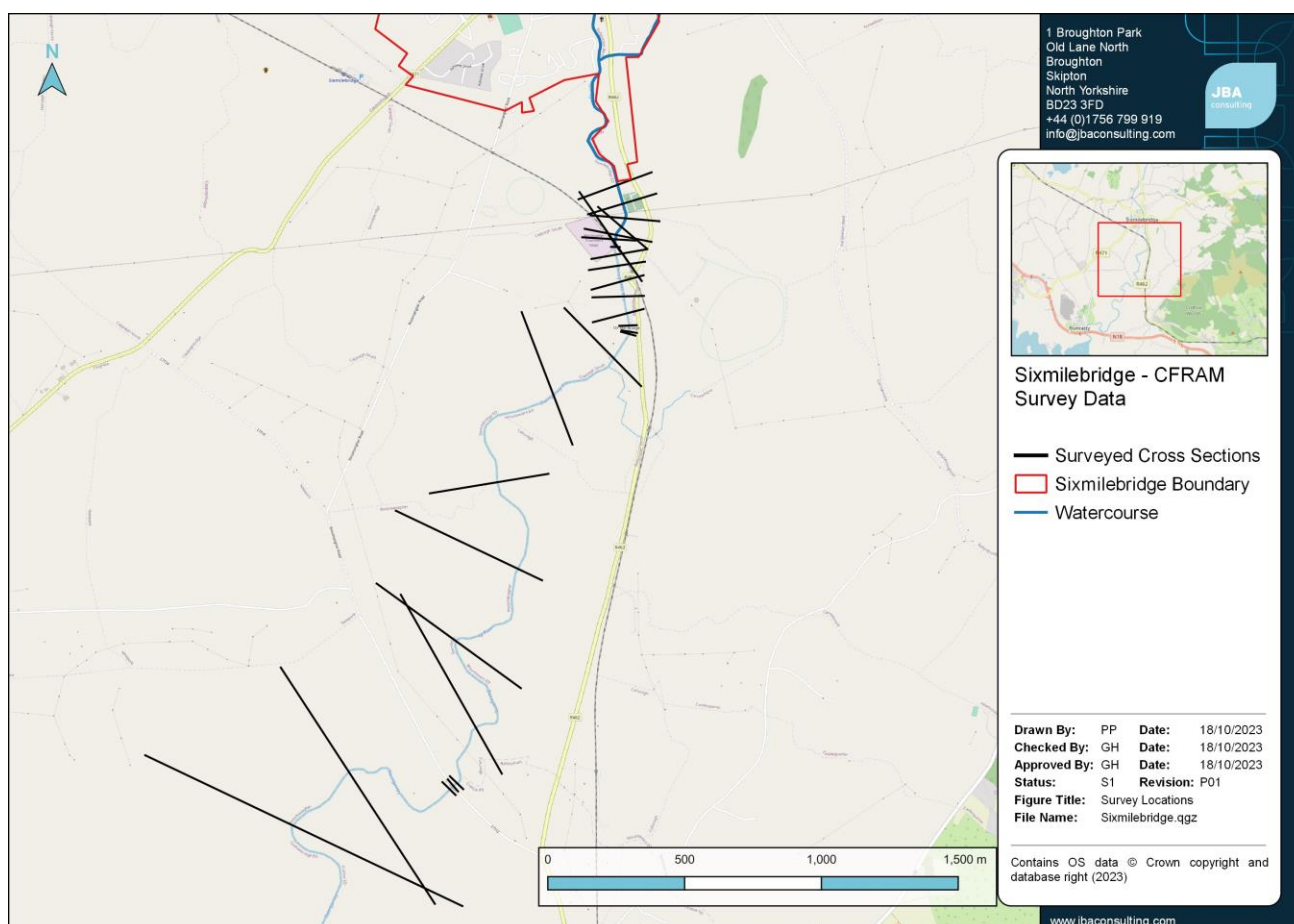


Figure 4-3: CFRAM Survey Data - Downstream

4.1.2 Additional survey requirements

Additional survey was required due to the recent development of the Gort na Núll embankment, as well as to update elements of the existing survey data that were insufficient to support this modelling study. The Sixmilebridge road bridge culvert in particular required updated survey data, as the 2012 survey included only one of the three arched culverts that comprise the entire structure. Hughes Hydro & Topographic Surveys (HHS) were subcontracted to collect the required survey data, and were instructed to provide the following:

- Topographic spot level readings of the Gort na Núll embankment to provide more accurate readings of the embankment crest levels.
- Three channel cross section surveys immediately upstream of the Sixmilebridge weir. This allowed for a more comprehensive understanding of riverbed levels in the storage area upstream of the weir as well as ground levels on the left bank.
- Updated channel cross section survey of the Sixmilebridge road bridge to provide a more complete representation of the upstream and downstream headwalls and culvert dimensions. Spot levels were also collected on the channel bed immediately upstream of the bridge which helped to gain a further understanding of the channel bed topography in relation to the upstream headwall.



Figure 4-4: Additional survey- Gort na Núll Embankment

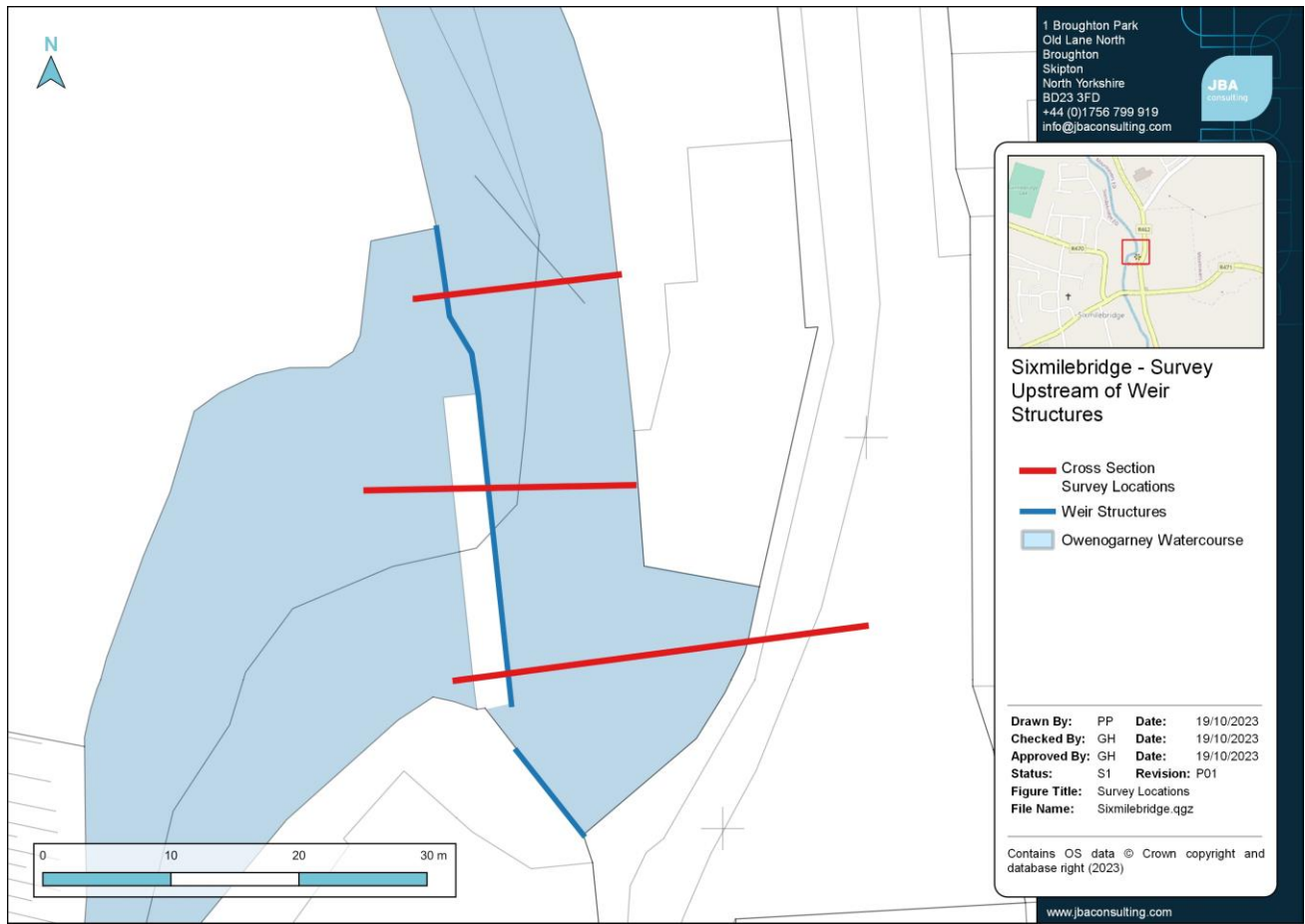


Figure 4-5: Additional Survey - Weir Cross Sections

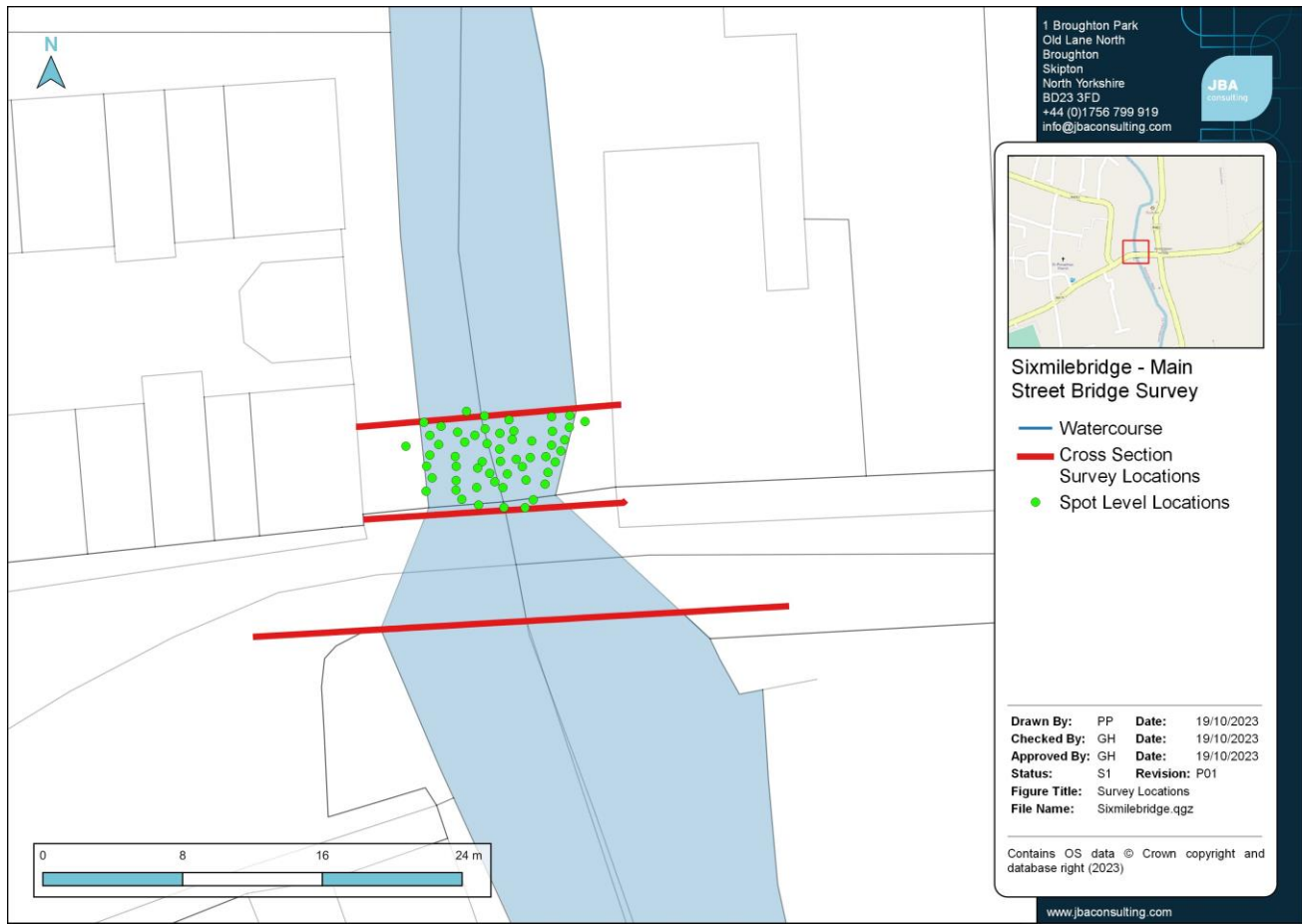


Figure 4-6: Additional Survey - Main Street Bridge

4.2 Digital terrain for 2D model domain

Digital Terrain Model (DTM) data was provided by the OPW for use in this model update study for the construction of the 2D model terrain. The DTM data provides complete coverage of Sixmilebridge and the areas of the catchment upstream and downstream of Sixmilebridge. The DTM data is high resolution with a 0.25m pixel size, giving accurate representation of floodplain features and ground levels.

The available DTM data comprises the following datasets:

- National IfSAR Data (Intermap dataset).
- LiDAR data from the CFRAM Programme, flown during 2011 and 2012.
- OPW 0.25m Coastal LiDAR data (from the Coastal Survey & Monitoring project)

Figure 4-7 shows the DTM coverage for the Sixmilebridge model update. Due to the lack of available DTM data for the upper reaches of the unnamed tributary, the extent of the tributary was reduced despite the availability of cross section data in this area. The exclusion of this data was, however, not likely to have major impacts on the results of the mapping study. The model reach extents are discussed in more detail in Section 5.2.

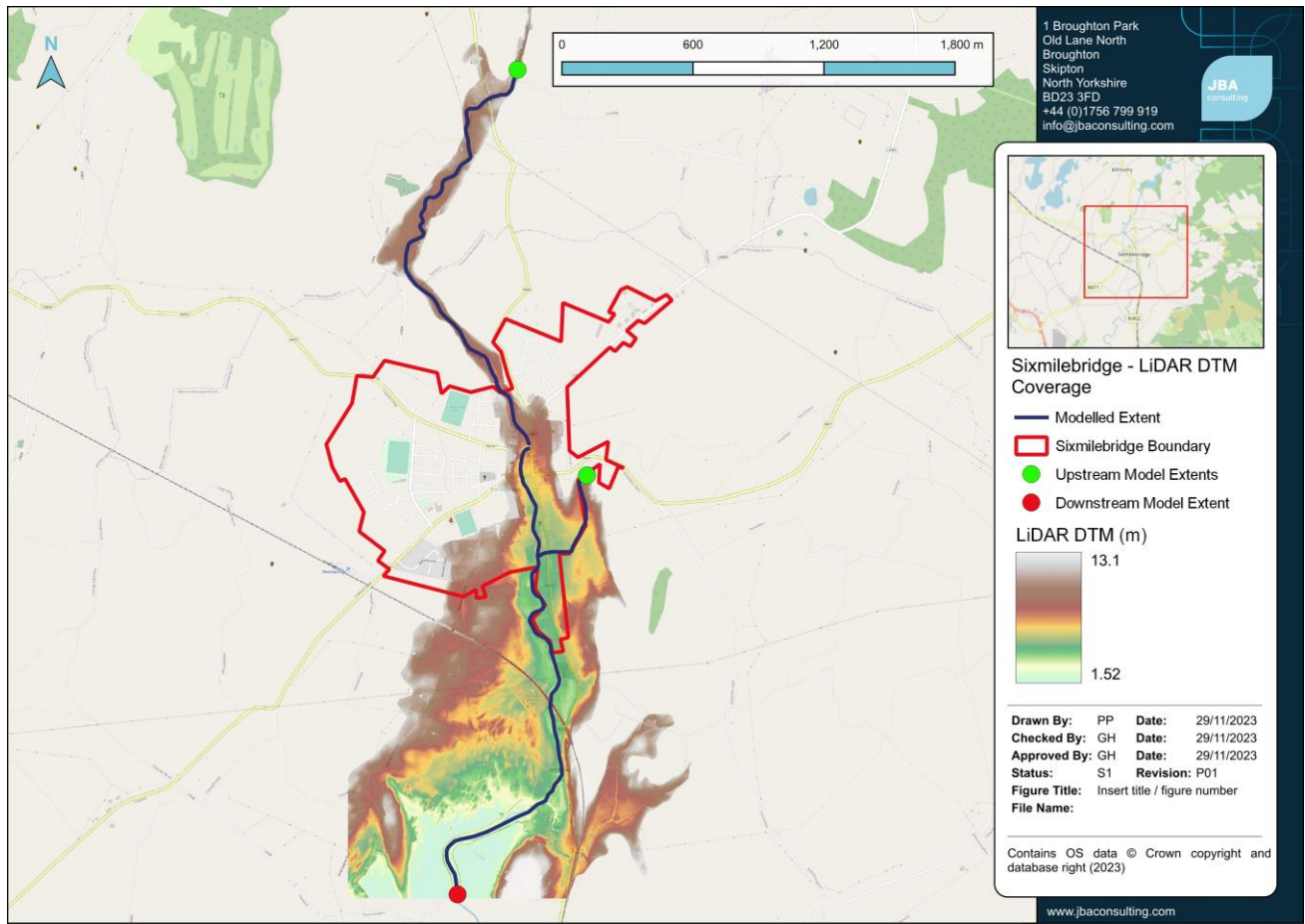


Figure 4-7: LiDAR DTM Data Coverage

5 Fluvial Hydraulic Model Construction and Schematisation

5.1 Modelling approach and software

A 1D-2D hydraulic model was developed using HEC-RAS v6.4.1, where the 1D component of the model simulates the channel fluvial hydraulics as well as any hydraulic structures along the modelled watercourse. The 2D component of the model simulates the out of bank flooding and floodplain modelling once water levels have exceeded channel capacity.

In some areas of the upstream section of the model, a 1D only modelling approach has been adopted to represent out of bank flooding as well as in-channel modelling. This approach has been applied in rural areas where 2D modelling was not deemed appropriate or beneficial to the outcomes of the modelling.

There is an 830m reach of the Owenogarney watercourse north of Sixmilebridge which was modelled entirely in 1D due to the steep nature of the channel topography.

5.2 1D-2D Model area and extent

Figure 5-1 shows the Sixmilebridge 1D-2D model extent. There are two watercourses that comprise the model, the main Owenogarney watercourse and the un-named tributary which converges with the Owenogarney watercourse south of Sixmilebridge. The model is subdivided into three 1D model reaches:

- Owenogarney Reach 1 (2.18km)
- Owenogarney Reach 2 (1.38km)
- Owenogarney Reach 3 (2km)
- Un-named Tributary (545m)

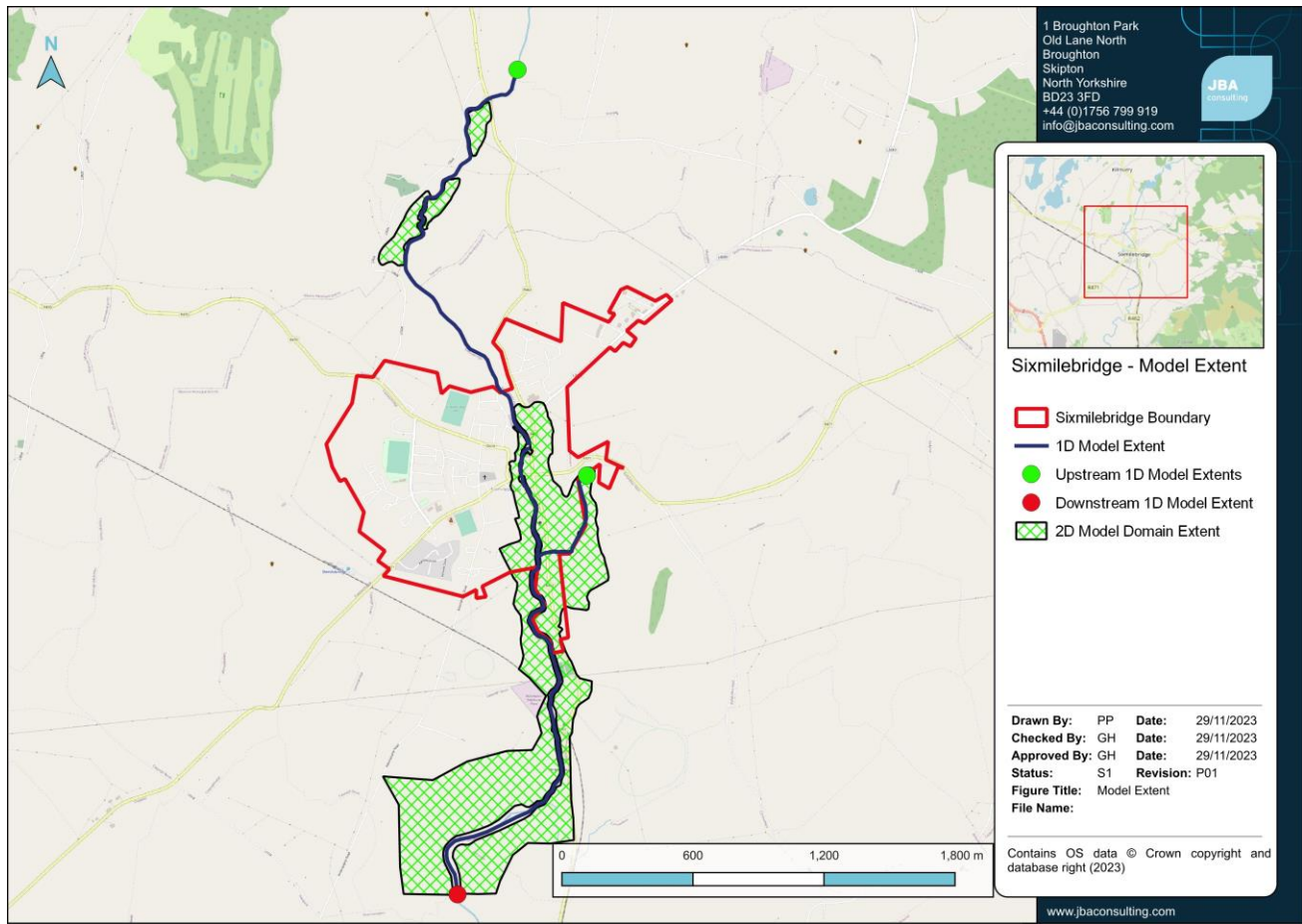


Figure 5-1: 1D-2D Model Extent

5.3 River cross-sections

The CFRAM river cross sections detailed in Section 4.1.1 were reviewed and the relevant cross sections representative of the 1D extents described in Section 5.2 were incorporated into the model. Figure 5-2, Figure 5-3 and Figure 5-4 show the modelled cross-sections throughout the extent of the model.

Interpolated cross-sections were added to stabilise the model where necessary. There are a greater number of interpolated sections in the upstream section of the model where the watercourse is characterised by a steeper bed gradient and frequent meandering, providing a more stable transition between surveyed sections.

In the downstream reaches of the model, interpolates are less frequent and are mostly utilised around channel structures such as culverts and weirs to add further stability. Interpolates have also been used at the confluence of the un-named tributary and the Owenogarney watercourse, a pre-requisite when modelling a confluence in HEC-RAS.

Cross section data from the 2023 survey was added upstream and downstream of the Main Street road bridge.

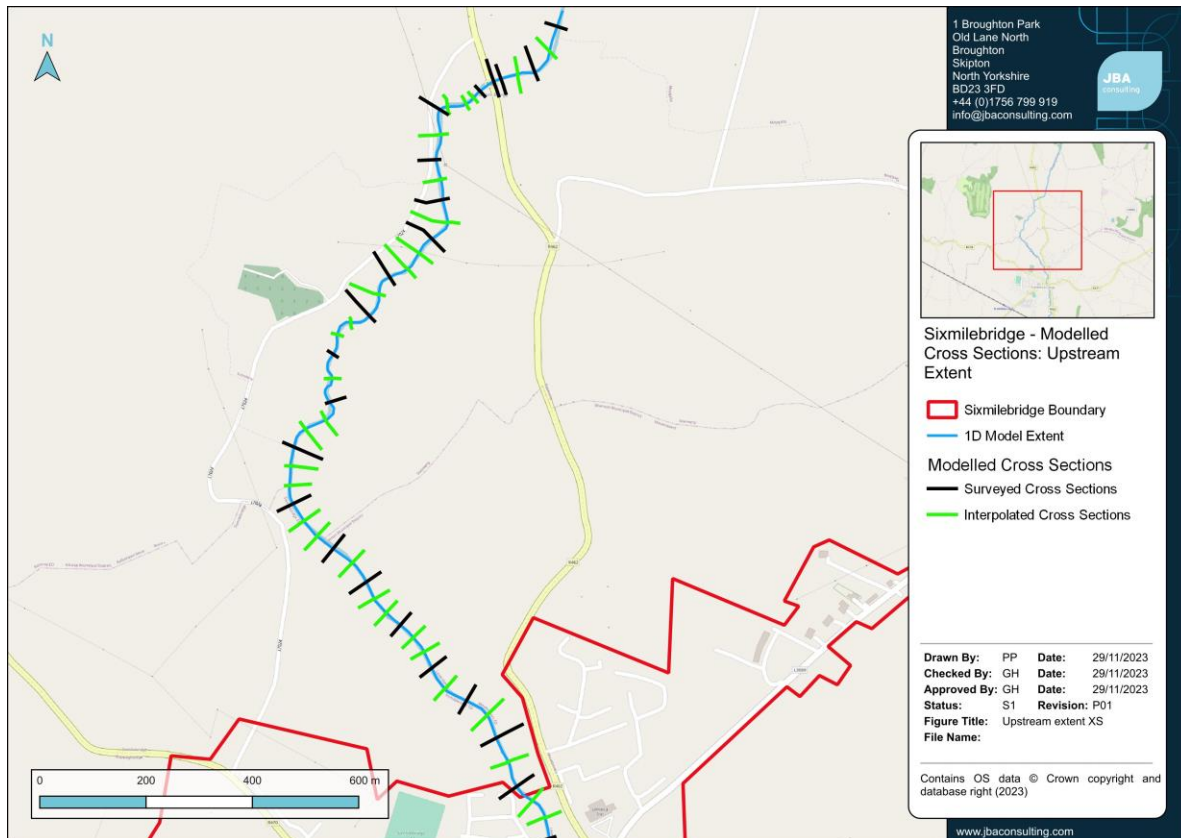


Figure 5-2: Modelled Cross-Sections - Upstream

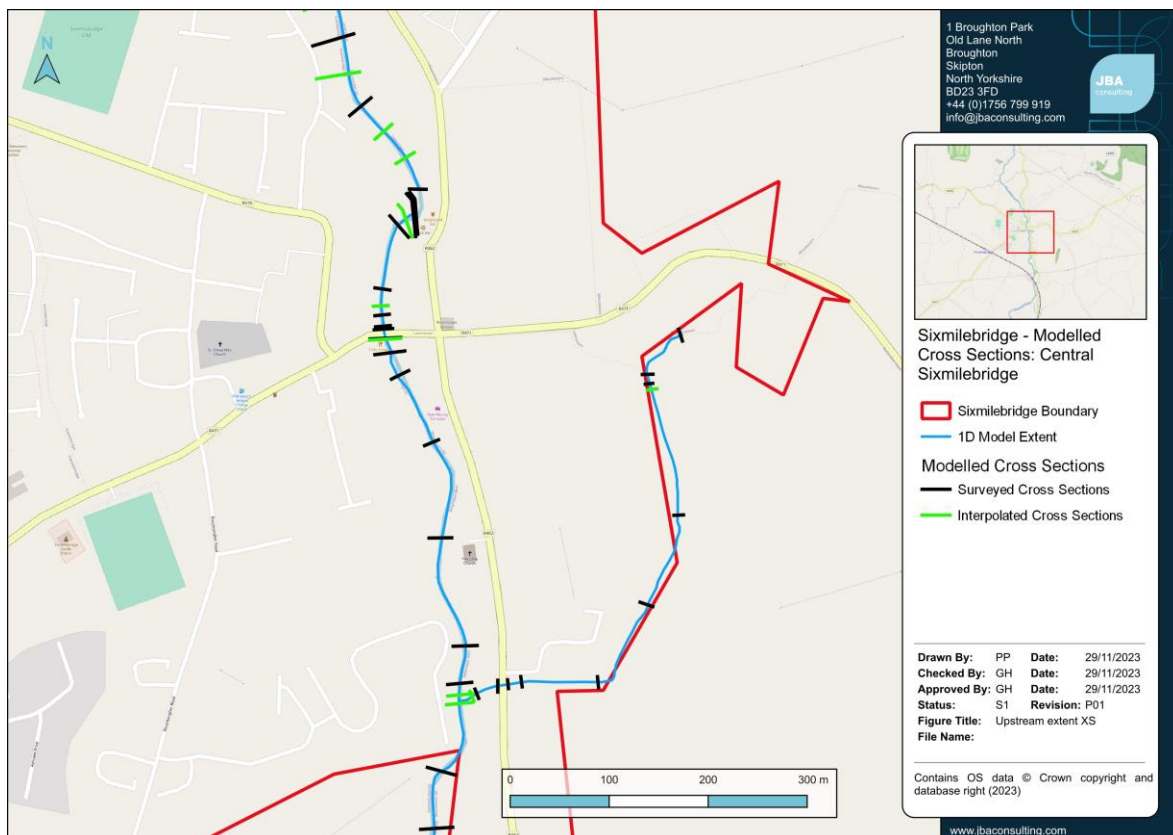


Figure 5-3: Modelled Cross-Sections - Sixmilebridge

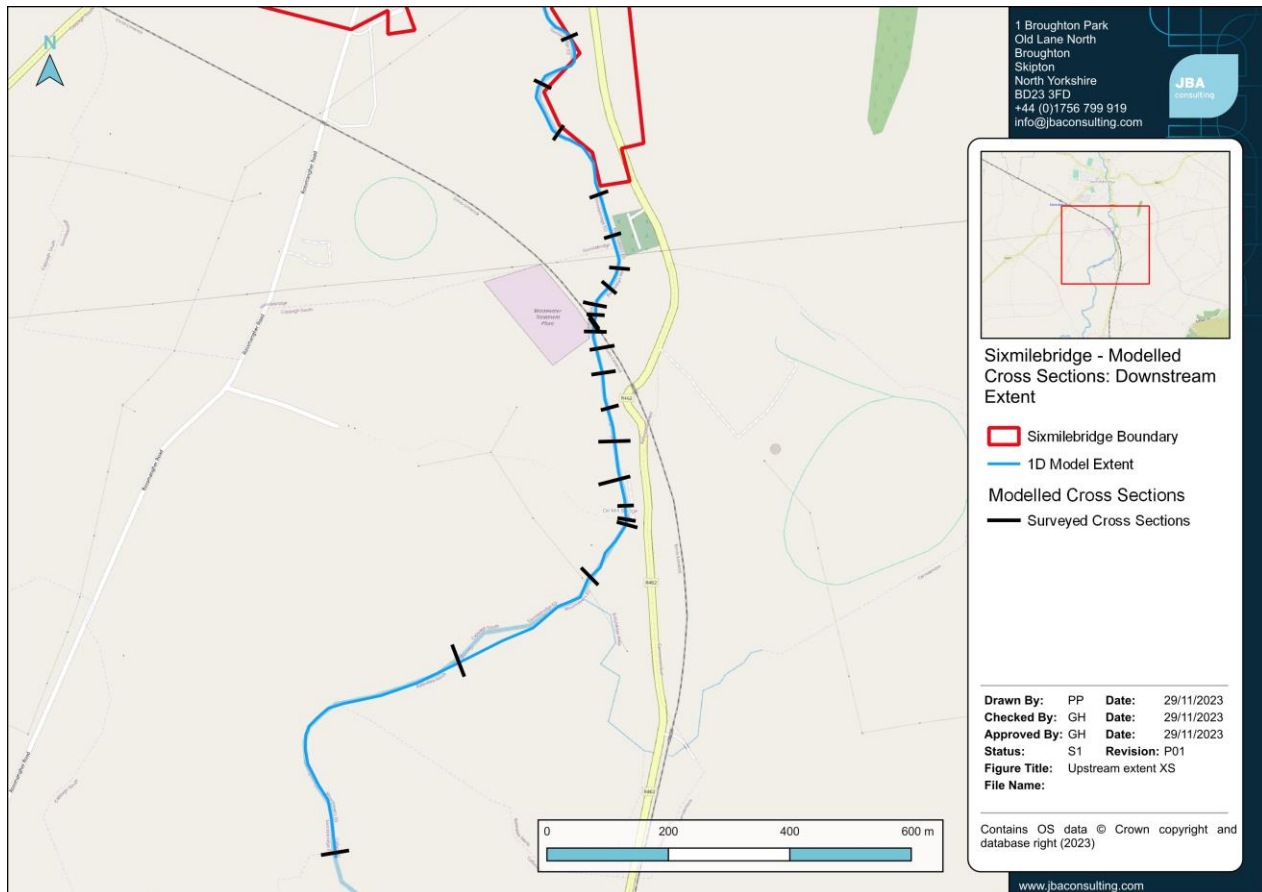


Figure 5-4: Modelled Cross-Sections - South Sixmilebridge

5.4 Structures

5.4.1 Owenagarney (Main River)

There are a number of hydraulically significant structures which have been incorporated into the model using data taken from the CFRAM survey and 2023 survey.

All modelled structures on the main Owenagarney river are located in the vicinity of the old mill in central Sixmilebridge in close proximity to one another. These are described in Table 5-1 and Figure 5-5.

Table 5-1: Owenogarney Modelled Structures

Structure name	Description	Photo
Old mill weir 001	The main weir on the Owenogarney river, located adjacent to the old mill. The weir conveys flow laterally from the main channel as it enters a storage area upstream of the mill. The weir is broad crested and approximately 36.62m in length with a crest level of approximately 7.13mAOD.	
Old mill weir 002	Small control weir adjacent to the mill pond storage area. Allows a controlled conveyance of flow to an internal dual culvert system beneath the old mill. Flow is then directed back into the main channel via two manmade channels.	
Sixmilebridge road bridge	The Main Street road bridge. The bridge structure on Sixmilebridge Main Street comprises three arched culverts, all uniform in shape and approximately 3.41m wide and 1.7m high.	

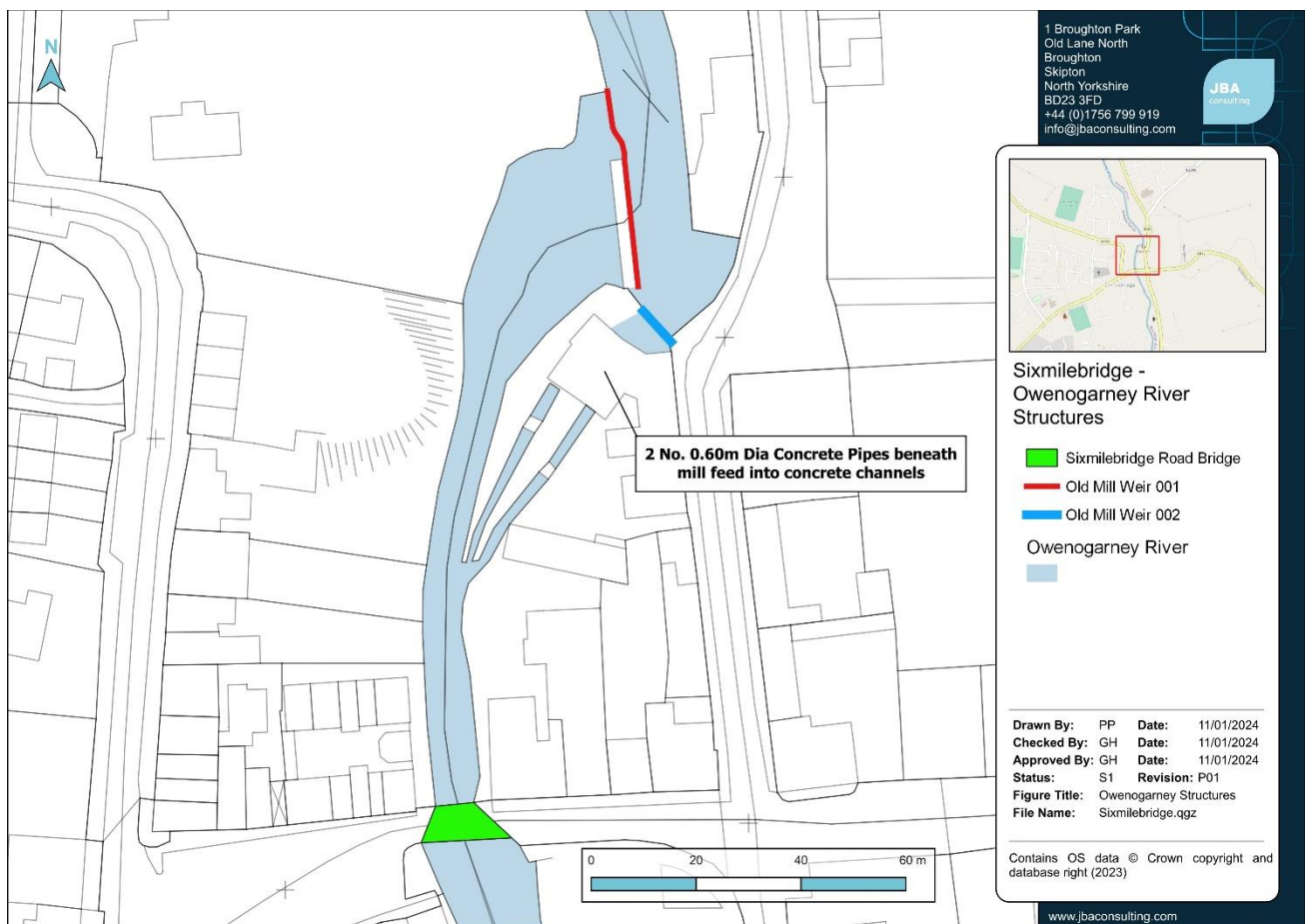


Figure 5-5: Owenogarney Structures

5.4.2 Unnamed tributary

The modelled tributary includes two structures:

- An upstream bridge with two 0.45m diameter pipes conveying flow downstream. The bridge deck of this structure is approximately 0.9m above channel bed levels.
- A bridge structure in the downstream region of the tributary with a box culvert approximately 1.38m wide and 0.67m high on the upstream face. The downstream face, although has similar dimensions to the upstream face, is more arched in shape compared to the boxed shape upstream.

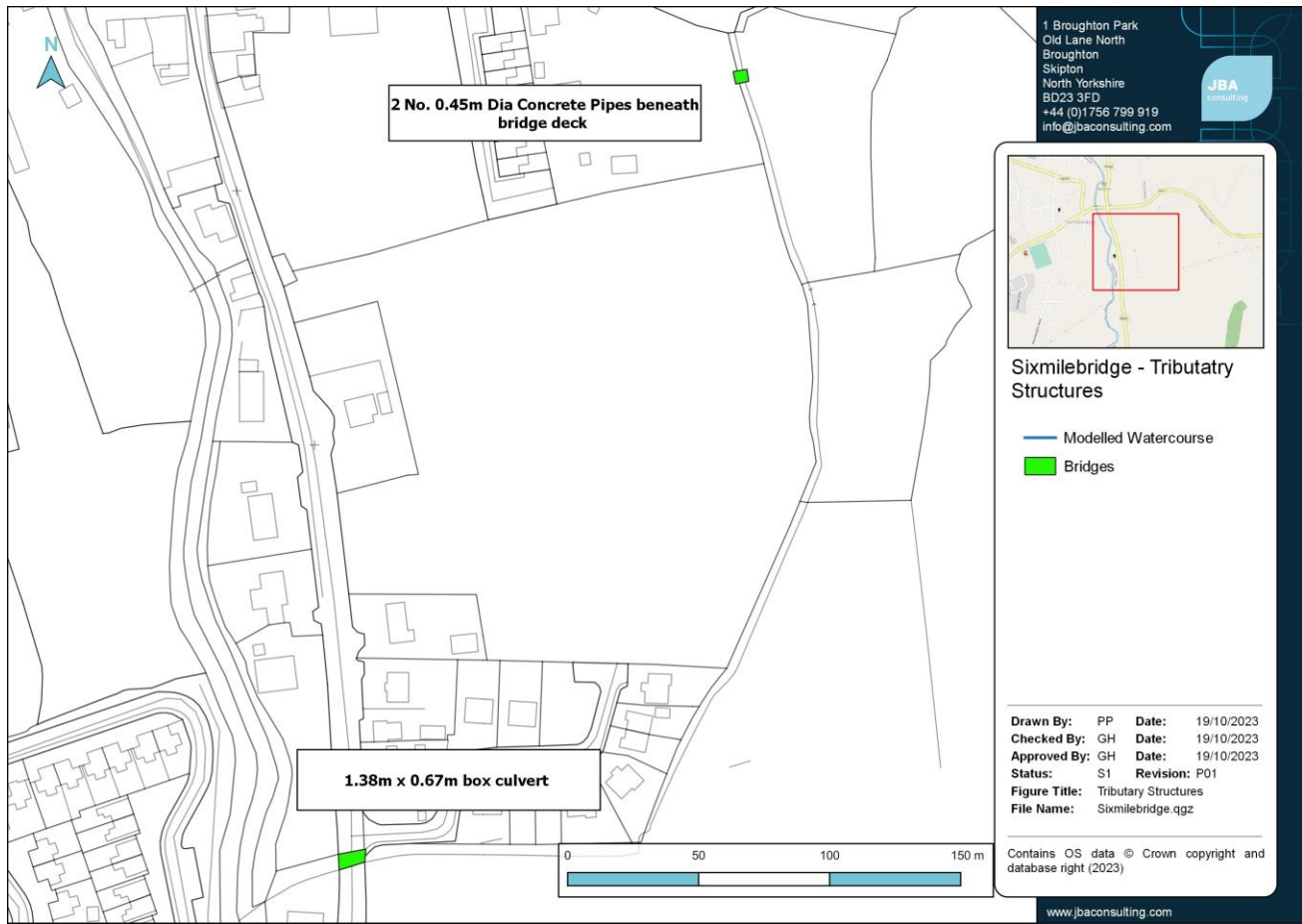


Figure 5-6: Tributary Structures

5.5 Structure model schematisation

5.5.1 Tributary bridges

The bridge structures on the tributary were modelled according to the structure dimensions provided in the original CFRAM survey. The upstream bridge of the tributary (River section 472) was modelled with a flat bridge deck with culvert pipes, while the downstream bridge (River section 50) was modelled as a bridge structure. The downstream bridge utilises a submerged inlet coefficient of 0.8 during high flows.

5.5.2 Owenogarney road bridge

The original CFRAM survey dimensions for the road bridge on the Owenogarney watercourse were not utilised in the model build for this bridge due to having incomplete data for the two wing culverts that form part of the structure. The bridge was re-surveyed during the 2023 survey carried out for this study; however, the wing culvert dimensions could not be acquired as they are located internally within the main structure. The bridge was modelled on the assumption that all three culverts are uniform in length and shape to the central culvert arch based on site photographs and observations from the surveyor on site.

The wing culverts of the bridge are also skewed from the main channel at their respective upstream inlets for approximately 1m before running parallel to the central culvert. In order to account for this skew, the upstream inlets have been reduced in size. This is a simplification of the actual structure for the benefit of the stability of the model, whilst enabling an appropriate level of flow through the structure.

The bridge schematisation (Figure 5-7) comprises a flat bridge deck with three arched culvert structures. The bridge utilises a submerged inlet coefficient of 0.8 during high flows.

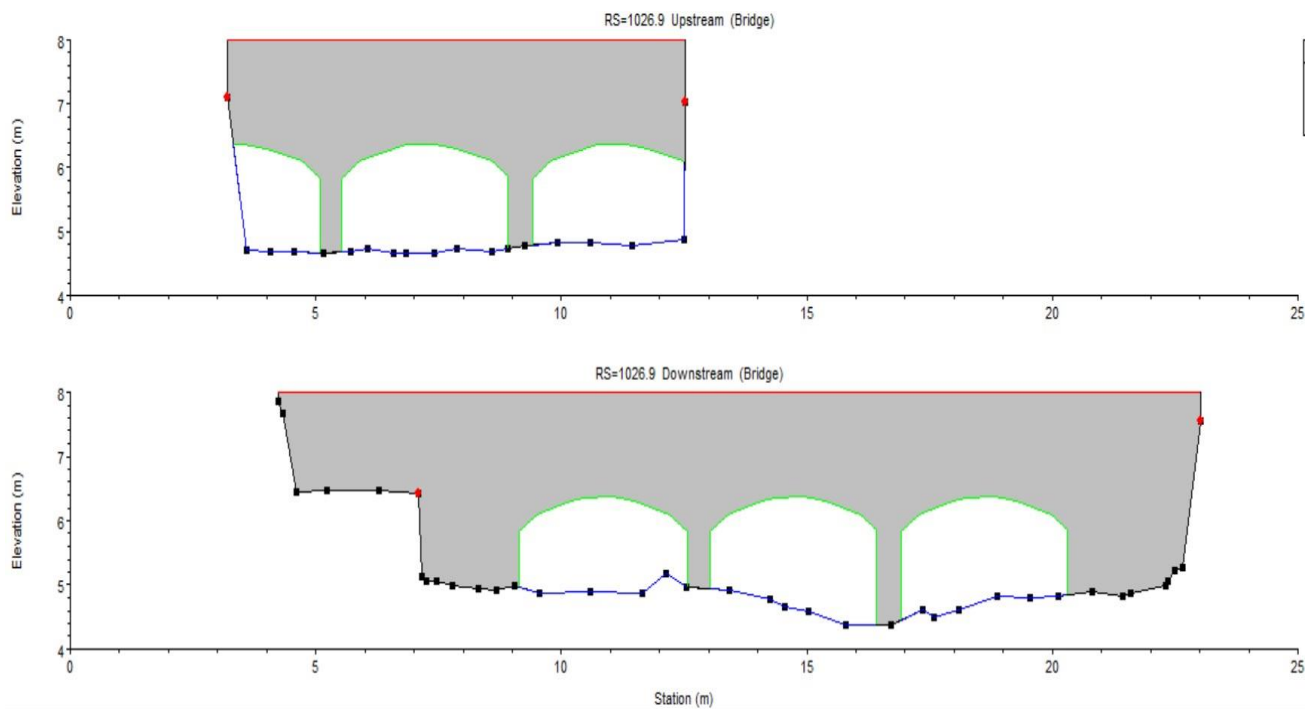


Figure 5-7: Main Street Bridge Model Configuration

5.5.3 Old mill storage area

A small storage area (Figure 5-8) was implemented immediately upstream of the old mill weir structures to stabilise the model and account for the abrupt change in velocity in this area. A storage area volume curve was calculated from a DTM modification to represent the storage area channel bed from available survey data.

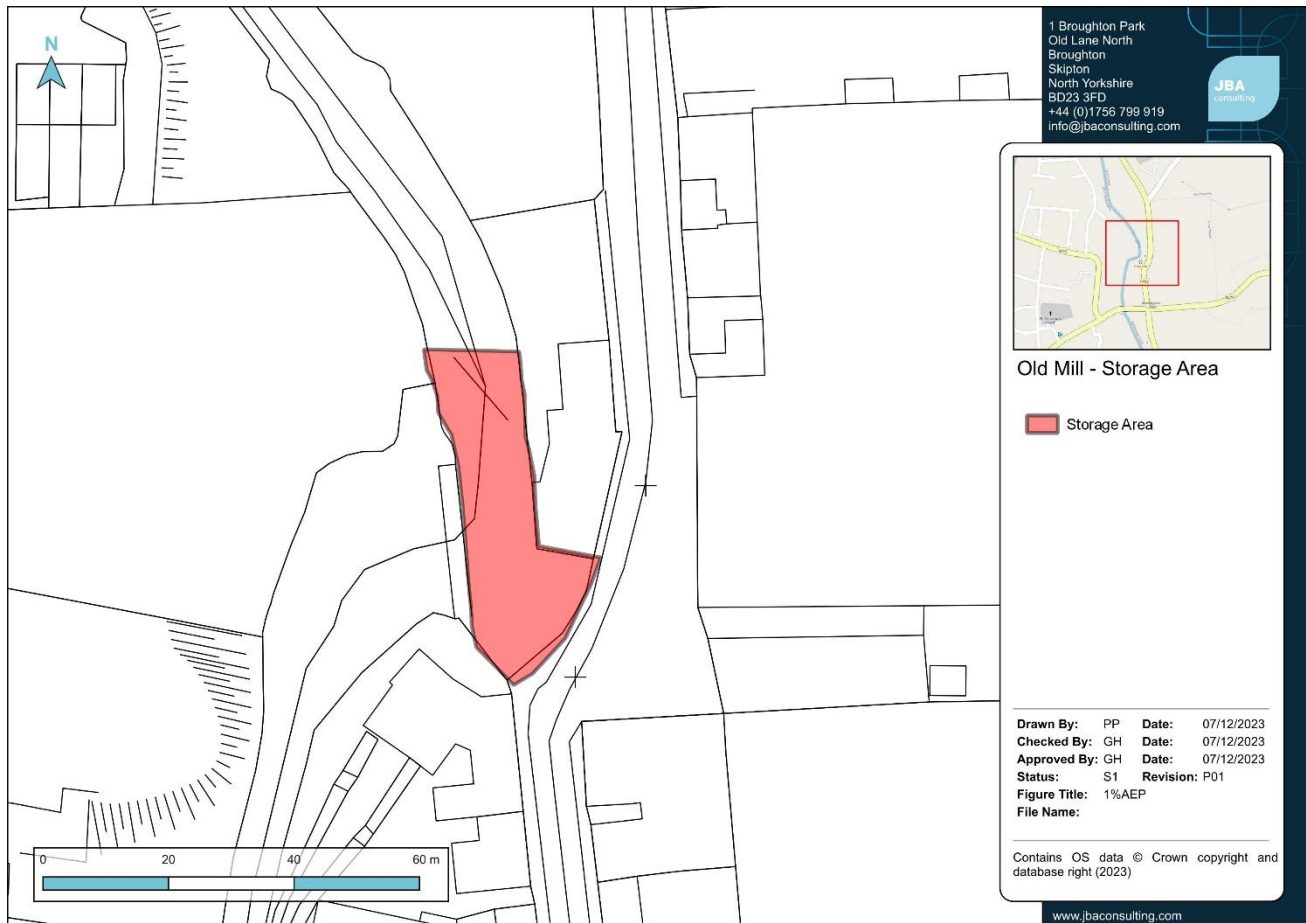


Figure 5-8: Old Mill Storage Area

5.5.4 Old mill weir 001

The old mill weir 001 was modelled using an inline structure with crest levels derived from the CFRAM survey data. A sluice gate adjacent to the weir is assumed to be shut based on site observations. An unorthodox weir coefficient has been applied for use in this weir due to forming part of a complex 1D network which resulted in unrealistic flow conveyance over the weir. A detailed justification of this modelling decision is outlined in Section 7.1.

5.5.5 Old mill weir 002

The model schematisation of the old mill weir 002 (Figure 5-9) is a simplification of the overall system to benefit the stability of the model and quality of the outputs. The schematisation comprises two 0.6m diameter culvert pipes embedded within a connection line that links the 1D storage area to the 2D model domain. The culvert pipes convey flow directly into two conveyance channels which were incorporated into the 2D domain using a DTM modification. From here flow is allowed to re-enter the 1D model domain via a 1D/2D lateral structure connection. All culvert and conveyance channel dimensions were modelled using surveyed dimensions from the CFRAM survey.

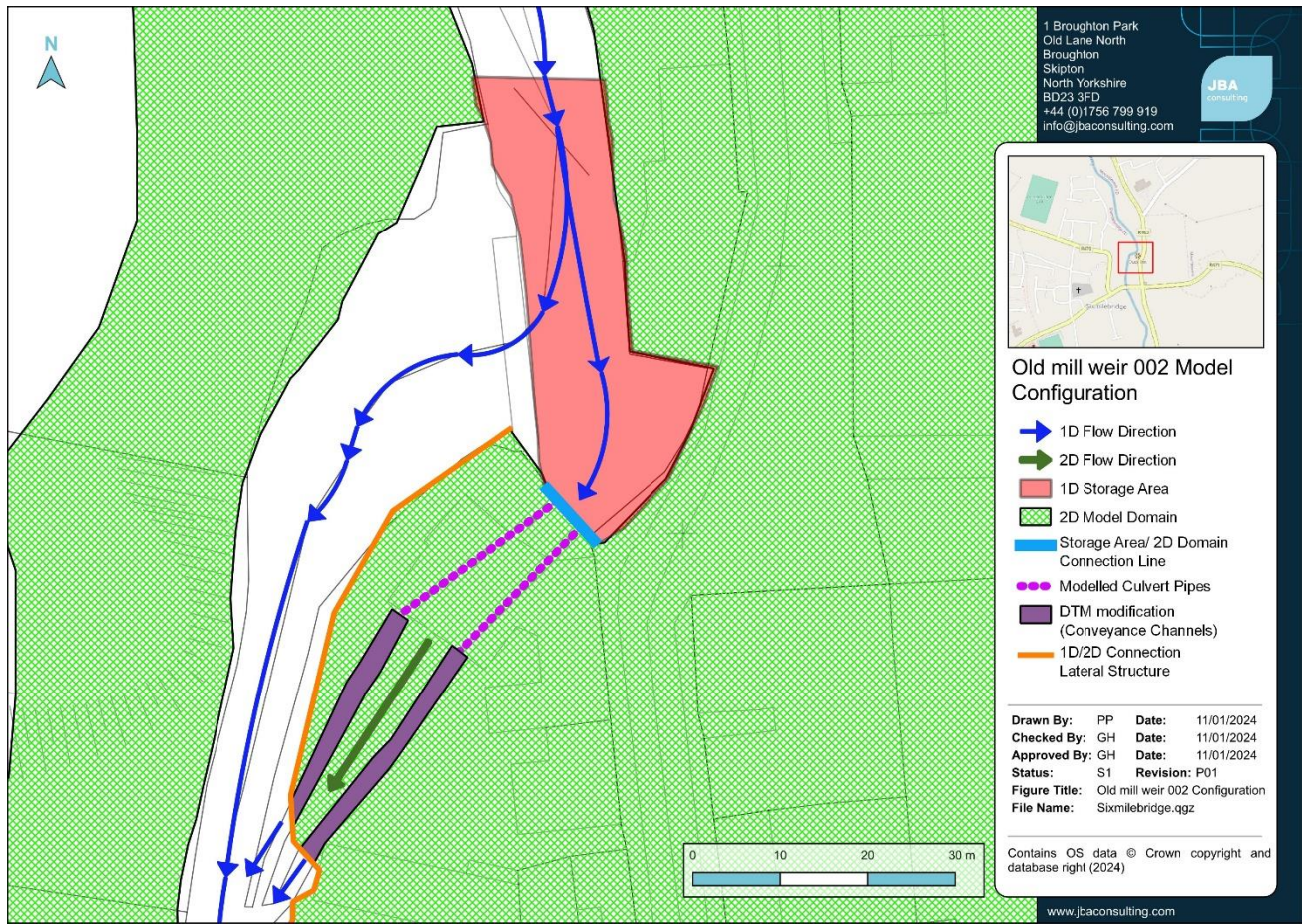


Figure 5-9: Old Mill Weir 002 Model Schematisation

5.6 Floodplain schematisation

There is a varied schematisation of floodplain modelling, with a 1D-only modelling approach adopted at the upstream reaches of the model and a 2D floodplain modelling approach adopted in the downstream reaches of the model in Sixmilebridge.

5.6.1 1D approach

The variation between 1D and 2D modelling approaches is largely driven by the local topography and channel characteristics. A 1D floodplain modelling approach is adopted in more rural areas upstream of Sixmilebridge where the channel topography is characterised by large, steep-sloping banks, meaning any out of bank flow is likely to run parallel to the channel direction. 1D modelling is most suitable in these areas where there is a relative uniformity in flow direction. The 1D modelling approach was achieved by extending channel cross-sections onto the left and right floodplains at each section. This resulted in cross-section data where channel bed elevations are derived from survey data and floodplain elevations are derived from DTM data.

5.6.2 2D approach

Sixmilebridge itself is located in the downstream reach of the model where the topography is flatter with more development. This means out of bank flows are much more likely to flow laterally and through a much coarser terrain. A 2D modelling approach was adopted in this area, which utilises a grid defined by DTM elevation data where flows are able to navigate laterally between cells. Some areas in the upstream reaches of the model utilised a 2D domain where the floodplains were likely to experience more lateral flow.

Figure 5-1 shows the locations where a 2D floodplain model domain was utilised.

5.7 2D model domain grid

Where a 1D-2D model approach was adopted, varied 2D model grid sizes can be defined depending on the level of detail required or where the local topography is likely to impact hydraulic flow. Smaller grid sizes are therefore typically used in more urbanised areas where the subtleties in the terrain are given more detailed representation.

In the Sixmilebridge model, a standard grid cell size of 5m was used for all 2D domains, and 2D refinement regions were utilised in the developed areas of Sixmilebridge to reduce the cell size to 2m. Figure 5-10 shows the location in the 2D domain utilising a reduced cell size. It should be noted that this is the only region of the model in which a 2m grid cells size has been used.

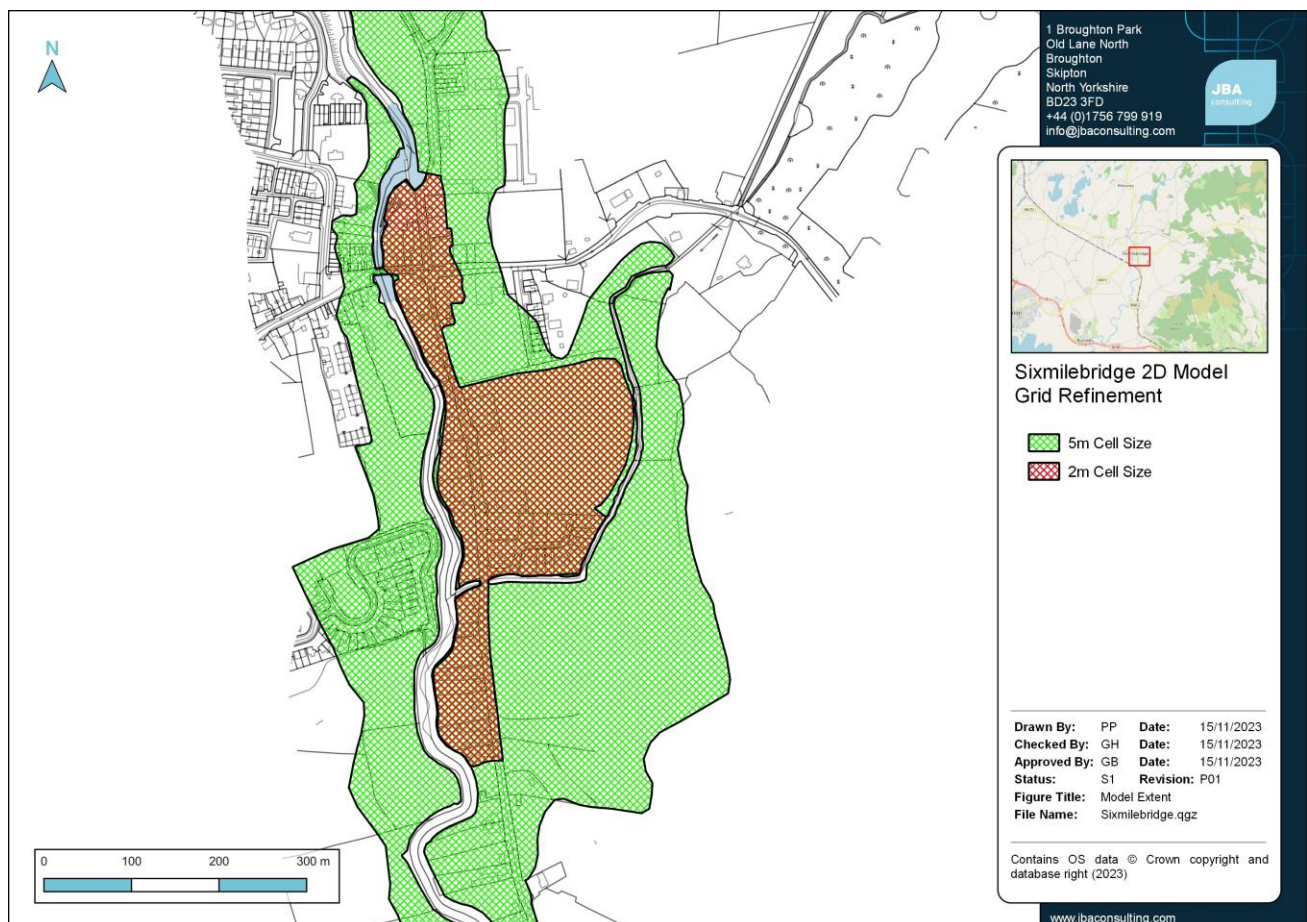


Figure 5-10: 2D Model Grid - 2m Refinement

5.8 Model break lines in the 2D domain

2D model break lines were further used to define abrupt linear changes in topography in the 2D domain. Model break lines are typically useful where roads, pavements, ditches, or other such features may influence flow paths on the 2D floodplain, and break lines enable the alignment of grid cells along abrupt changes in elevation to allow flows to transition more effectively between cells. This not only improves the quality of flood mapping outputs but also 2D model stability.

Figure 5-11 shows the locations where break lines have been used to define linear features in the Sixmilebridge domain. This includes the R462 road which is a major influence on flood flows on the Owenogarney left bank floodplain. Other features include local ditches and embankments in the southern areas of Sixmilebridge which are also likely to influence flow paths.

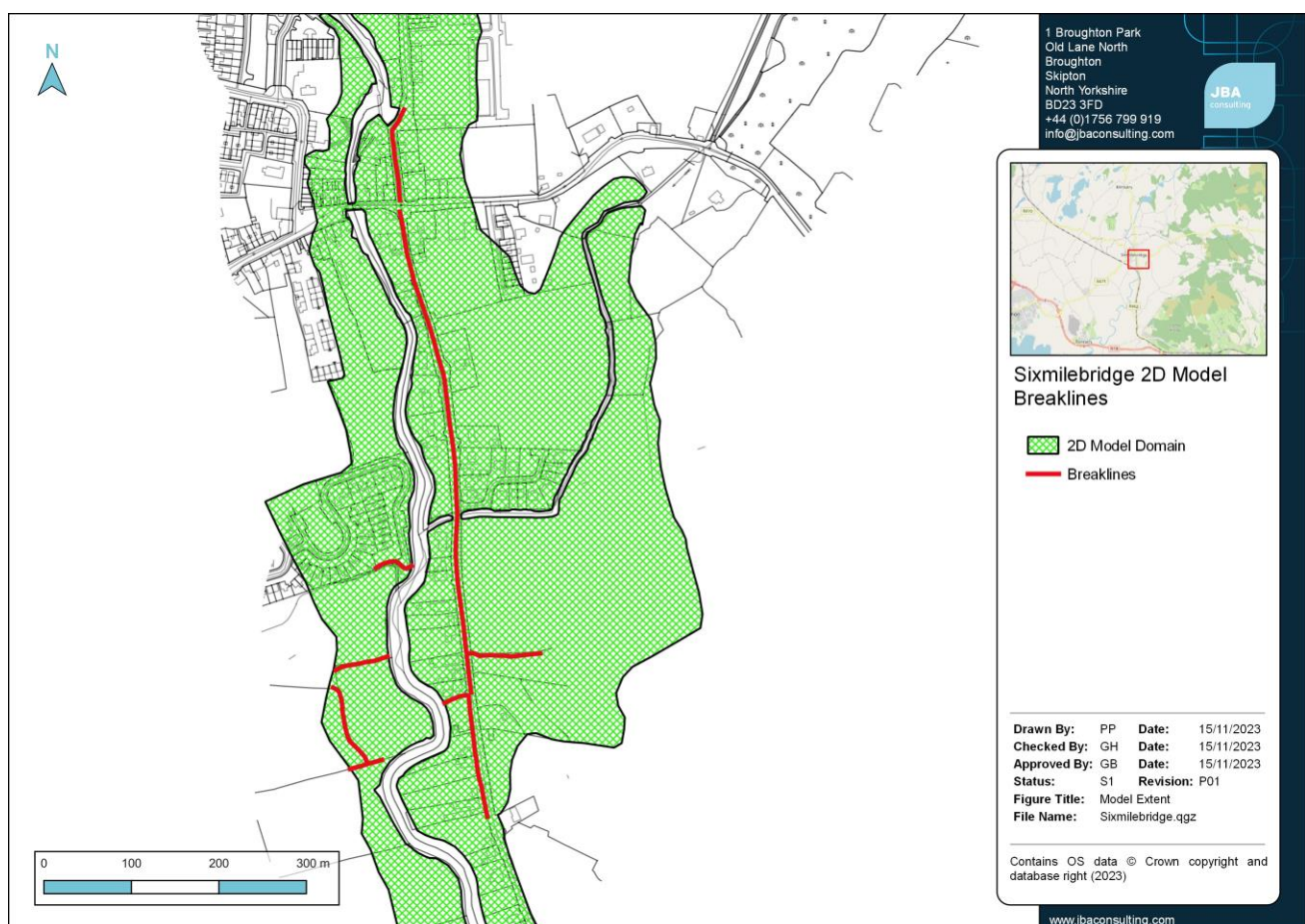


Figure 5-11: 2D Model Break lines

5.9 DTM modifications

Modifications were made to the underlying DTM data where features required more accurate representation or inaccuracies were observed in the DTM when compared to survey data. DTM modifications were applied within HEC-RAS by defining a modification extent with a polygon or line features and using control points to apply levels derived from survey data. The following modifications were made:

- The unnamed tributary had an unrealistic channel depth compared to cross-section survey data, meaning flood depths was not completely represented in initial model outputs. A line feature was applied along the length of the tributary enforcing the lowest elevation level from each cross section and interpolating between each section. Bank levels were also enforced where deemed to be inaccurate compared to surveyed levels.
- Spot levels survey levels were enforced along the length of the Gort na Núll embankment for more accurate representation of the embankment crest levels.
- Survey levels were enforced along the outlet channels of the Old Mill culvert system for more accurate depth representation.
- Cross section survey levels of the mill pond storage area upstream of the old mill were used to enforce the channel bed depths into the DTM. This allowed for a volume calculation for the 1D storage area used in this area of the model.
- Road levels taken from cross section survey data at the mill pond storage area were applied to the DTM. DTM levels in this area were inaccurate due to the DTM filtering process which removes various features from the DTM. Overtopping from this storage area is a key point of flood inundation onto the floodplain, therefore it was important to represent road levels accurately.
- Threshold levels were applied to buildings in the central areas of Sixmilebridge in order to represent potential flood ingress to properties. It should be noted that the threshold levels are indicative and were inferred from street level mapping and photographs and not derived from detailed surveying methods.

5.10 Hydraulic roughness

Hydraulic roughness, or friction, is represented by Manning's coefficient "n" in the hydraulic model. The value of 'n' accounts for a range of factors that influence overall roughness either in the channel or across the floodplain. Factors included within the overall evaluation of Manning's 'n' include bed materials and size, vegetation, surface irregularities, channel bed forms, erosional and depositional features, channel sinuosity, and obstructions, all of which influence channel and floodplain conveyance.

5.10.1 Channel roughness

Channel roughness values are assigned to three elements of each channel cross-section; the channel bed, the left bank and the right bank. A uniform Manning's n value of 0.035 was applied to the channel bed for all reaches of the model up to the Sixmilebridge railway crossing, representing the roughness associated with clean straight channels with a moderate amount of weeds and stones characteristic of the modelled watercourses. Beyond the railway crossing to the downstream extent of the model, an increased value of 0.04 was applied to the channel bed to account for increased vegetation within the channel observed from photographs of areas at the downstream reaches of the model. The left and right banks of the channel were assigned Manning's n values between 0.04 and 0.06 throughout the model, which accounts for the additional roughness from vegetation, shrubs

and stones found on the banksides. These values were based on photographs and visual observations obtained during site visits.

5.10.2 Floodplain roughness

Manning's n values were also assigned to the varying surface types on the 2D floodplain. A land cover layer was developed which defined different surfaces into spatial categories with a Manning's n value assigned to each category. Table 5-2 displays the floodplain roughness values.

Table 5-2: Floodplain roughness values

Land use	Manning's n
Buildings	0.3
Gardens	0.07
Grassland	0.045
Roads	0.025
Woodland	0.1

5.11 Model boundaries - inflows

Model inflows were derived from the detailed hydrological analysis carried out for the local catchment. The hydrology report included in Appendix A details the hydrological analysis methods employed for this study, as well as the calculated peak flows for both the Owenogarney watercourse and the unnamed tributary. Calculated flow hydrographs were applied at the upstream extents of each watercourse.

Although flows can also be distributed laterally at appropriate Hydrological Estimation Point (HEP) locations, this method was not employed as the Generalised Extreme Value (GEV) growth curve used to calculate peak flows for the Owenogarney watercourse indicated that additional inflows were not required to maintain the estimated growth.

Table 5-3 displays the peak flows from the inflow hydrographs of each modelled design event, taken from the upstream-most HEP from the hydrological analysis study.

Table 5-3: Model Peak Flow Values

Annual Exceedance Probability (AEP)	Owenogarney Peak Flow (m ³ s)	Tributary Peak Flow (m ³ s)
50%	21.81	1.04

Annual Exceedance Probability (AEP)	Owenogarney Peak Flow (m³/s)	Tributary Peak Flow (m³/s)
20%	27.27	1.3
10%	30.98	1.48
5%	34.47	1.64
2%	38.61	1.84
1%	41.88	2.0
0.5%	45.15	2.15
0.1%	52.14	2.49

5.12 Model boundaries - Downstream conditions

The Owenogarney river discharges to the Shannon estuary which is located approximately 5.5km southwest of Sixmilebridge. Fluvial-tidal joint probability has therefore been considered in this modelling study by utilising fluvial-tidal combinations at the downstream boundary of the model.

A stage hydrograph downstream boundary was applied using peak tidal curves derived from the Irish Coastal Wave and Water Level Modelling Study (ICWWS) for the Owenogarney catchment (Table 5-4). Table 5-5 shows the fluvial-tidal joint-probability combinations used for all present-day events, as well as additional combinations used for sensitivity testing.

Table 5-4: Owenogarney Peak Tidal Levels

AEP	Present Day	MRFS	HEFS
50%	3.78	4.28	4.78
20%	3.99	4.49	4.99
10%	4.15	4.65	5.15
5%	4.31	4.81	5.31
2%	4.52	5.02	5.52
1%	4.67	5.17	5.67
0.5%	4.83	5.33	5.83
0.1%	5.19	5.69	6.19

Table 5-5: Fluvial-Tidal Joint Probability Combinations

Fluvial %AEP Event	Tidal %AEP Event
All design runs	50% AEP
10%AEP	10%AEP
1%AEP	10%AEP
10%AEP	1%AEP
10%AEP	0.5%AEP

6 Model Calibration and Verification to Historical Events

6.1 Verification against Hydrological Estimation Points (HEP)

Peak flow values from the model results have been verified against the peak flow estimates for a range of HEPs along the Owenogarney watercourse. The HEP peak flow values were calculated during the hydrological analysis for this study and detailed in Section 6.1.8 in the Hydrology report. Figure 6-1 shows the location of each HEP used for this analysis, and Table 6-1 shows the peak flow values derived from the model results at each respective location for a range of return periods. The peak flow value measurements take into account both channel and floodplain flow at each location.

The results of this analysis indicate that the model output flows are generally consistent with the estimated peak flows at each location, with discrepancies generally remaining under 1m³/s. For this reason, it was not considered necessary to add additional inflows along the modelled watercourse.

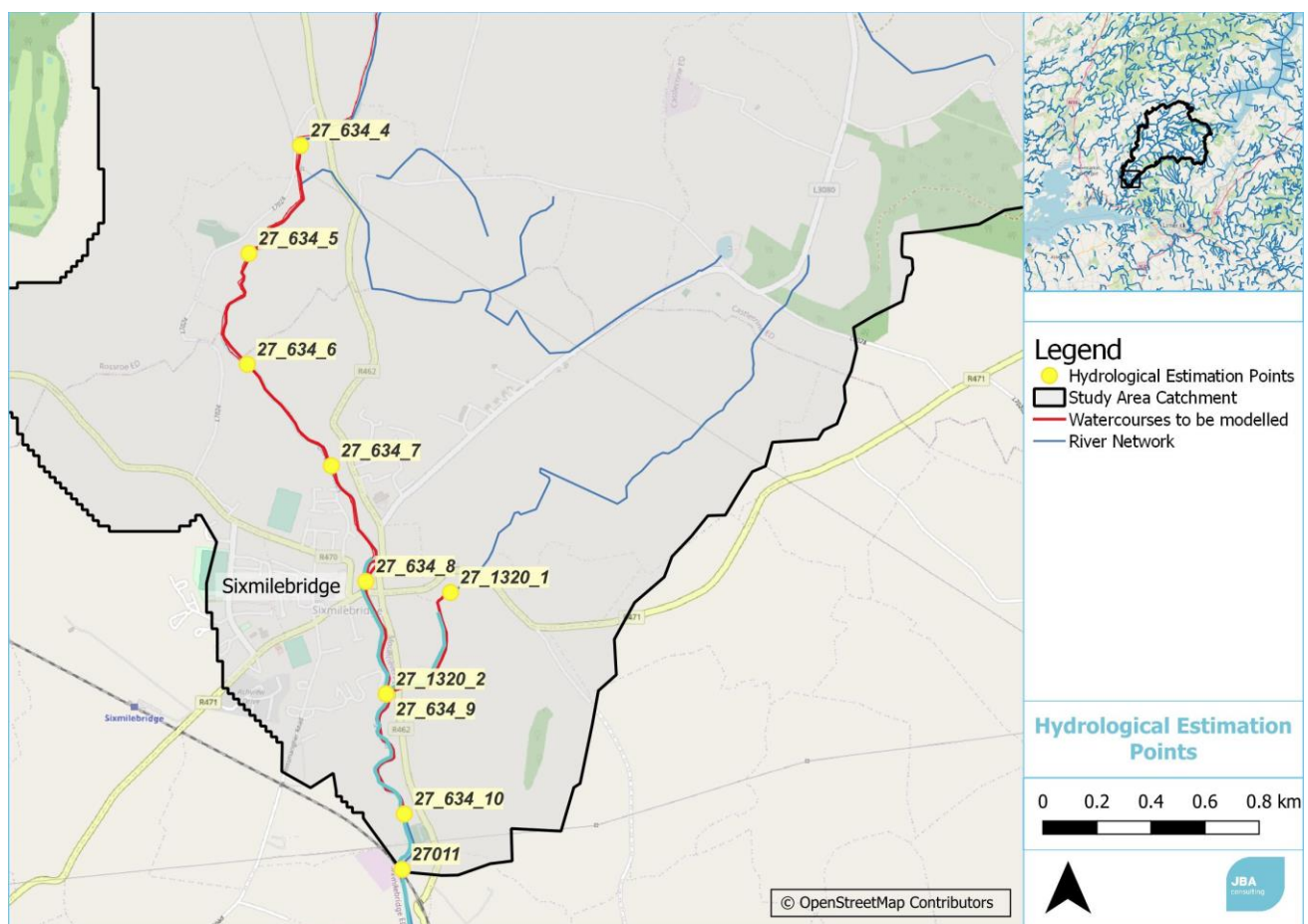


Figure 6-1: HEP Locations

Table 6-1: HEP Peak Flow Analysis

HEP ID	Annual Exceedance Probability (Return Period)							
	50% (2yr)		10% (10yr)		1% (100yr)		0.1% (1000yr)	
	HEP Peak flow	Model Peak flow	HEP Peak flow	Model Peak Flow	HEP Peak flow	Model Peak Flow	HEP Peak flow	Model Peak Flow
27_634_4	21.81	21.81	30.98	30.98	41.88	41.88	52.14	52.13
27_634_5	22.06	21.80	31.32	31.00	42.35	41.88	52.72	52.08
27_634_6	22.12	21.77	31.41	30.97	42.47	41.90	52.86	52.13
27_634_7	22.12	21.77	31.41	30.98	42.47	41.95	52.86	52.13
27_634_8	22.14	21.77	31.43	30.97	42.50	41.80	52.91	51.98
27_634_9	22.24	21.91	31.58	31.14	42.70	42.01	53.16	52.36
27_634_10	22.43	22.71	31.85	31.95	43.06	42.53	53.61	52.66
27011	22.70	23.05	32.23	32.33	43.58	42.79	54.25	52.87

6.2 Verification against historical events

6.2.1 November 2009 flood event

The model results have been verified against publicly available information gathered from the November 2009 flood event in Sixmilebridge. This was considered an appropriate event to verify the model outputs due to the availability of photographs of flooding extents and recorded water levels at the Owenogarney river gauge. The 2009 event is also likely to represent a higher probability flood event in Sixmilebridge.

The Owenogarney river gauge (27011) is located at the railway bridge crossing approximately 1.15km south of Sixmilebridge village. The gauge records both flow (m³/s) and water level (mAOD) values from the Owenogarney River. Gauge data for the Owenogarney River can be accessed from the OPW online records¹ which indicate that the mean water level during the November 2009 event was 4.24mAOD, with a maximum level of 4.27mAOD. A comparison with the model results indicates that this event corresponds

¹ OPW - waterlevel.ie

closest to the 10%AEP modelled event, which shows water level values of approximately 4.22mAOD at the gauging station location.

Figure 6-2 to Figure 6-3 show photographs taken of the November 2009 Sixmilebridge flooding. This indicates that water levels at the mill pond upstream of the weir structure did not overtop the wall onto the road, and that the wall did not breach during this event. Flooding did however occur downstream of the Sixmilebridge road bridge as flooding inundated the car park on the left bank and on the roads.



Figure 6-2: Weir Pond November 2009 Water Level



Figure 6-3: November 2009 Sixmilebridge Car Park Flooding



Figure 6-4: November 2009 Sixmilebridge Road Flooding



Figure 6-5: November 2009 Flooding - Road Junction



Figure 6-6: November 2009 Flooding - Aerial Photo

6.2.2 November 2009 flood extent analysis

Figure 6-7 shows the approximate flood extents for two confirmed flooded areas based on the available photographs of the November 2009 event compared to the baseline 10%AEP event. The 10%AEP event is considered the most appropriate to compare with the 2009 event with a 4.21mAOD maximum water level compared to the maximum 2009 4.27mAOD water level recorded at the Owenogarney river gauge.

Figure 6-7 also shows an additional modelled scenario which included channel bank walls at the mill pond and the channel section upstream of the road bridge which were previously removed from the model. Channel bank walls in Sixmilebridge were not included in the Baseline flood model as they are not formal flood defences and would be considered ineffective during a major flood event. Information gathered from the November 2009 event however indicates that channel bank walls were not breached, and therefore any analysis would benefit from a modelled scenario with the inclusion of channel walls to compare to the November 2009 event more accurately.

The results of this analysis show that the inclusion of the bank wall between the mill pond and the road shows a slight reduction in flooding in the car park resulting from the removed flow path originating from the mill pond. The inclusion of channel bank walls upstream of the road bridge also results in increased flooding on the left and right banks, resulting from increased channel overtopping immediately upstream of the channel walls.

The 10%AEP event with the inclusion of channel walls indicates that flood extents from a 10%AEP event closely match the November 2009 event. Although flooding extents in the car park appear slightly smaller compared to the 2009 event, this can be attributed to the water level being slightly lower in the modelled event. It could therefore be concluded that the November 2009 flooding is representative of an event slightly higher than a 10%AEP event and provides an evidence base for the validity of the model outputs.

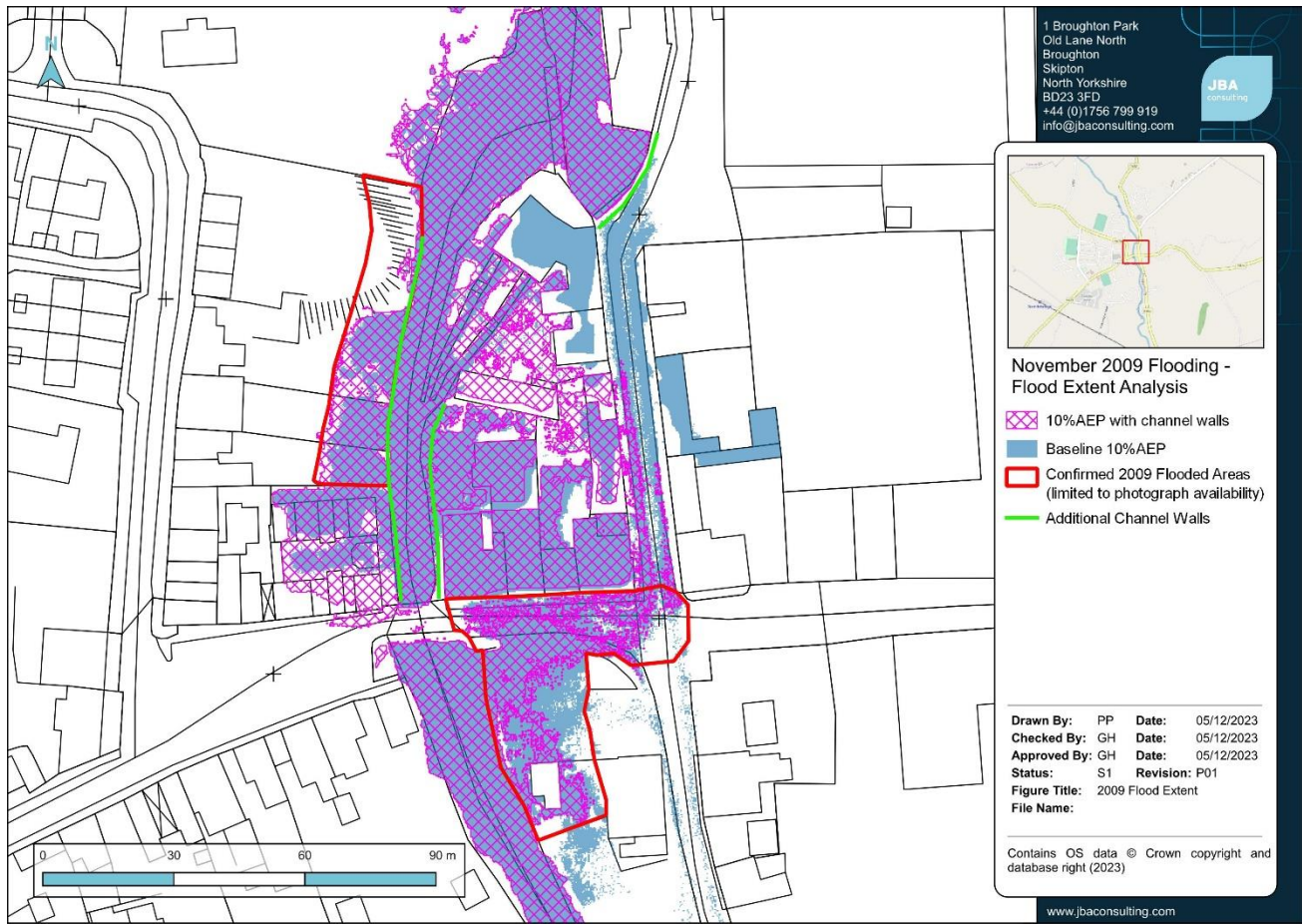


Figure 6-7: November 2009 flood Extent analysis

6.2.3 1995 flood event

Photographs of the 1995 Sixmilebridge flood event are available online, which are shown in Figure 6-8 and Figure 6-9. There are no available recorded water levels from the Owenogarney river gauge from the 1995 event, therefore this event cannot be directly calibrated to any specific modelled scenarios from his study. The photographs do provide an indication, however, of expected flooding areas during a major flood event. A comparison with the modelled 1%AEP event, shown in Figure 6-10, shows that the modelled extents closely match the flooded areas from the 1995 event, which provides an indication towards the validity of the predicted flood extents for larger modelled events.



Figure 6-8: 1995 Flood Event - Sixmilebridge Village



Figure 6-9: 1995 Flood Event - Downstream (Railway Crossing)

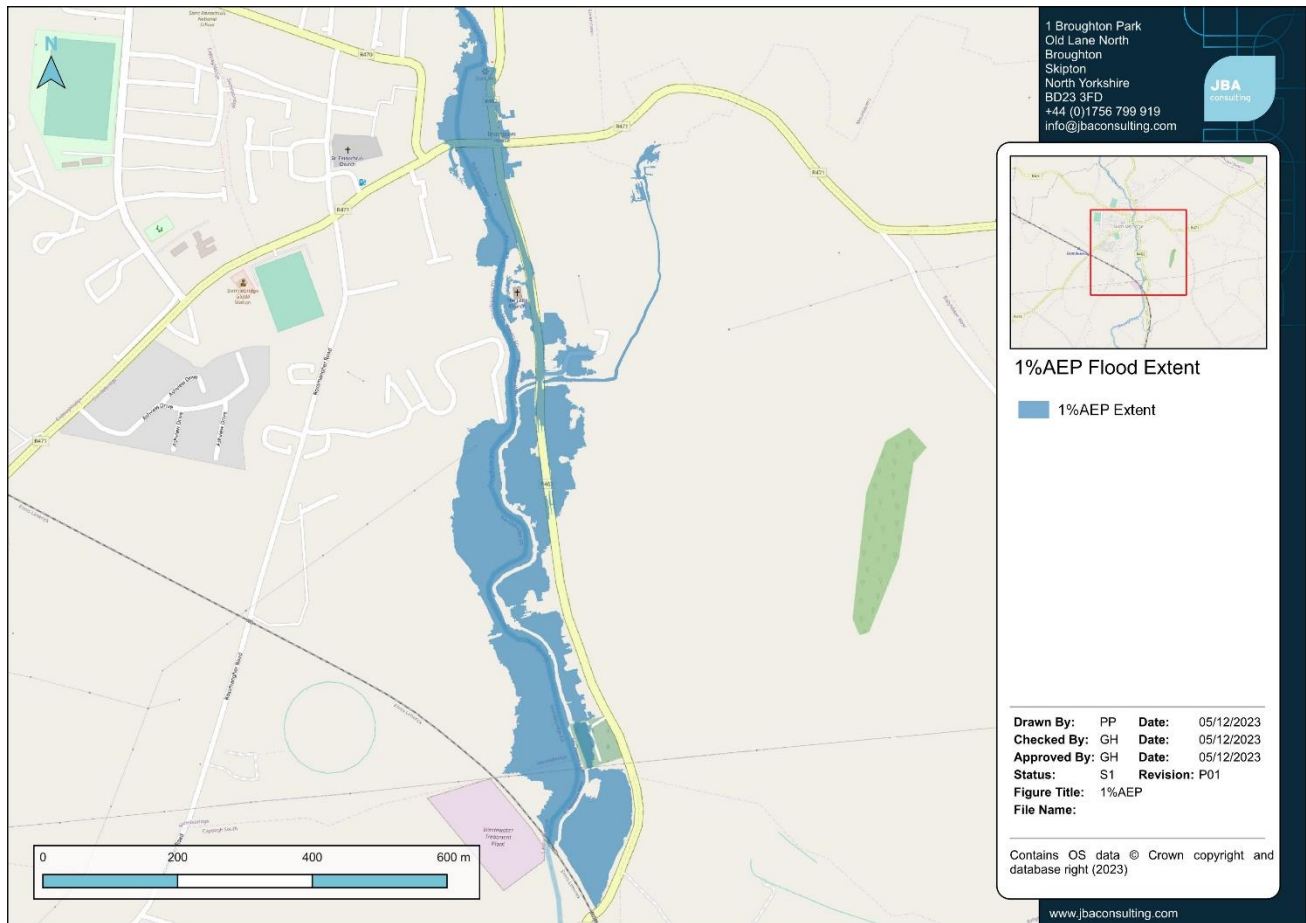


Figure 6-10: Modelled Baseline 1%AEP Extent

7 Model Sensitivity

Sensitivity tests are carried out on hydraulic models to assess the sensitivity of the model to changing hydraulic parameters. The results of a sensitivity analysis can give an indication of the confidence levels of the results generated from the model.

The following sensitivity tests were undertaken for this modelling study, which take into consideration the varying hydraulic characteristics of the Owenogarney catchment being modelled:

- Weir Coefficient sensitivity.
- 1D and 2D hydraulic roughness increased by 20%.
- 1D and 2D hydraulic roughness decreased by 20%.
- Sixmilebridge road bridge modelled with one arch.
- Sixmilebridge road bridge modelled with ineffective flow areas.
- 2D model utilising a 5m grid size for the entire 2D domain.
- Fluvial-tidal joint probability sensitivity analysis

Figure 7-1 shows the locations of a selection of model nodes taken from the baseline model. These nodes will be referred to in the remainder of this section for each sensitivity test (where appropriate) where water level values have been derived for analysis for each node. Water level values from each sensitivity scenario will be compared to the corresponding levels from the baseline 1%AEP to assess the sensitivity of in-channel water levels at different areas of the model.

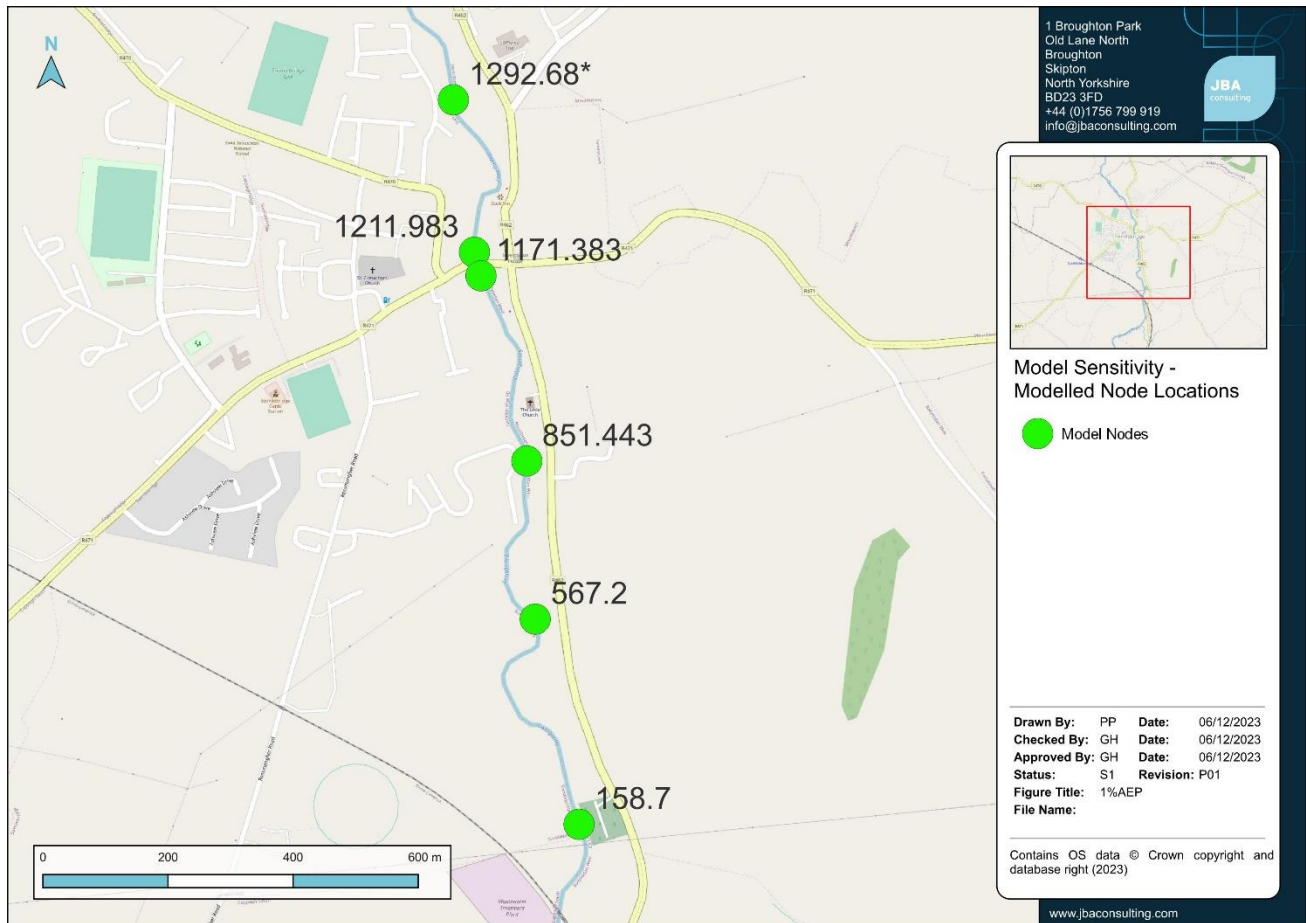


Figure 7-1: Model Sensitivity: Model Node Locations

7.1 Weir coefficient sensitivity

7.1.1 Context

Due to the complex nature of the hydraulic structure network in the vicinity of the old mill in Sixmilebridge, tests were undertaken to ensure that flow conveyance at the old mill weir 001 was accurately represented in the model. Initial testing of the model indicated that due to the weir's lateral position relative to the main channel, as well the main channel entering a storage area immediately upstream of the weir, that standardised weir coefficients generated unrealistic head losses at the weir structure.

A small 2D model was developed to represent hydraulic flow more accurately over the weir structure, the results from which could then be compared to the results of a suite of 1D weir coefficient tests in order to determine the optimum parameters to generate the most realistic results in the 1D weir model.

7.1.2 2D weir model build

A 2D only model was developed in the area of interest in order to evaluate the structure hydraulics compared to the 1D weir modelling approach. A 2D model approach can simulate flow over a defined structure in DTM data and can provide a more realistic

representation of its hydraulic characteristics when run independently from a complex 1D model network.

Figure 7-2 shows the configuration for the 2D weir model. The key model parameters are as follows:

- A 2D domain encompassing the area of interest with a 1m grid cell size for greater accuracy.
- An upstream boundary condition from which flow was applied to model using a flow hydrograph for a 1%AEP event.
- A normal depth boundary condition providing an outflow from the 2D model.
- A DTM modification on the weir structure using surveyed levels.
- Break lines applied over the weir structure and bankside to improve the efficiency of the model.
- A 2D connection line, allowing the implementation of culvert barrels to recreate the culvert outflow from the upstream storage area.

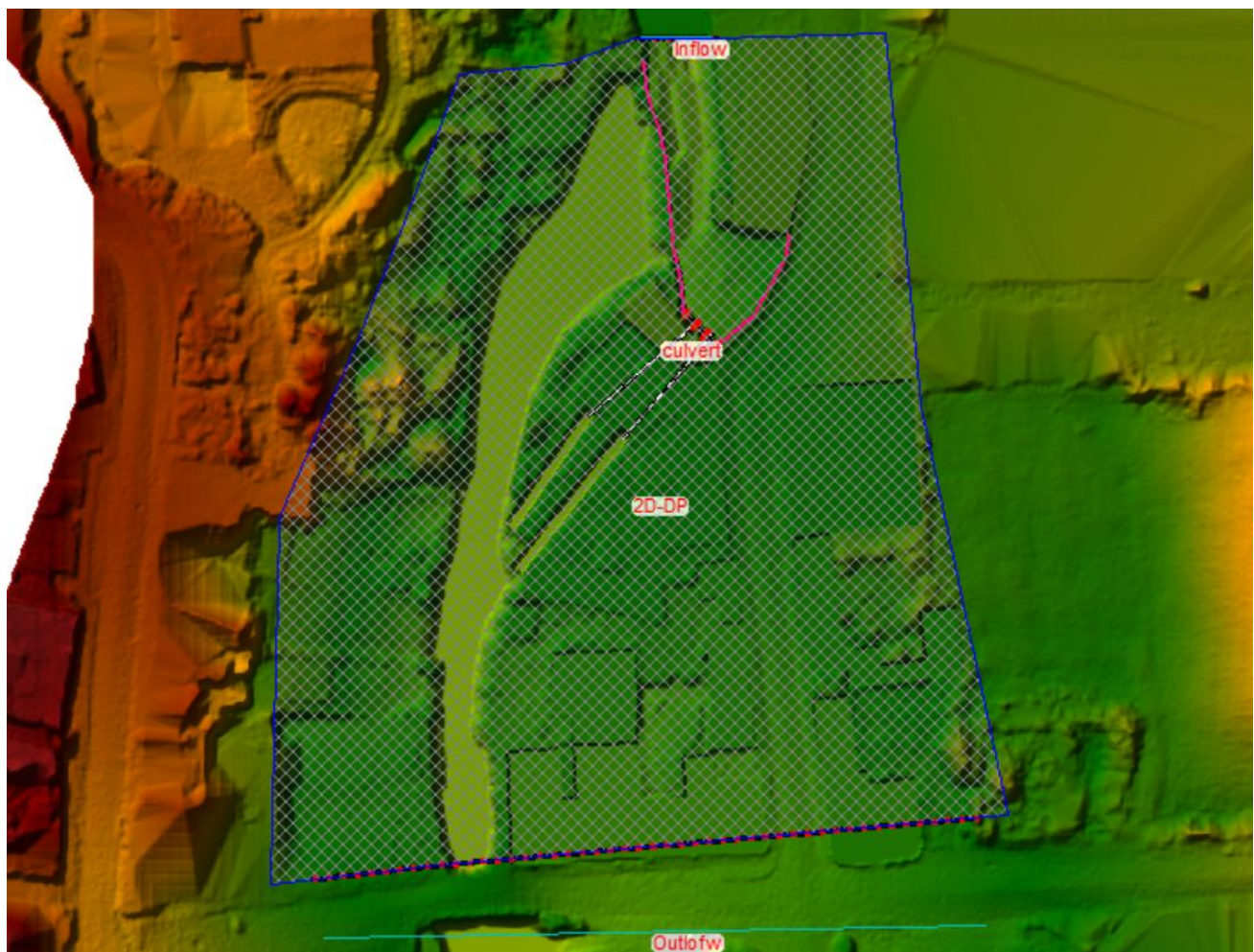


Figure 7-2: 2D Weir Model Configuration

7.1.3 1D weir coefficient testing

A number of 1D simulations were run, each utilising a varying weir coefficient value. A weir coefficient value is calculated from a range of factors to determine the efficiency in which hydraulic flow is conveyed over a structure. A typical weir coefficient in standard SI units may range from 1.4 - 1.7 depending on the physical characteristics of the structure, however initial testing of the model indicated that a standard weir coefficient was too low to represent flow over the weir structure in this particular model. This can be attributed to the fact that the weir operates as a lateral structure in relation to the main channel which enters a storage area immediately upstream of the weir.

Comparing the results of varied weir coefficients to the results of a 2D model simulation, however, can provide an evidence base for the use of an unorthodox coefficient value for this structure to attain the most accurate weir flow.

Weir coefficient values of 2.5, 3.5, 4 and 5 were tested using the baseline 1D model for the 1%AEP as part of this analysis.

7.1.4 Results analysis

A 43m profile line was drawn from an area upstream of the weir to downstream of the weir in order to assess varied water level profiles across a modelled simulation time of 39 hours. Water levels are not expected to be consistent between the 1D model results and 2D model results at the weir structure itself due to the inclusion of a storage area in the 1D model, which produces a flat level as opposed to the natural gradient shown in the 2D model results. The quality of each sensitivity simulation will therefore be assessed according to the likeness of the most upstream water levels to the 2D model results as well as the rate at which water levels rise at the weir structure itself compared to the 2D model results.

Figure 7-3 and Figure 7-4 show the water level profiles for all simulated results for the 0hr and 39hr simulation times respectively. This comparison allows for an analysis of water levels at the starting point of the simulation compared to increased flow values later into the simulation.

It should be noted that there is a discrepancy between the downstream 2D water level compared to the 1D modelled downstream water levels. This is because channel bed elevations downstream of the weir are not represented in the DTM, however channel bed DTM levels upstream of the weir and the weir structure itself are derived from surveyed channel bed levels. This test is designed to assess the weir structure as a hydraulic control and its impact on channel water levels immediately upstream, therefore the downstream conditions in this test are not considered impactful to the results and will not be considered in the results analysis.

The results indicate that a weir coefficient of 5 produces upstream water levels that are comparable to the 2D model results at the start of the modelled simulation. At the 39-hour simulation time however, flow values have increased significantly and the water levels of the 2D model simulation rise at a faster rate than those of the '5' and '4' weir coefficient simulations. At the 39hr simulation time upstream water levels from the '3.5' weir coefficient

simulation closely match those of the 2D simulation with a difference of approximately 2cm, whilst water levels at the weir structure itself are shown to raise at a similar rate to the 2D simulation. At the start of the simulation the '2.5' weir coefficient simulation shows water levels over 10cm higher than the 2D simulation, and this difference continues to increase by the 39hr simulation time.

It can therefore be concluded from this analysis that a standardised weir coefficient value between 1.4 and 1.7 was not appropriate for use in the hydraulic configuration of this weir. This analysis shows that a higher weir coefficient of 3.5 is more realistic hydraulically when compared to a 2D only simulation, and therefore has been adopted within the baseline model.

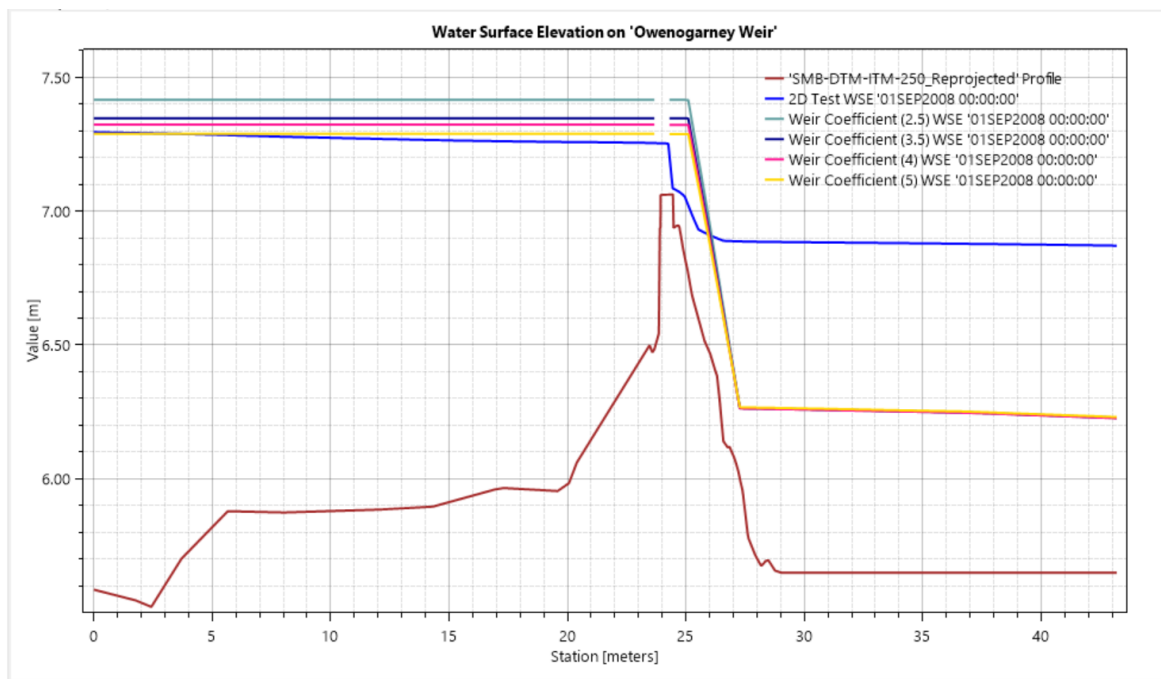


Figure 7-3: Weir Water Level Profiles: 0hrs

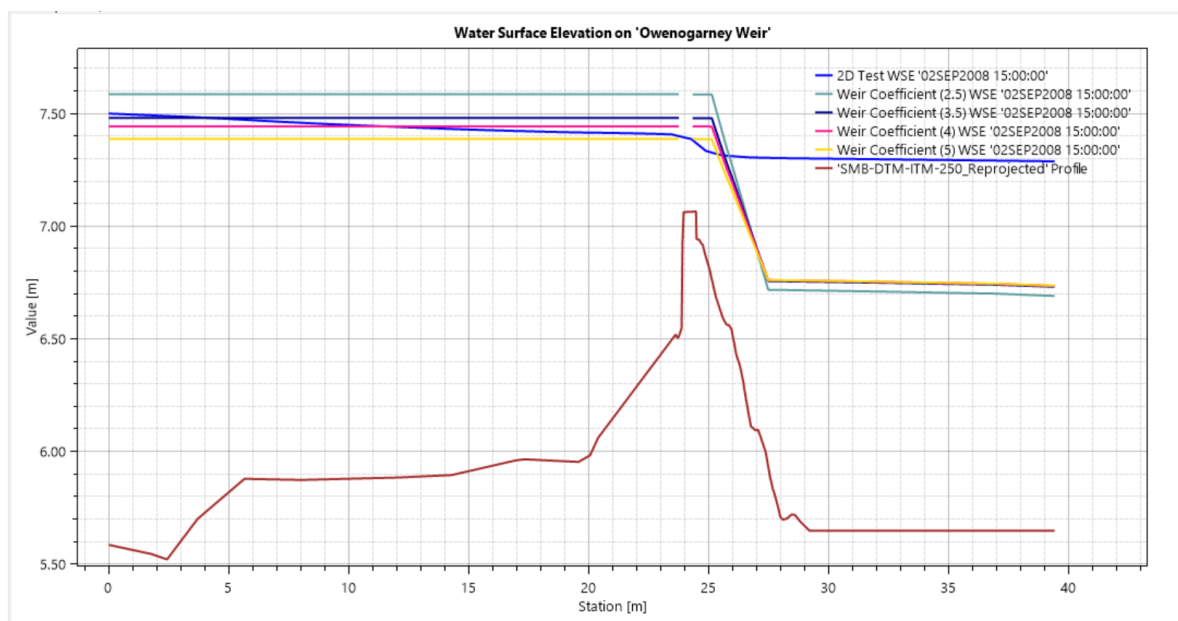


Figure 7-4: Weir Water Level Profiles: 39hrs

7.2 Hydraulic roughness+20%

A sensitivity test was carried out to assess the impacts of increasing channel and floodplain roughness values on model results. Roughness values were increased by 20% for the entirety of the model for both the 1D and 2D elements of the model. Table 7-1 shows the maximum 1D water level values for a selection of model nodes (Figure 7-1) for this sensitivity analysis.

Increasing roughness values shows a notable increase in water level values, particularly in areas further downstream of Sixmilebridge. Water level increases are generally shown to increase between 0.1m and 0.2m higher than baseline water levels across the model.

Figure 7-5 indicates that flood extents are likely to increase from the baseline results after increasing roughness values by 20%.

Table 7-1: Model Sensitivity - Roughness+20% Water Levels

Model Node	Baseline 1%AEP WL (mAOD)	1%AEP +20% roughness WL (mAOD)	Difference (m)
1492.68*	8.04	8.14	0.1
1211.983	7.16	7.16	0
1171.383	6.15	6.31	0.16
851.443	4.72	4.92	0.2
567.2	4.64	4.81	0.17
158.7	4.52	4.68	0.16

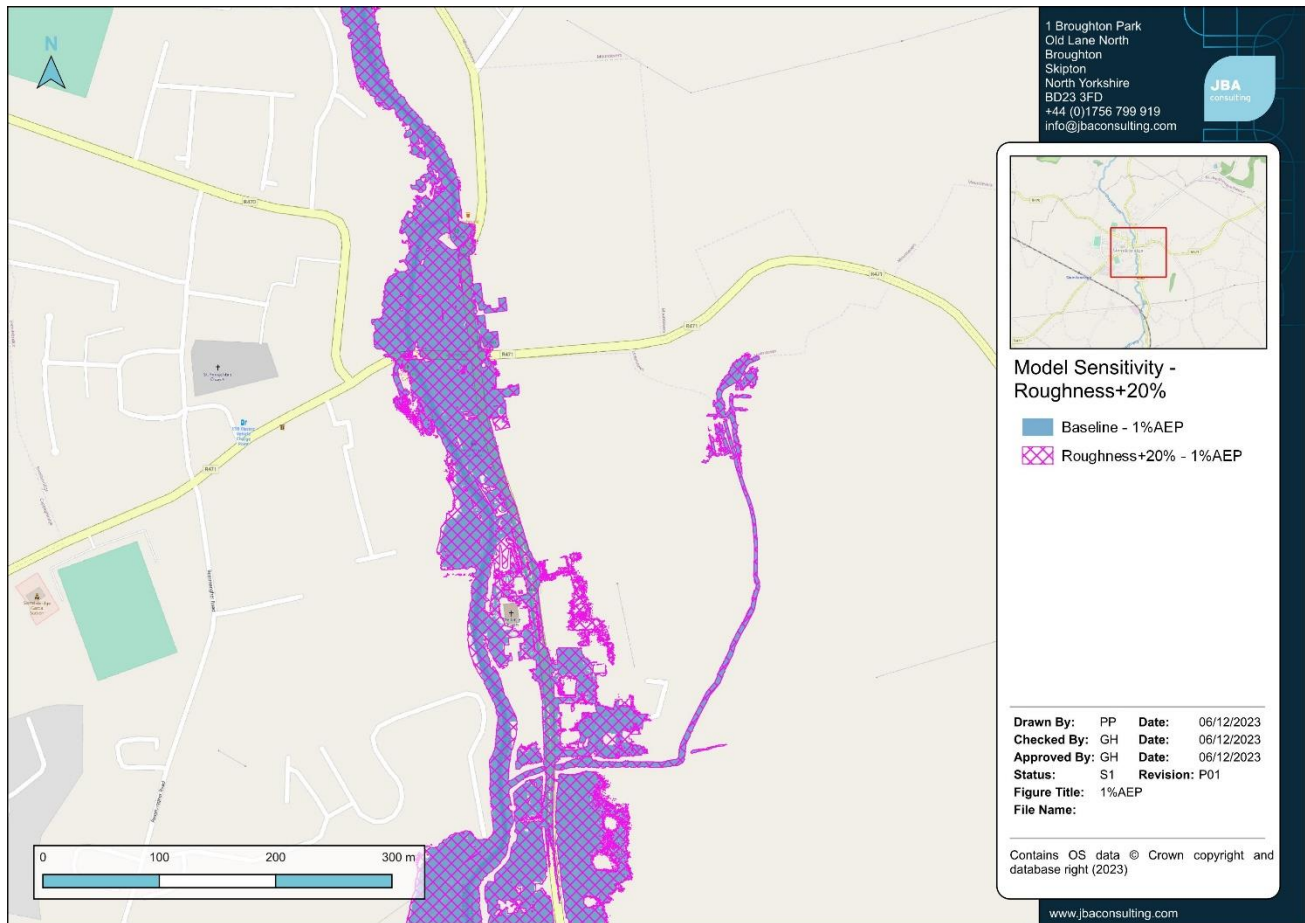


Figure 7-5: Model Sensitivity: Roughness+20% Flood Extent

7.3 Hydraulic roughness-20%

A sensitivity test was carried out to assess the impacts of decreasing channel and floodplain roughness values on model results. Roughness values were decreased by 20% for the entirety of the model for both the 1D and 2D elements of the model. Table 7-2 shows the maximum 1D water level values for a selection of model nodes (Figure 7-1) for this sensitivity analysis.

Decreasing roughness values shows a notable increase in water level values, particularly in areas further downstream of Sixmilebridge. Water levels are generally shown to decrease by between 0.11m and 0.23m from baseline water levels across the model.

Figure 7-6 indicates that flood extents are likely to decrease from the baseline results after decreasing roughness values by 20%.

Table 7-2: Model Sensitivity - Roughness -20% Water Levels

Model Node	Baseline 1%AEP WL (mAOD)	1%AEP -20% roughness WL (mAOD)	Difference (m)
1492.68*	8.04	7.93	-0.11
1211.983	7.16	7.17	0.01
1171.383	6.15	5.92	-0.23
851.443	4.72	4.51	-0.21
567.2	4.64	4.44	-0.2
158.7	4.52	4.34	-0.18

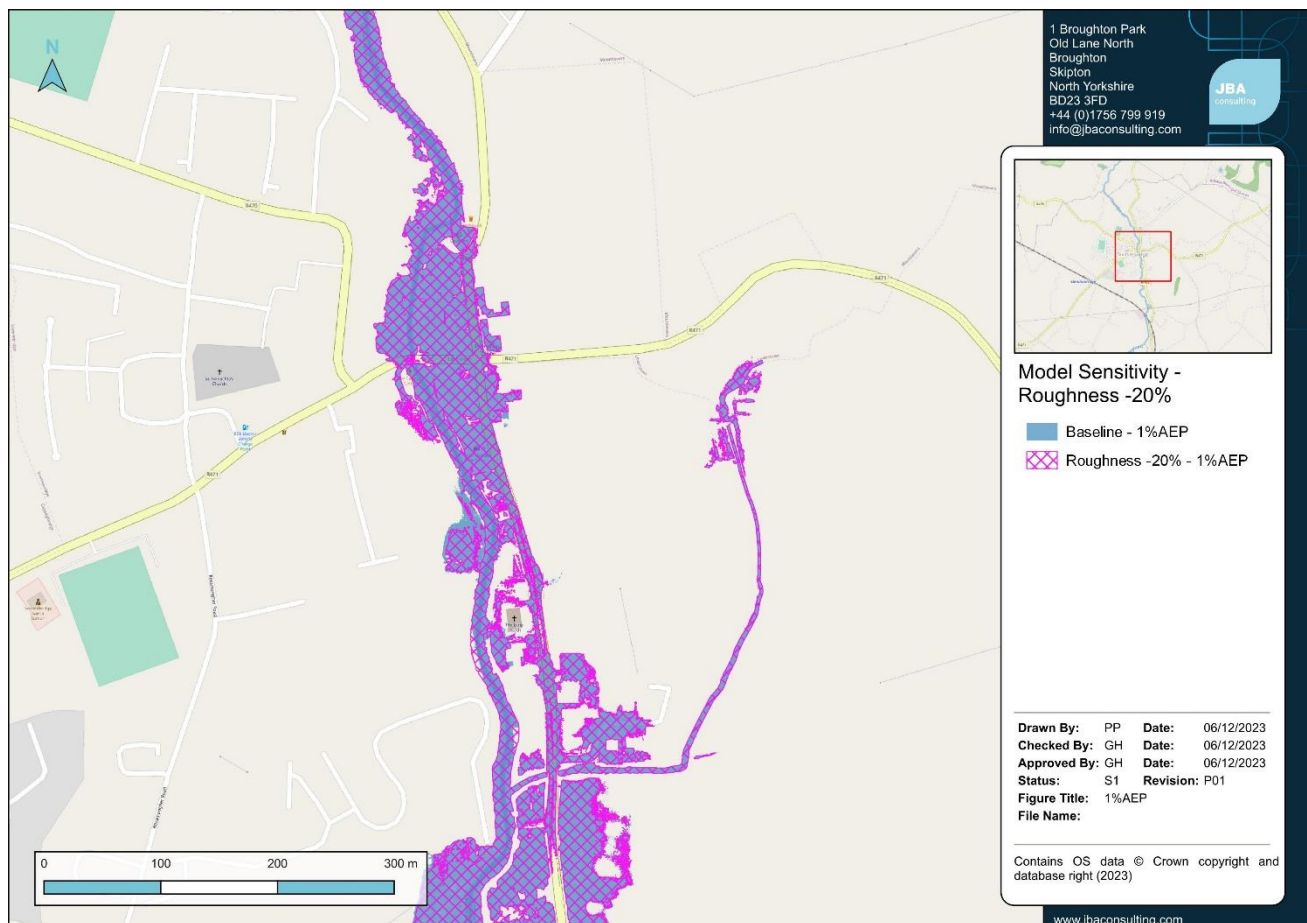


Figure 7-6: Model Sensitivity - Roughness -20% Flood Extent

7.4 Single bridge arch

A sensitivity test was carried out to assess the impact on modelled results from changing the structure dimensions of the main Sixmilebridge road bridge from three arches to a single arch for the 1%AEP event, which would reduce the amount of flow conveyance through the bridge. Table 7-3 displays maximum 1D water level values for a selection of model nodes (Figure 7-1) for this sensitivity analysis.

The results indicate that immediately upstream of the road bridge water levels show an expected rise by approximately 0.41m from the baseline scenario, however water levels further up and downstream show a very minor change (approximately 1cm).

Figure 7-7 shows a flood extent comparison between the baseline 1%AEP and the 1%AEP with a single bridge arch. This shows an increase in flood extent from the baseline simulation resulting from the increased overland flow resulting from increased out of bank flooding upstream of the bridge.

Table 7-3: Model Sensitivity - Single Bridge Arch Water Levels

Model Node	Baseline 1%AEP WL (mAOD)	1%AEP with single arch WL (mAOD)	Difference (m)
1492.68*	8.04	8.05	0.01
1211.983	7.16	7.57	0.41
1171.383	6.15	6.11	-0.04
851.443	4.72	4.71	-0.01
567.2	4.64	6.63	-0.01
158.7	4.52	4.51	-0.01

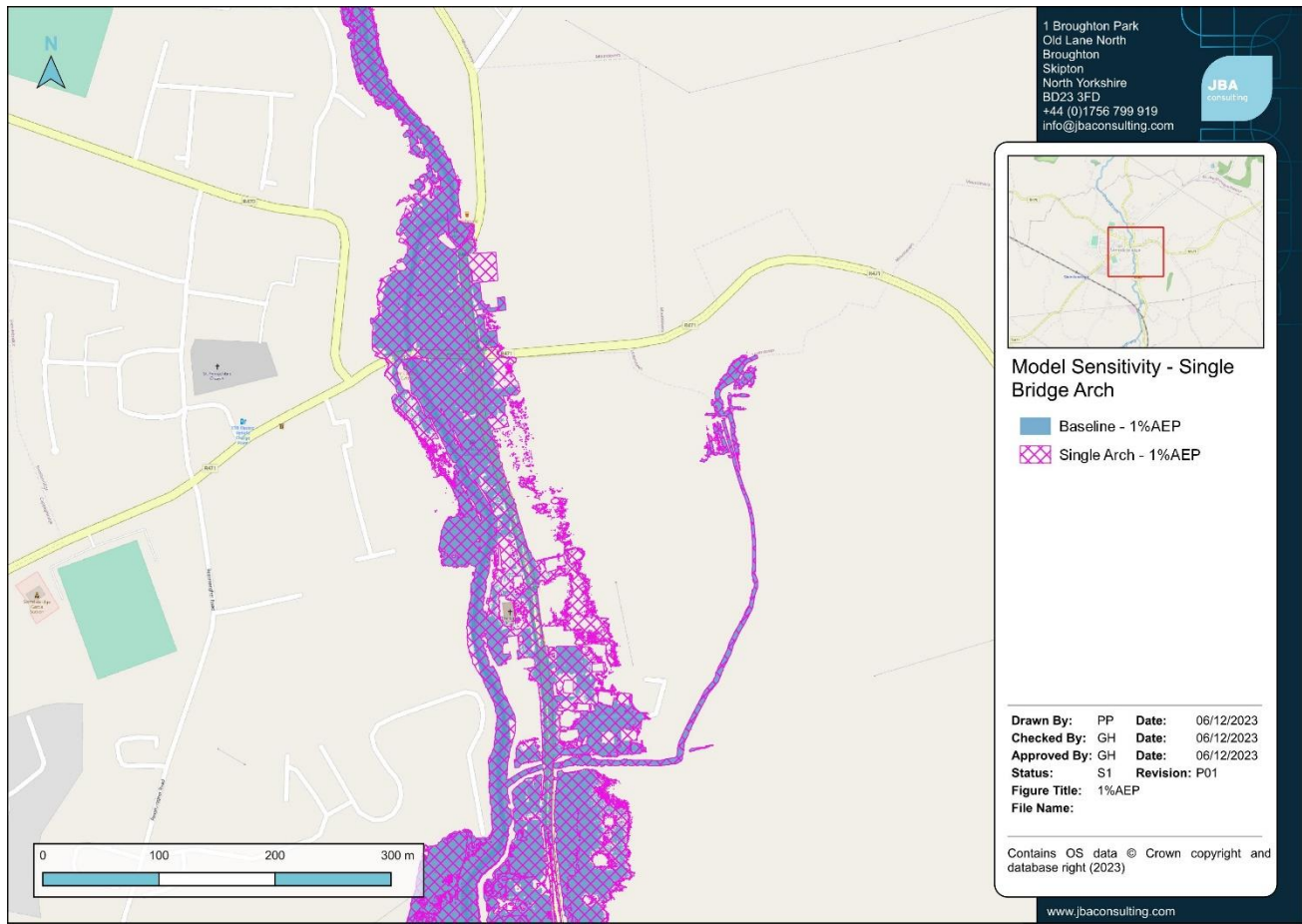


Figure 7-7: Model Sensitivity - Single Bridge Arch Flood Extent

7.5 Ineffective flow areas

Ineffective flow areas are often applied to portions of a cross section in which water will pond and the velocity of that water at the downstream section is close to zero. Ineffective flow areas have been applied to the Sixmilebridge road bridge to assess the effects of ineffective flow areas around a structure. Ineffective flow areas were applied at the central piers of the bridge at its upstream cross section. Table 7-4 shows the tabulated water level results from this analysis compared with those from the baseline 1%AEP event. This shows that the use of ineffective flow areas at the bridge structure had no impact on water level results.

Figure 7-8 shows a flood extent comparison of this analysis with the baseline 1%AEP event, which suggests that the use of ineffective flow areas has had no significant impact on flood extents.

Table 7-4: Model Sensitivity: Ineffective Flow Areas Water Levels

Model Node	Baseline 1%AEP WL (mAOD)	1%AEP with ineffective flow areas WL (mAOD)	Difference (m)
1492.68*	8.04	8.04	0
1211.983	7.16	7.16	0
1171.383	6.15	6.15	0
851.443	4.72	4.72	0
567.2	4.64	4.64	0
158.7	4.52	4.52	0

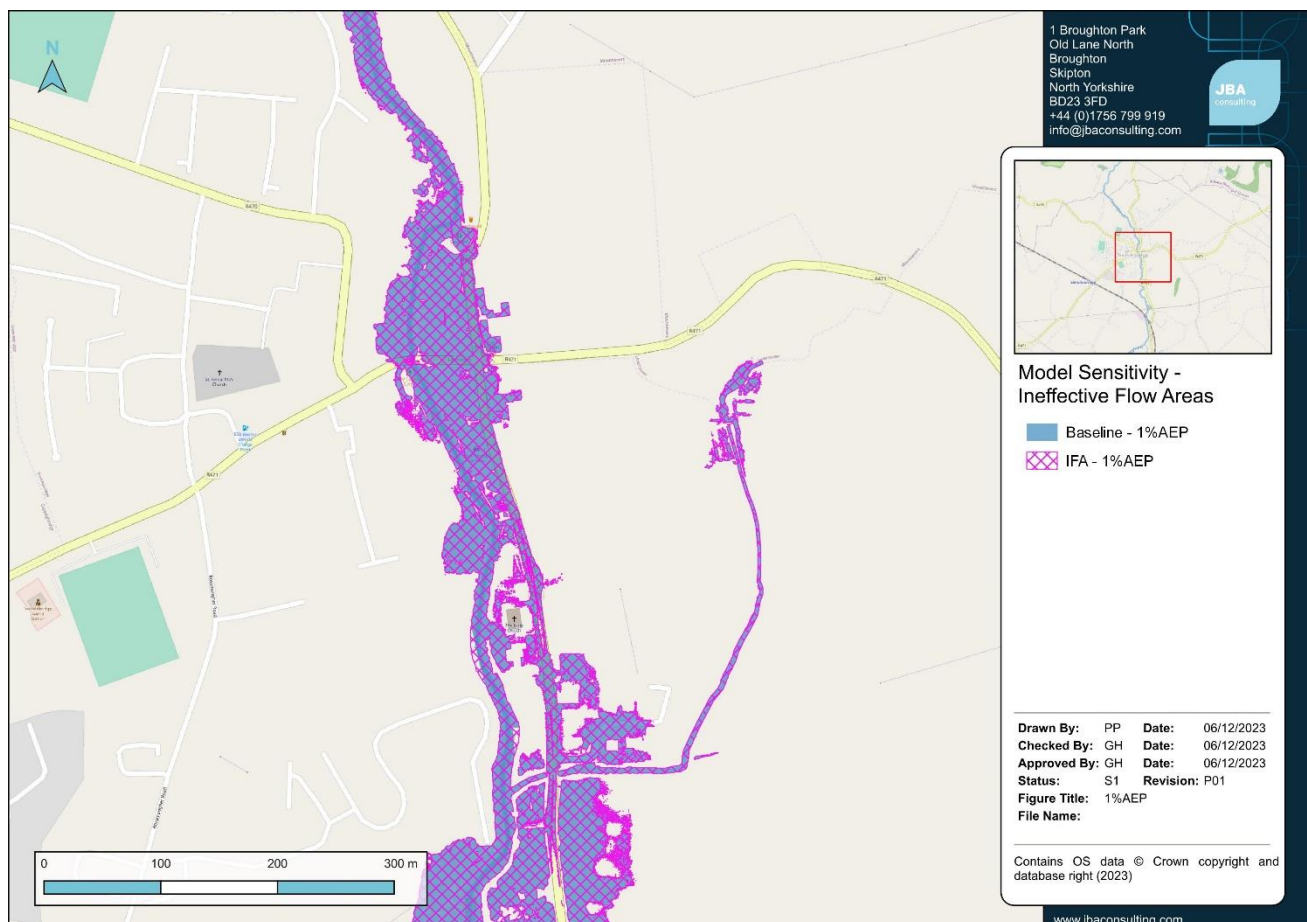


Figure 7-8: Model Sensitivity: Ineffective Flow Areas Flood Extent

7.6 2D flow area (5m grid cell size)

A sensitivity analysis was undertaken to assess the impact of using an increased 2D grid cell size in the developed areas of Sixmilebridge. The baseline model utilises a refinement region to reduce the cell size to 2m in developed areas for an increased level of detail, while a 5m cell size is used in more rural regions. This test will apply a 5m cell size across all 2D model areas for this analysis.

Table 7-5 shows the tabulated water level results from this testing, compared with the baseline 1%AEP event. This indicates that there is little to no effect on in channel water levels.

Figure 7-9 shows a flood extent comparison with the baseline 1%AEP event, which shows slight discrepancies between the results with both minor increases and decreases in extents visible throughout. It can be concluded however that there are no significant changes from the baseline results.

Table 7-5: Model Sensitivity: 2D Domain (5m) Water Levels

Model Node	Baseline 1%AEP WL (mAOD)	1%AEP with ineffective flow areas WL (mAOD)	Difference
1492.68*	8.04	8.04	0
1211.983	7.16	7.16	0
1171.383	6.15	6.15	0
851.443	4.72	4.72	0
567.2	4.64	4.64	0
158.7	4.52	4.51	-0.01

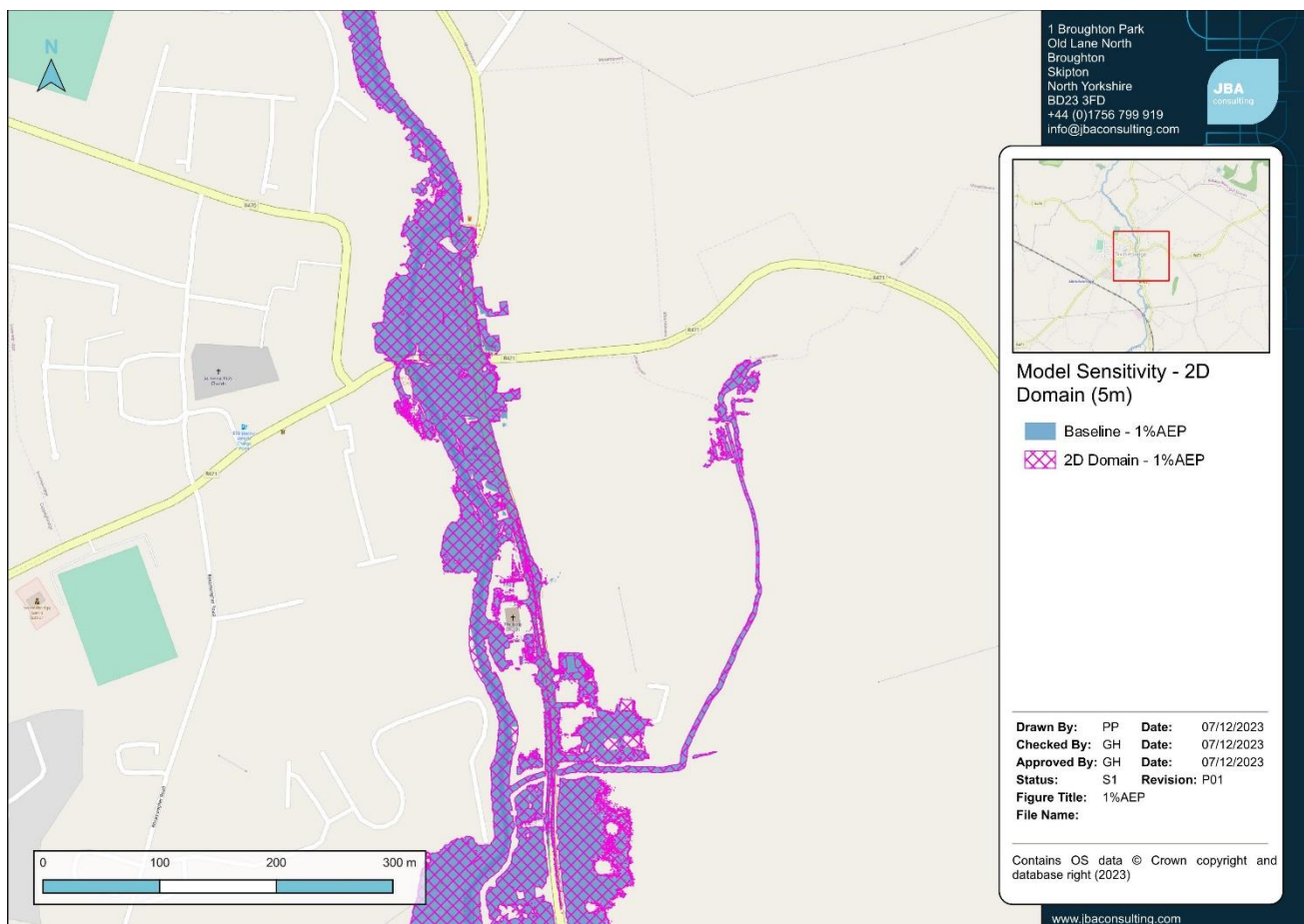


Figure 7-9: Model Sensitivity: 2D Domain (5m) Flood Extent

7.7 Fluvial-tidal joint probability analysis

7.7.1 Fluvial 10%AEP joint probability sensitivity

A sensitivity analysis was carried out to assess the model's sensitivity to increased downstream boundary levels, using the ICWWS extreme tidal level estimates for the 10%AEP, 1%AEP and 0.5%AEP tidal levels combined with a 10%AEP fluvial event.

Table 7-6 shows the modelled water levels for each modelled joint probability scenario compared with the baseline 10%AEP. It should be noted that the baseline event is modelled with a 50%AEP tidal downstream boundary. The results indicate that increased tidal levels at the downstream boundary show little impact to water levels in the upstream areas of Sixmilebridge, however levels show a significant increase further downstream.

Figure 7-10 shows a flood extent comparison for each modelled event in this analysis. This indicates that there is no significant increase in flood extent in central Sixmilebridge, with some localised increases in the southern region of the village. Notable extent increases are shown on the left and right bank floodplains immediately upstream of the railway crossing.

Table 7-6: Model Sensitivity: 10%AEP Joint Probability Analysis Water Levels

Model Node	Baseline 10%AEP WL (mAOD)	10%AEP(F) 10%AEP(T) WL (mAOD)	10%AEP(F) 1%AEP(T) WL (mAOD)	10%AEP(F) 0.5%AEP(T) WL (mAOD)
1492.68*	7.84	7.84	7.84	7.84
1211.983	6.81	6.81	6.81	6.81
1171.383	5.91	5.91	5.91	5.91
851.443	4.46	4.56	4.71	4.76
567.2	4.36	4.49	4.68	4.73
158.7	4.26	4.42	4.62	4.68

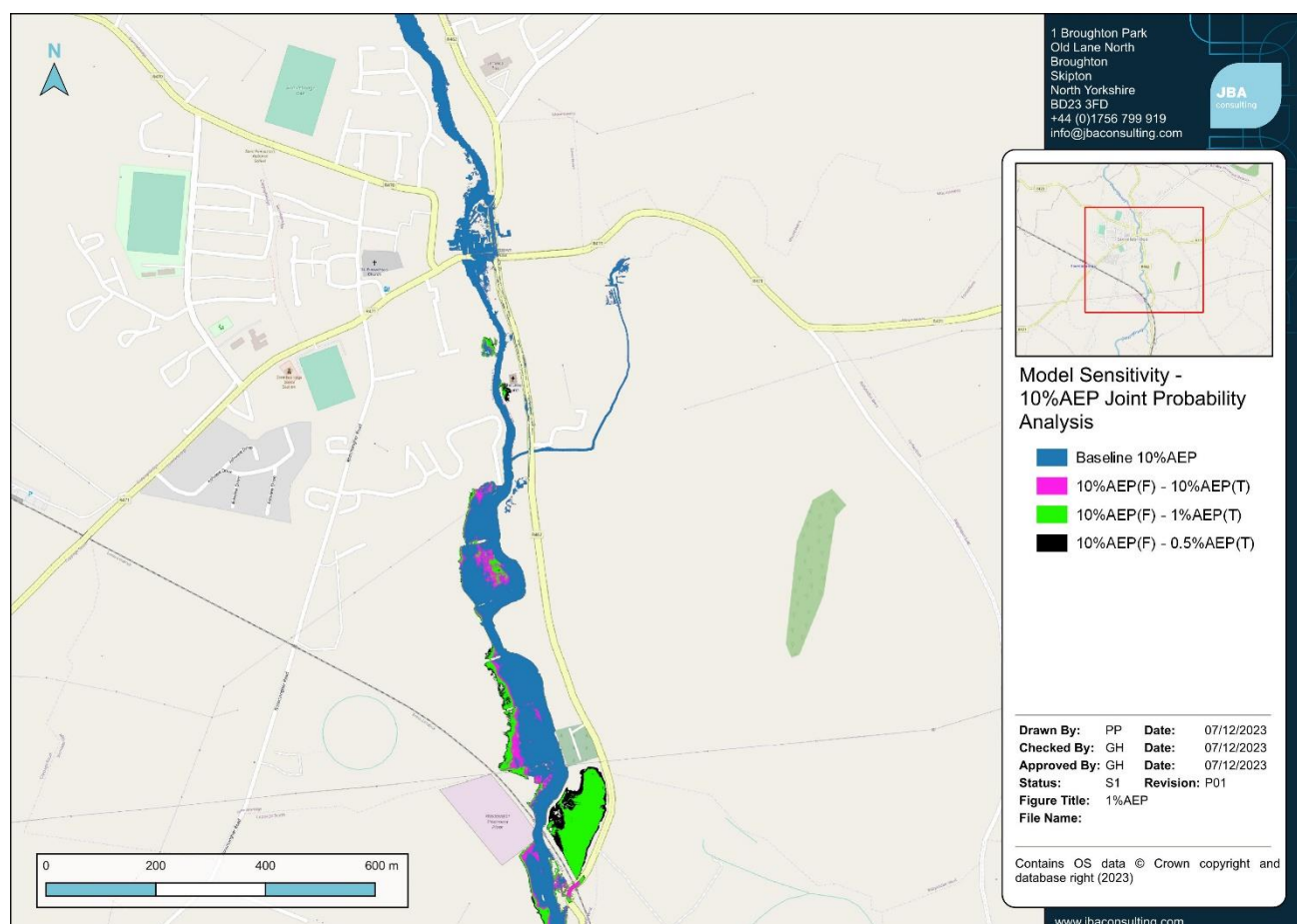


Figure 7-10: Model Sensitivity: 10%AEP Joint Probability Analysis Flood Extents

7.7.2 Fluvial 1%AEP joint probability sensitivity

A sensitivity analysis was carried out to assess the model's sensitivity to increased downstream boundary levels, using the ICWWS extreme tidal level estimates for the 10%AEP tidal event combined with a 1%AEP fluvial event.

Table 7-7 shows the modelled water levels for the increased joint probability scenario compared with the baseline 1%AEP. It should be noted that the baseline event is modelled with a 50%AEP tidal downstream boundary. The results indicate that increased tidal levels at the downstream boundary show little impact to water levels in the upstream areas of Sixmilebridge, however levels show an increase further downstream. Compared to the fluvial 10%AEP joint probability analysis, the differences from the baseline levels are significantly lower indicating that increased tidal influences have a lesser impact on higher fluvial events.

Figure 7-11 shows a flood extent comparison with the baseline 1%AEP event. There is a slight increase in flood extent in the increased joint probability scenario, mainly in areas downstream of Sixmilebridge. there is no significant increase in flood extent in central Sixmilebridge, with some localised increases and decreases in the southern region of the village.

Table 7-7: Model Sensitivity: 1%AEP Joint Probability Analysis Water Levels

Model Node	Baseline 1%AEP WL (mAOD)	1%AEP(F) 10%AEP(T) WL (mAOD)	Difference (m)
1492.68*	8.04	8.04	0
1211.983	7.16	7.16	0
1171.383	6.15	6.15	0
851.443	4.72	4.78	0.06
567.2	4.64	4.72	0.08
158.7	4.52	4.61	0.09

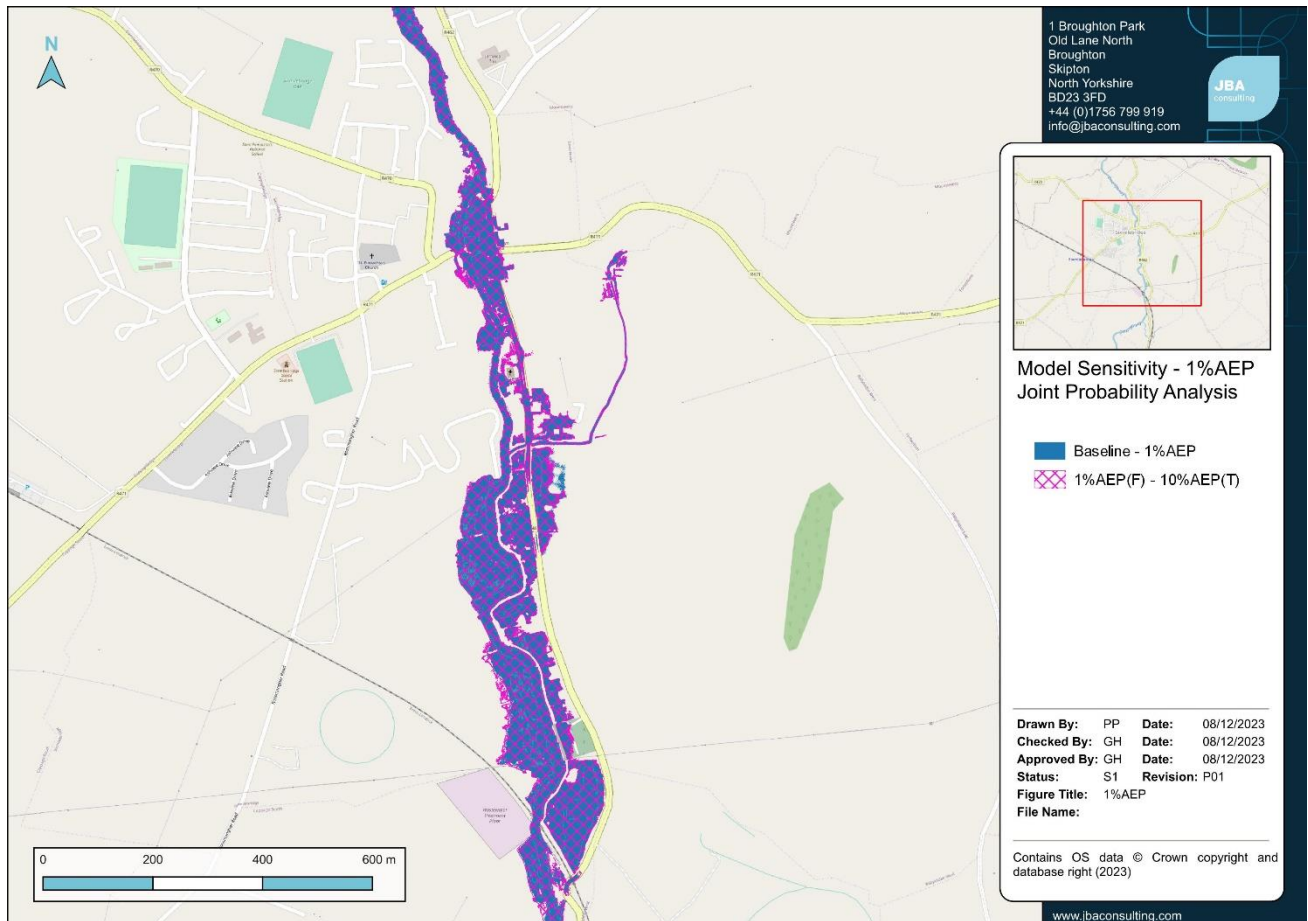


Figure 7-11: Model Sensitivity - 1%AEP Joint Probability Analysis Flood Extents

8 Key Model Assumptions and Model Limitations

8.1 Tidal boundaries

The model assumes a conservative approach to the joint probability of fluvial and tidal influences to allow for a worst-case scenario. This was achieved by timing the fluvial inflow hydrographs and tidal stage hydrographs for maximum levels to coincide. Tidal curves were generated using estimated extreme levels from the ICWWS point S25 in the Shannon estuary. This is the most local estimation point to the study area.

Tidal levels were initially derived from official ICWWS extreme sea level estimates, however calibration with recorded river gauge levels indicated that tidal level influences were too low. Revised ICWWS estimated extreme levels were there subsequently adopted for this study, originating from coastal flood modelling outputs from the Limerick Flood Relief Scheme (FRS) which recorded higher sea level estimates. Although the revised ICWWS estimates are unofficial, these were considered the most appropriate by the OPW for this modelling study and therefore adopted over official estimates.

8.2 Structures

8.2.1 Road bridge

The Sixmilebridge road bridge was simplified due to the complexity of the structure. The bridge structure comprises three arched culverts, two of which are winged culverts that are partially skewed from the channel at the upstream face before realigning to run parallel to the channel direction. The complex nature of the structure meant that it was impossible to acquire detailed internal survey dimensions of the two wing culverts.

With information gained from visual inspections on site and site photographs, it is assumed that all three culverts are uniform to the dimensions recorded for the central culvert. The culvert openings of the wing culverts were narrowed in the hydraulic model to account for the energy loss from the skewed nature of the culvert openings, however it is acknowledged that this is a simplified representation of the bridge structure dimensions.

8.2.2 Old mill weir

The sluice gate adjacent to the old mill weir 001 is assumed to be closed and acts as a weir structure in conjunction with the formal weir.

An unorthodox approach to the weir modelling parameters has been taken, where a weir coefficient value of 3.5 (SI units) was applied. This is due to the complex 1D network in this area meaning that the weir did not realistically represent flow using standardised weir coefficient values. Further justification for this decision is detailed in Section 7.1.

8.2.3 Channel bank walls

Channel bank walls were removed from the model on the assumption that they would not provide adequate protection in the event of a major flood. This may result in an overpredicting of out of bank flooding for smaller events where walls may provide a level of protection. Figure 8-1, Figure 8-2, and Figure 8-3 show photographs of walls which were excluded from the model.



Figure 8-1: Wall removal - Mill pond roadside wall



Figure 8-2: Wall removal - channel bank walls upstream of the road bridge



Figure 8-3: Wall removal - Left bank wall adjacent to the car park

8.3 Survey data

Surveyed cross section levels from the CFRAM survey at cross section 1323.036 are assumed to be incorrect, due to being higher in some areas than the surveyed levels of the adjacent weir. The cross section is located immediately upstream of the Sixmilebridge weir structure, and it is assumed that this is due to have occurred in error during data processing. Through a comparison with neighbouring cross section data, the recorded levels are assumed to be 1m higher than actual levels and have been lowered by 1m in the model cross section.

Surveyed levels from the CFRAM survey at cross section 1318.036 included an unusually low channel bed value of 3.64mAOD, approximately 2m lower than neighbouring levels. This was assumed to be incorrect when also compared to neighbouring cross sections, and the cross section was adjusted to avoid potential model instabilities resulting from abrupt changes in channel bed elevation. This cross section is located immediately downstream of the Sixmilebridge weir structure. Its close proximity to the aforementioned erroneous cross section compounded the suspicions that this cross section may have been recorded incorrectly.

8.4 DTM data quality

A key area of channel bank overtopping is from the old mill storage area on the adjacent road where a channel wall has been removed. Initial model testing indicated that this area flooded very early in the model simulation, which was due to DTM elevation in this area being too low. This is likely due to poor DTM filtering, and was verified against available cross section surveys at the storage area which confirmed that DTM elevation levels at the roadside are approximately 20cm lower than surveyed levels. A DTM modification was made in this area to be more representative of surveyed levels.

9 Model File Naming

The hydraulic model files are provided as a separate deliverable to this document. The model file naming convention for each model scenario and event file are documented in the following sections.

9.1 HEC-RAS file naming

Below is a comprehensive list of HEC-RAS file types and their definitions:

- **.prj** is the Project file. Contains current plan files, units and project description.
- **.g01** is the Geometry file. Cross-sectional data, hydraulic structures and modelling approach data are stored here.
- **.f01** is the Steady Flow file. Profile information, flow data and boundary conditions written in this file.
 - For Unsteady Flow, **.u01** is the flow file extension. This is where hydrographs and initial conditions are stored, as well as any user-defined flow options.
 - For Quasi-Unsteady Flow (for a sediment analysis), **.q01** is the flow file extension.
- **.p01** is the Plan file. Contains a list of the associated input files, and all simulation options.
- **.001** is the Output file. Contains all of the computed results from the associated plan. This file is written in binary format and can only be read from the user interface.
 - For Unsteady Flow, a **.dss** file is automatically generated as an output. This file contains time series data that is viewable by any program that can read dss files (typically HEC software).
- **.r01** is the Run file for steady flow analysis. Contains all of the necessary input data required for the RAS computational engine. The run file is created during the model simulation and is not required to view final results.
 - For Unsteady Flow, **.x01** is the extension.
- **.comp_msgs.txt** is the Computational Message text file. Records the computational messages that pop up in the computation window. The messages file is not required to view final results but can be useful in troubleshooting errors identified by RAS.
- **.hyd01** is the Detailed Computational Level output file. This can be switched on in the Unsteady Flow Analysis window.
- **.p01.rst** is a Restart File (also called a Hot Start File, or Initial Conditions File). This option can be switched on by the user in the Output Control Options window.

- **.c01** is the Geometric Pre-Processor output file. Contains the hydraulic properties tables, rating curves, and family of rating curves for each cross-section, bridge, culvert, storage area, inline and lateral structure.
- **.b01** is the Boundary Condition file.
- **.bco01** is the Unsteady Flow Log output file.
- **.p01.blf** is the Binary Log file.
- **.IC.O01** is the Initial Conditions file.

9.2 Sixmilebridge model file names

9.2.1 File naming convention

The file codes below are used to define model files by scenario and event type, which are named according to the following naming structure: [model] _ [scenario] _ [event].

Model Codes:

- **SMB** - Sixmilebridge model

Scenario Codes:

- **Current** - Present day baseline scenario
- **MRFS** - Mid Range Future Scenario (climate change)
- **HEFS** - High End Future Scenario (climate change)
- **SENS** - Sensitivity Scenario
- **JP** - Joint Probability Scenario

Event Codes:

- **50%AEP** - 1 in 2-year/ 50% annual probability
- **20%AEP** - 1 in 5-year/ 20% annual probability
- **10%AEP** - 1 in 10-year/ 10% annual probability
- **5%AEP** - 1 in 20-year/ 20% annual probability
- **2%AEP** - 1 in 50-year/ 2% annual probability
- **1%AEP** - 1 in 100-year/ 1% annual probability
- **0.5%AEP** - 1 in 200-year/ 0.5% annual probability
- **0.1%AEP** - 1 in 1000-year/ 0.1% annual probability

9.2.2 Geometry files

- **SMB_Geometry_Baseline (.g15)** - Used to run all 'Current', 'MRFS', 'HEFS' and 'JP' scenarios.
- **SMB_Geometry_SENS_roughness+20% (.g01)** - Used to run the +20% roughness sensitivity test.
- **SMB_Geometry_SENS_roughness-20% (.g02)** - Used to run the -20% roughness sensitivity test.

- **SMB_Geometry_SENS_bridge_single_arch (.g03)** - Used to run the single arch sensitivity test.
- **SMB_Geometry_SENS_bridge_IFA (.g06)** - Used to run the Ineffective Flow Area bridge sensitivity test.
- **SMB_Geometry_SENS_2D_domain_5m_only (.g07)** - Used to run the 2D domain sensitivity test.
- **SMB_Geometry_SENS_2D_with_walls (.g14)** - Used to run the 1%AEP with channel walls included.
- **SMB_Geometry_SENS_2D_weir (.g08)** - Used to run the 2D only model to assess the old mill 001 weir hydraulics.
- **SMB_Geometry_SENS_weircoefftest_2.5 (.g09)** - Used to test a weir coefficient of 2.5 for the old mill weir 001.
- **SMB_Geometry_SENS_weircoefftest_3.5 (.g10)** - Used to test a weir coefficient of 3.5 for the old mill weir 001.
- **SMB_Geometry_SENS_weircoefftest_4 (.g11)** - Used to test a weir coefficient of 4 for the old mill weir 001.
- **SMB_Geometry_SENS_weircoefftest_5 (.g12)** - Used to test a weir coefficient of 5 for the old mill weir 001.

A Hydrology Analysis Report

Sixmilebridge Flood Map Update

Hydrology Report

Final Report

September 23

2023s0353

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Contract

This report describes work commissioned by Office of Public Works (OPW), by a signed contract dated 10/03/2023. The OPW's representative for the contract was Kevin Macken of the OPW. Anastasiya Ilyasova of JBA Consulting carried out this work.

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Purpose

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Abbreviations

1D	One Dimensional (modelling)
2D	Two Dimensional (modelling)
AEP	Annual Exceedance Probability
CFRAM	Catchment Flood Risk Assessment and Management
DoEHLG.....	Department of the Environment, Heritage and Local Government
FARL.....	FEH index of flood attenuation due to reservoirs and lakes
FB	Freeboard
FEH.....	Flood Estimation Handbook
FFL.....	Finish Floor Levels
FRA.....	Flood Risk Assessment
FSR.....	Flood Studies Report
FSU.....	Flood Studies Update
GSI.....	Geological Survey of Ireland
ICWWS	Irish Coastal Wave and Water Level modelling Study
OPW	Office of Public Works
PFRA	Preliminary Flood Risk Assessment
RR.....	Rainfall-Runoff
SAAR	Standard Average Annual Rainfall (mm)
SFRA	Strategic Flood Risk Assessment
SSFRA.....	Site Specific Flood Risk Assessment
URBEXT	FEH index of fractional urban extent
WL.....	Water Level

1 Introduction

JBA Consulting was commissioned by Office of Public Works (OPW) to update the flood maps prepared under the Shannon CFRAMS (Shannon Catchment Flood Risk and Management Study) for the Sixmilebridge AFA (Area of Further Assessment) in Co. Clare. The work is a result of change in the hydrology assessment for Owenagarney River as part of the SSFRA (for 18/594) on behalf of Clare County Council in 2019. The SSFRA has identified an unnamed tributary of Owenagarney River discharging downstream of Sixmilebridge that has not been modelled under the Shannon CFRAMS. There is flood history on record on this tributary, as such it is necessary to re-do the Shannon CFRAM hydrology for Sixmilebridge.

As part of the Flood Map Update, a hydraulic model with corresponding hydrological inputs is to be developed for Sixmilebridge and the unnamed tributary with hazard maps created for both. This Hydrology Report aims to provide an overview of the available hydrological data for the area and the proposed methodology for deriving flows to be used in the hydraulic model.

1.1 Catchment (Study Area) Overview

The main hydrological feature in Sixmilebridge is the Owenagarney River. The river rises in Moylussa hills, with highest point 531.6mOD. The catchment stretches to Rogmount Hill and is wider in the upper reach with more tributaries feeding the Owenagarney River upstream. The downstream part of the study catchment is narrower. The total area of the Owenagarney River study catchment downstream of Sixmilebridge is 162km². The river then continues for another 9km before discharging to the Shannon Estuary. Castle Lake is upstream of the study area on the Owenagarney River and may attenuate flood discharge. The unnamed tributary that has been identified as part of the SSFRA (for 18/594) east from the Owenagarney River on the left bank has a catchment area of 1.67km² and discharges just downstream of the Sixmilebridge. The tributary catchment is narrow and steep with the highest point 85mOD. The study area catchment with watercourses is presented on Figure 1-1.

The study area is predominantly rural, the urban area is only associated with the Sixmilebridge. The upstream part and the south edge of the catchment consists of various forest types, peat bogs, moors, and woodland. A number of small waterbodies are within the catchment. Refer to Figure 1-2 for the land use classification.

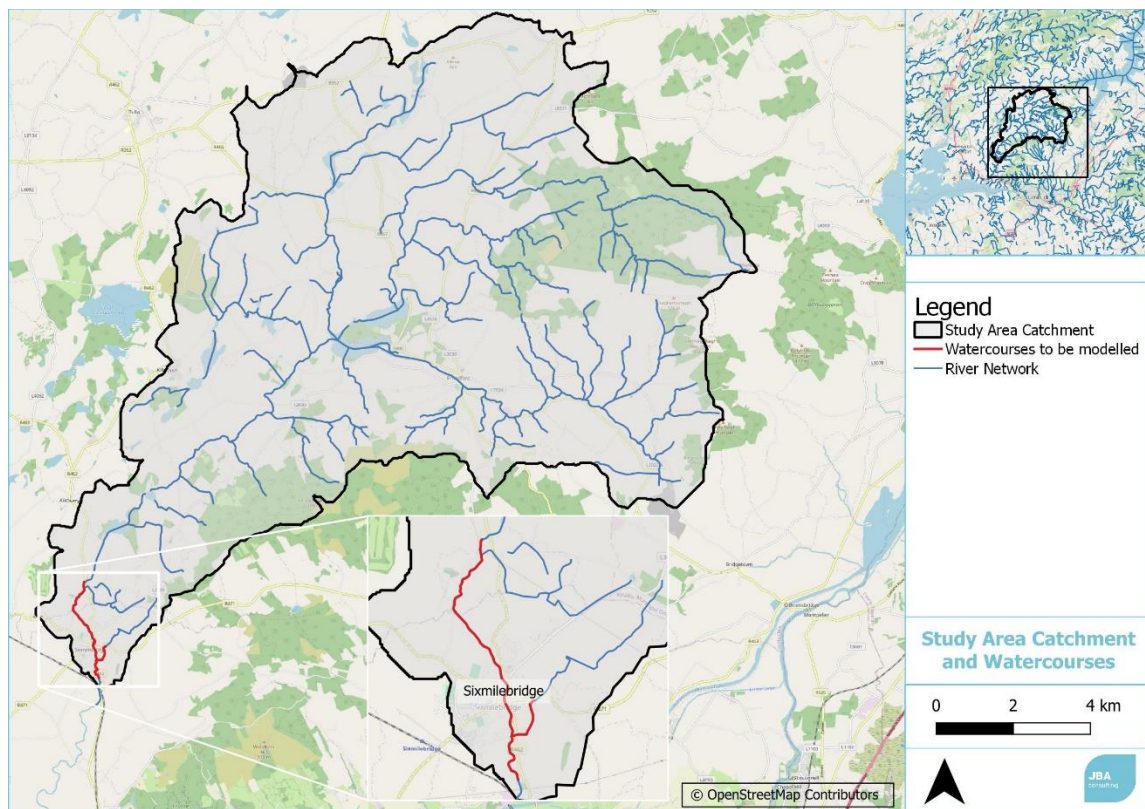


Figure 1-1: Study Area Catchment and Watercourses to be modelled

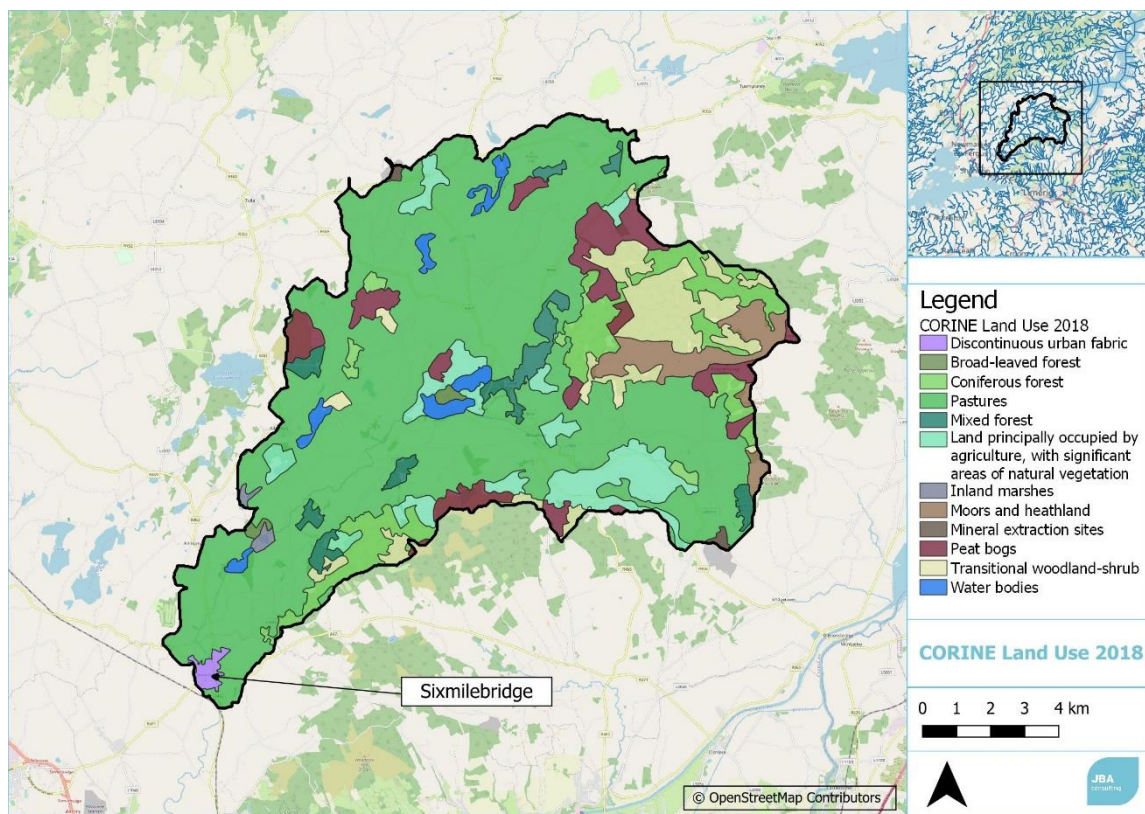


Figure 1-2: Study Area CORINE Land Use 2018

2 Review of Available Data

This section provides a summary of the available hydrological data for the study including flood history, hydrometric and meteorological gauges. The previous detailed summary for the area (Shannon CFRAMS) is reviewed separately in Section 3.

2.1 Hydrometric Data

There are two active gauging stations within the study area: Owenogarney Br. (27011) and Castle Lake (27014). The hydrometric summary of both is presented in Table 2-1. The location of these stations is shown on Figure 2-1. The contributing catchment area downstream of Castle Lake gauge is 5.4% of the total Owenogarney gauge catchment area.

Table 2-1: Hydrometric Summary of study area gauging stations

Gauge Name	Owenogarney (Rly) Br	Castle Lake Outlet
Number	27011	27014
Catchment Area	161.80 km ²	153.00 km ²
Operator	OPW	Clare Co Co
Watercourse	Owenogarney	Owenogarney
Type	Recorded	Recorder
FSU ranking	NA	NA
Data record	1994-Present; 26 years	2012-Present; 10 years
Notes	Fluvial peak flow manually extracted from the semi-tidal record.	Could be lake outflow dominant. There is a drinking water abstraction from Castle Lake for the Shannon Public Water Supply Scheme.

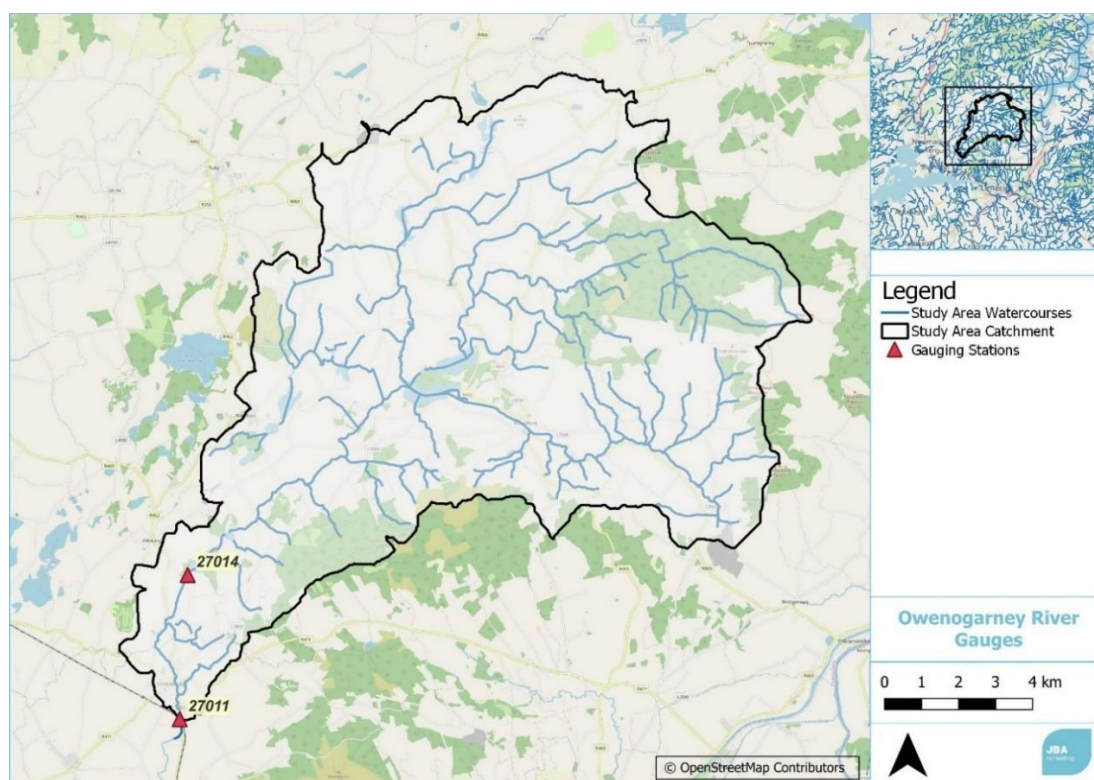


Figure 2-1: Location of the Owenogarney river gauging stations

2.1.1 Owenogarney (RL) Br Gauge 27011

The gauge 27011 has a record of 28 years which provides consistent AMAX data to calculate the QMED at the gauge, but the record has a break from 24/02/2001 to 03/10/2001 which means hydrometric year 2000 is only half available. It is therefore possible that the water level has risen above the maxima given. The gauge is tidally influenced, however, the tide has been manually removed from the record by OPW and the flow has been estimated using the rating curve. The recorded AMAX fluvial flow data is presented on Figure 2-2. Considering all this information the AMAX series could be potentially reliable. A Single-Site analysis is undertaken further in Section 6. As the OPW have extracted and supplied fluvial AMAX records from this tidally influenced gauge no correction of the visible increasing trend in AMAX records has been carried out.

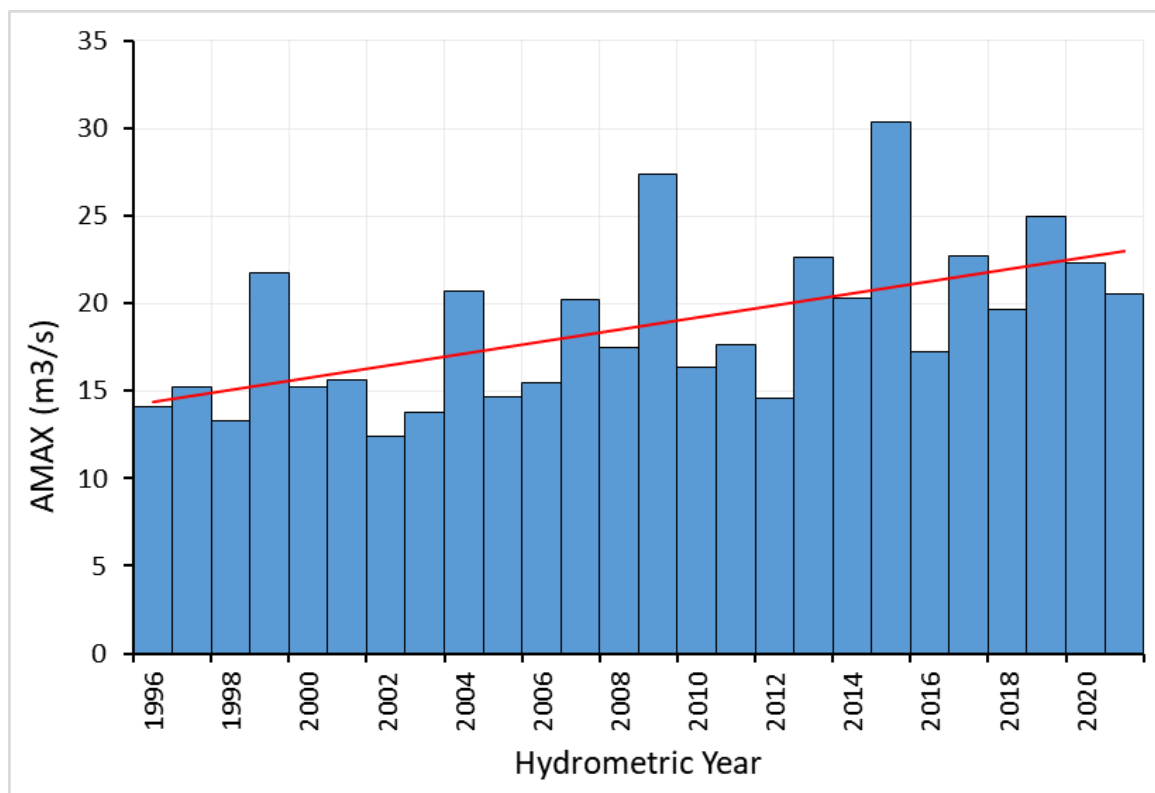


Figure 2-2: AMAX series, station 27011 - Owenogarney (RL) Br

2.1.2 Castle Lake O/L Gauge 27014

Gauge 27014 has been installed by EPA on 08/10/2012 to monitor outflow from Castle Lake. On the EPA website, the station is marked as having a good rating. The gauge record is too small (only 11 years) to be used in the hydrological analysis, and it is also a lake outflow dominant, providing slow attenuation and therefore very long hydrographs. The review of the peak flows for the gauge will be undertaken in Section 6.

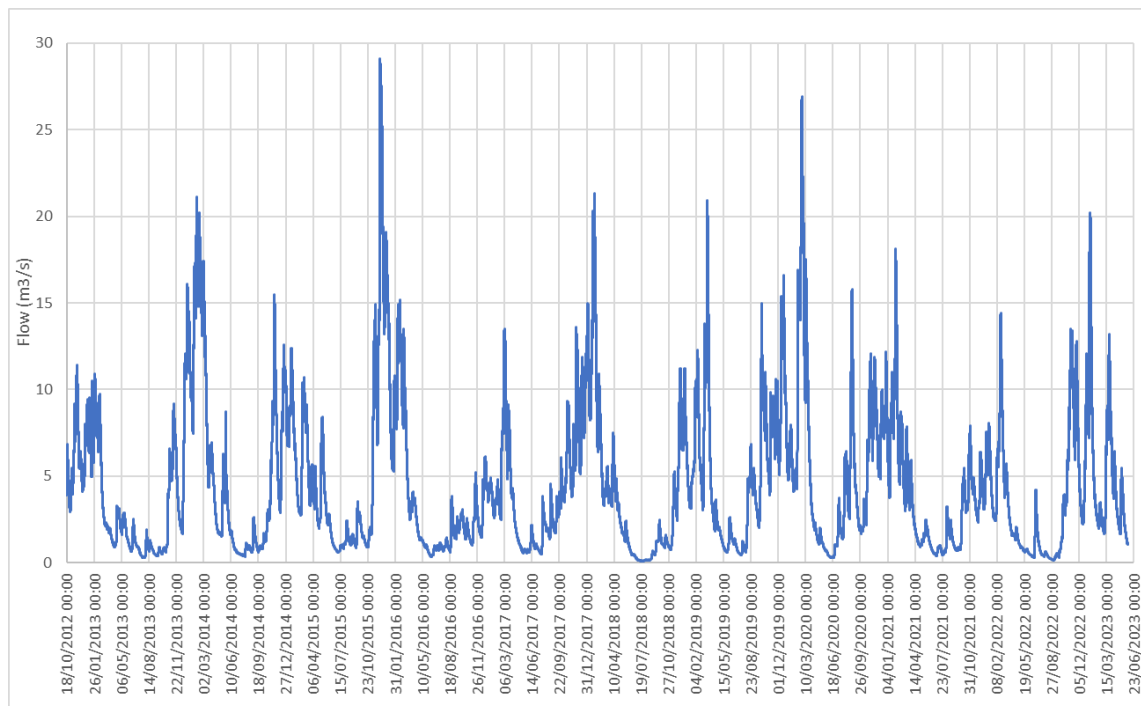


Figure 2-3: Flow time series, station 27014 – Castle Lake (O/L)

2.1.3 Combined Review of Hydrometric Data

Castle Lake gauge is located c.4.5km upstream of Owenagarney Br. Gauge. Review of the stage peak series was conducted for Owenagarney Br and Castle Lake gauges (the flow series are not fully available on Owenagarney Br). The Owenagarney Br stage was scaled to match the same range as Castle Lake record. It is noted that the hydrograph shape at the Owenagarney Br correlates well with the Castle Lake gauge hydrographs. This can be seen on Figure 2-4 below. The tidal influence on Owenagarney Br has not been removed, so the stage series presented is semi-tidal. Due to the high influence of the lake, outflow is shown on hydrographs for both gauges, care needs to be taken when applying the gauge record data.

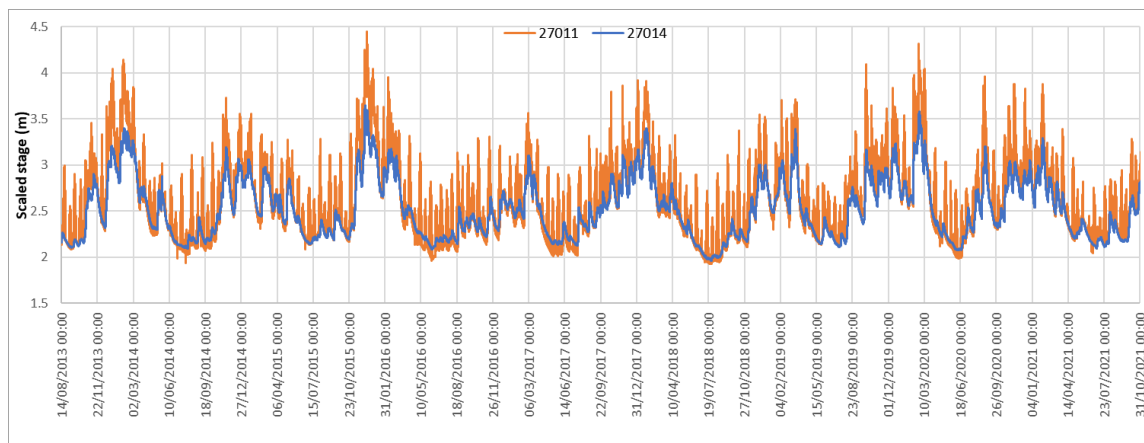


Figure 2-4: Stage series comparison for stations 27011 and 27014

2.2 Review of River Network and Descriptor Data

2.2.1 River Network

The EPA blueline river network was reviewed for the study area and compared with satellite imagery, background mapping (ESRI and Bing satellite, OSM standard base map data, Ordnance Survey Ireland 50k mapping, historic mapping). Review of these data has identified two additional tributaries of the Owenagarney River in the lower part of the catchment. These changes to the river network are shown on Figure 2-5.

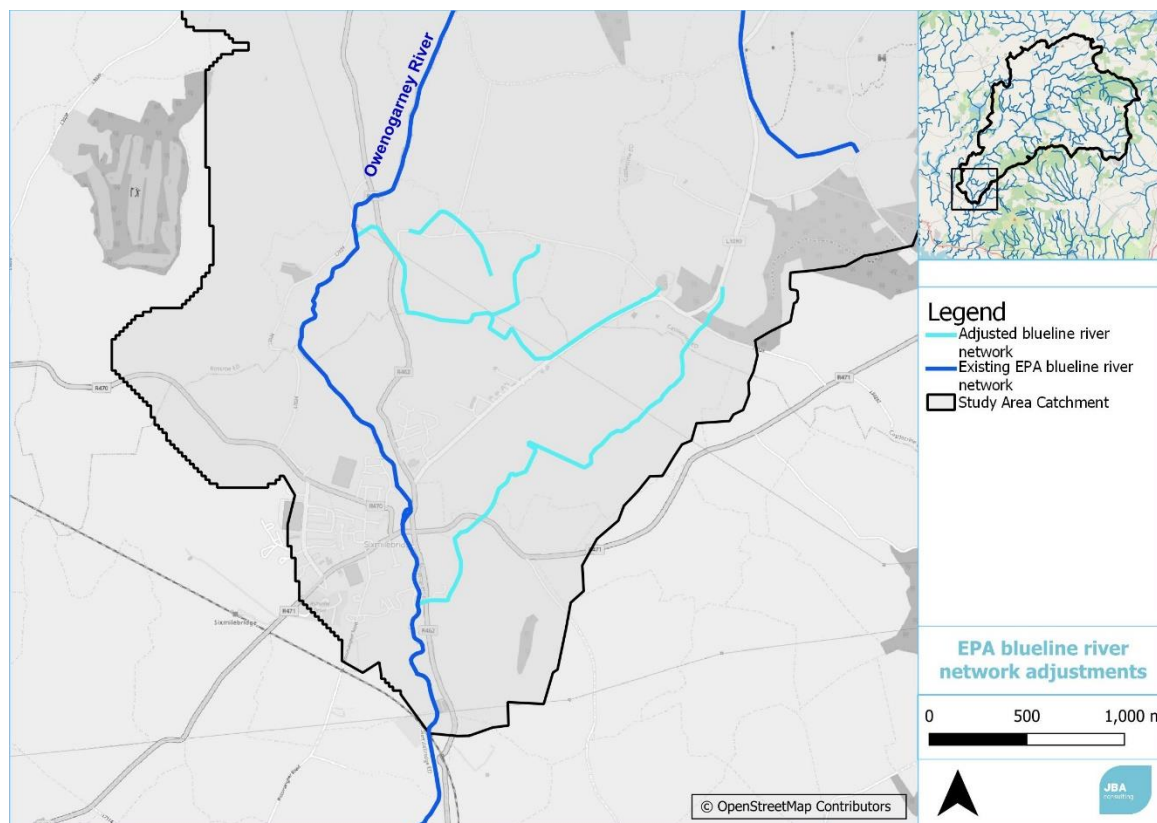


Figure 2-5: EPA blue line network adjustments

2.2.2 Topographic Data and Mapping

To verify the catchment boundaries of the study area OpenStreetMap Topography 10m contour lines and Esri Topography 20m contour lines were used. The catchment boundary verification and update were conducted using the OpenStreetMap 10m contour lines. Catchment descriptors which are dependent on topographic data have been updated accordingly.

2.2.3 Hydrology Estimation Points Update

A total of 10 Hydrological Estimation Points (HEPs) were derived for this study. This doesn't include the HEPs downstream of Owenagarney Br gauge that are included in the CFRAM HEPs, as these are not covered by the proposed modelled extent. However, the physical catchment descriptors will still be updated for them to incorporate the changes in the river network. These will be supplied as accompanying GIS datasets. Appendix A contains catchment maps with location of hydrological estimation points.

2.2.4 Catchment Descriptors

A review of the key catchment characteristics was conducted for Hydrological Estimation Points (HEPs) used in the Shannon CFRAM study. This was done along with the review of the topography and watercourses. These include AREA, SAAR, URBEXT and S1085. It should be noted that the SAAR values used in this study are for the period 1961 – 1990. The latest datasets (1991-2020) are not publicly available yet. Refer to Table 2-2 for the findings of the review. Appendix B contains table with catchment descriptors used for each HEP in the hydrological assessment for the study.

Table 2-2: Catchment descriptors review

Descriptor	Data used in original FSU or Shannon CFRAM	Updated data set	Need for adjustment?
AREA	Catchment areas delineated using a hydrologically adjusted DTM	OpenStreetMap Topo 10m contours	Visual inspection of the catchment boundaries carried out to ensure that watersheds and flow paths are properly accounted for. New catchment added.
SAAR	Gridded annual rainfall data for the period 1961 – 1990 from MET Eireann used to generate catchment SAAR values	No change to dataset used	The latest annual average rainfall period data (1991 – 2020) is not publicly available yet.
FARL	Waterbodies defined using EPA waterbodies shapefile and the % of catchment area with lakes or reservoirs calculated	No major alterations or adjustments to the initial dataset	No changes to FARL within the study area.
BFI Soil	Catchment wide value of baseflow index estimated using watercourse data	No change to dataset used	No changed to BFI Soil, however values of BFI soil for tributary catchment based on the existing values from FSU.
URBEXT	Corine land cover 2000 dataset used to get % of urban land use within the catchments	Corine land cover 2018 dataset released	No changes to URBEXT, but calculated for the new tributary catchment
MSL	Stream length measured from GIS shapefile of watercourses	Stream length measured from GIS shapefile of updated watercourses	MSL for each individual catchment measured based on updated river network.
S1085	Slope determined using OSi DEM data	No update	S1085 values for the new catchment calculated.
DRAIN2	Calculated by dividing channel length by catchment area	Updated topographic data available for catchment area review	DRAIN2 values for the new catchment calculated.
ARTDRAIN2	OPW arterial drainage network	No update	No changes in channels affected by arterial drainage, no need for adjustment.

2.3 Meteorological Data

2.3.1 Rainfall Data

There are no rainfall gauging stations within the Owenogarney catchment, but there are number of stations in the neighbouring catchments. The rainfall data in the form of DDF will only be used for this study to generate the peak flows and hydrographs for the unnamed tributary. The methodology for estimating the peak flows for the Owenogarney is based on the Flood Study Update (FSU), so no other rainfall records are required.

2.3.2 Met Eireann Depth Duration Frequency (DDF) Data

Rainfall depths from various return period and storm durations have been sourced from the Met Eireann Depth Duration Frequency (DDF 2007) dataset. The dataset allows rainfall depths to be assigned to annual exceedance probabilities and was developed using available gauged data. One DDF¹ point within the catchment was identified, as it is very small with no spatial difference within it. The 6 hrs storm duration is considered the critical for the tributary. Table 2-3 shows the rainfall depths in mm associated with 6hrs duration for the unnamed tributary. These values will be used for the FSR RR method for the tributary.

Table 2-3: Rainfall depths (mm) for 6hrs duration and return periods for tributary

Duration (hrs)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
6	21.2	27.4	31.8	36.5	43.5	49.6	56.1	72.3

2.4 History of Flooding

A summary of all the recorded flood events that have occurred in Sixmilebridge are provided in this section. The information was collected from the OPW website www.floodinfo.ie, Shannon CFRAM Study Hydrology report and an internet search. Table 2-4 summarises the peak level records available for each event. A description of the most significant historic events is outlined in the following sub-sections.

Table 2-4: List of flood events occurred in Sixmilebridge

Flood Event Date	OPW Flood Event ID	Source	Max Level (mOD) recorded on Owenogarney Br gauge	Max Level (mOD) recorded on Castle Lake gauge
January 1991	ID-560	fluvial, pluvial	NA	NA
January 1995	ID-11037	fluvial, pluvial	NA	NA
January 2005	ID-4484	pluvial	3.756	NA
November 2009	ID-376	fluvial	4.246	NA
Recurring	ID-4485	pluvial	NA	NA
January 2023²	-	fluvial	4.004	20.2

2.4.1 27th January 1995

The cause of 1995 flooding was the very heavy and prolonged rainfall allied to the generally impervious nature of the catchment. The flood event lasted for almost two weeks. The most severe impact of the flooding was the inundation of 14 houses and gardens that lie between the left bank of the river and the Sixmilebridge to Cratloe Road. The sewage works on the left bank was completely flooded that caused backing up of the sewage system. The flood waters in the unnamed tributary also back up when the Owenogarney river was in flood, contributing further to the flooding of the houses.

¹ The DDF points relate to rainfall data between 1941-2004

² [https://www.clarecoco.ie/your-council/\[news\]/rosmanagher-road-in-sixmilebridge-temporarily-closed.html](https://www.clarecoco.ie/your-council/[news]/rosmanagher-road-in-sixmilebridge-temporarily-closed.html)

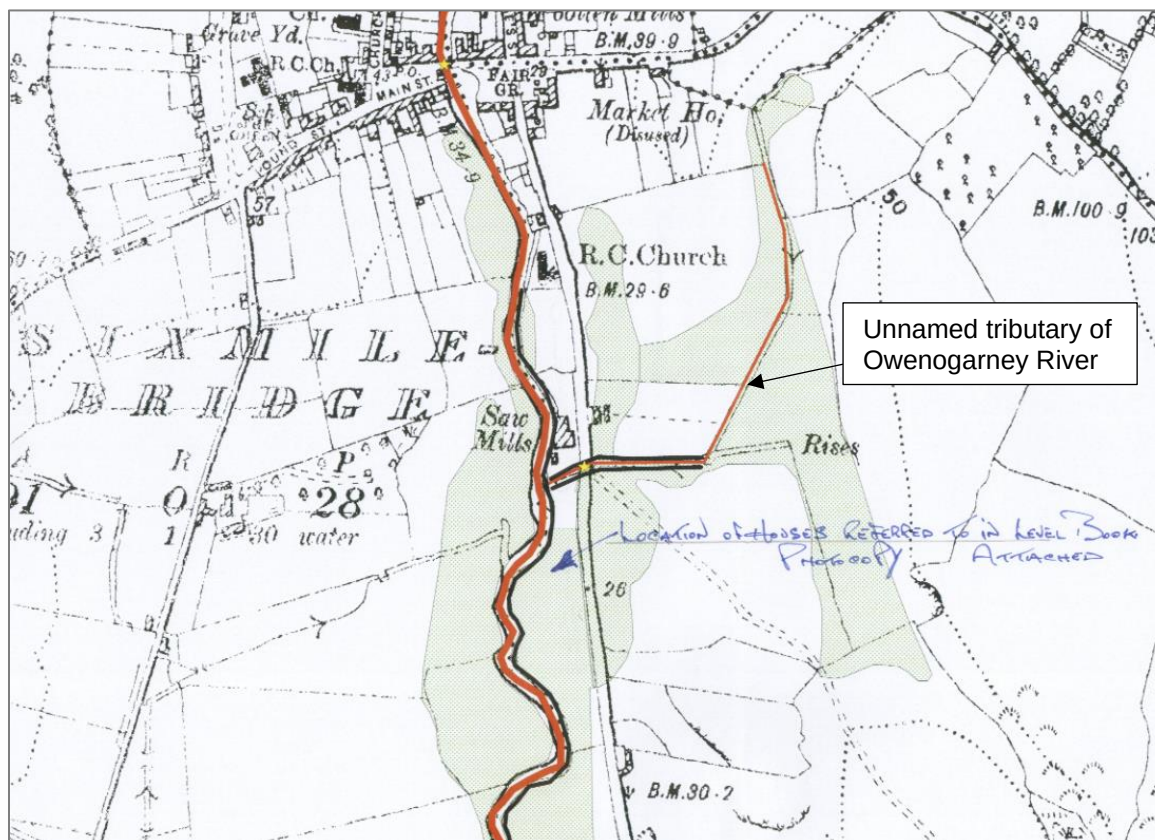


Figure 2-6: Extract from the January 1995 flood extent map

2.4.2 7th January 2005

During the January 2005 flooding event several road sections at and in the vicinity of Sixmilebridge were flooded, mainly due to prolonged heavy rainfall and inadequate drainage capacity. 450mm of water was recorded on the road close to the railway. This problem occurs once every five to 10 years (ID-4485).

2.4.3 22nd November 2009

This flooding event started on Thursday, 19th November 2009 and ended on Friday, 22nd November 2009. The source of the flood waters was the Owenagarney River. The event caused flooding of the Sixmilebridge car park and the local roads.

2.5 Previous Studies

Two previous studies examining flood risk and flows within the study area are relevant for this study:

- The Shannon Catchment Flood Risk and Management study (Shannon CFRAM)
- Site-Specific Flood Risk Assessment for Planning Application 18/594

The Shannon CFRAM being the most detailed hydraulic modelling study for the area is reviewed in detail in Section 3.

The SSFRA was prepared by JBA for a site located adjacent to the unnamed tributary, that was not include in the Shannon CFRAM study. The report provides an updated flows for the Owenagarney River at Sixmilebridge that are greater than those estimated by the Shannon CFRAM for the reach. The SSFRA outcomes are further discussed in Section 4.

The methodology and final flow results from this study are compared against those derived for Flood Map Update in Section 6.

3 Review of Shannon CFRAM Hydrology

This section reviews the Shannon CFRAM hydrology and highlights new information available or changes in the catchment that have occurred since its completion.

Sixmilebridge has been modelled as an AFA (Model S03) for Shannon CFRAM study. Nine HEPs has been used for the model at Owenogarney River, these are presented on Figure 3 1. The unnamed tributary on the left bank has not been included in the hydrological analysis.

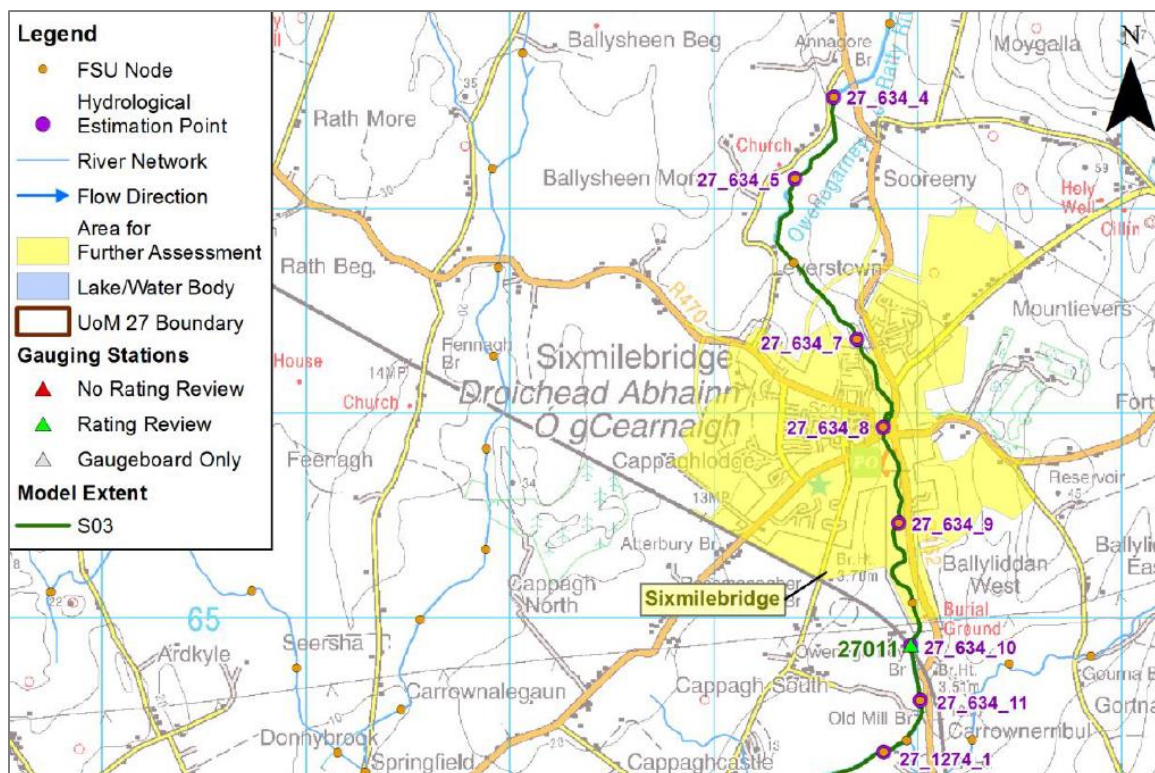


Figure 3-1: Extract from Shannon CFRAM Hydrology Report, HEP map for Model S03

3.1 Rating Review Summary – 27011 – Owenogarney (Rly) Br

As part of the hydrological analysis in the Shannon CFRAM study, station 27011 has undertaken a rating review analysis. The rating equations at Owenogarney Br gauge have not been revised as the OPW rating equations appear to be a better representation of the site. The model rating seems to correlate well with all the sites check gaugings up to QMED (15 m³/s). However, for flows greater than QMED the model rating diverges away from the check gaugings. The OPW rating fits well to the majority of high flow gaugings and is confirmed by these gaugings up to 1.8xQMED. The CFRAM rating review for the 27011 station is presented on Figure 3-2. It was recommended that the rating at this site is not revised for high flows and the most recent OPW rating should continue to be used.

It is noted that OPW's Hydro-Data web-database reports that the AMAX series from 2005 to 2012 are tidal peaks. The AMAX data received from OPW came with the comment that 'fluvial peaks are estimated from semi-tidal hydrographs'. Therefore, the AMAX series should be treated with caution if it is used at all. The lack of scatter in the check gaugings suggests that these were collected during periods of low tide, and the original rating is therefore believed to be representative during low tides. The estimate of QMED excluding the data from 2005 to 2012 is similar to the estimate including that data (both 15 m³/s).

The station only has level and flow AMAX data from 1996 when drainage works were completed local to the station. Given the fact that OPW have reported that the AMAX flows may be affected by the tide it was recommended that this data is not used for QMED estimation or flood frequency analysis in the Shannon CFRAM study.

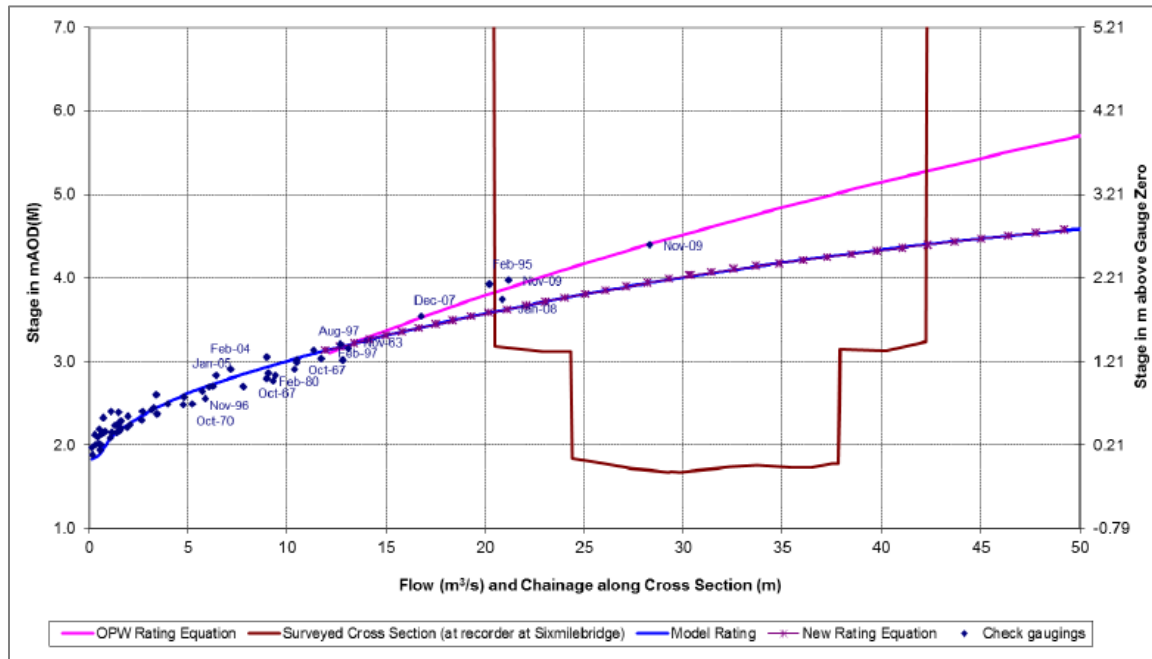


Figure 3-2: Rating Review of station 27011 – Owenogarney (Rly) Br

3.2 Review of Index Flood Flow Estimation (QMED)

Index Flood Flow or QMED is a median annual maximum flood. At a gauged location QMED is approximately equal to flood with a return period of 2 years (50% AEP) and as a rule-of-thumb it is approximately equivalent to bank full flow. QMED is the initial flow estimation value needed to derive larger event flows for gauged and ungauged catchments. At ungauged locations different methodologies using Physical Catchment Descriptors (PCDs) are used to derive QMED while at gauged locations recorded data is used to establish the value.

In the Shannon CFRAM both gauged and ungauged QMED values were derived. The conclusion on the gauged QMED analysis was the gauge is affected by tides rendering the AMAX series potentially unreliable.

To calculate the QMED for all ungauged Hydrological Estimation Points (HEPs) the FSU 7-variable ungauged catchment descriptor equation was used and adjusted by applying a weighted geometric mean value from hydrologically similar gauges which is 1.19.

3.3 Review of Growth Curve Generation

The growth curve used in CFRAM study was Extreme Value Type 1 (EV1) curve derived from the pooling group method based on the standard hydrological similarity. The Owenogarney gauge is tidally influenced and therefore the growth curve derived from a single site analysis was deemed unsuitable in the CFRAM study. The growth curves reviewed under the Shannon CFRAM hydrology for the Owenogarney river are presented on Figure 3-3.

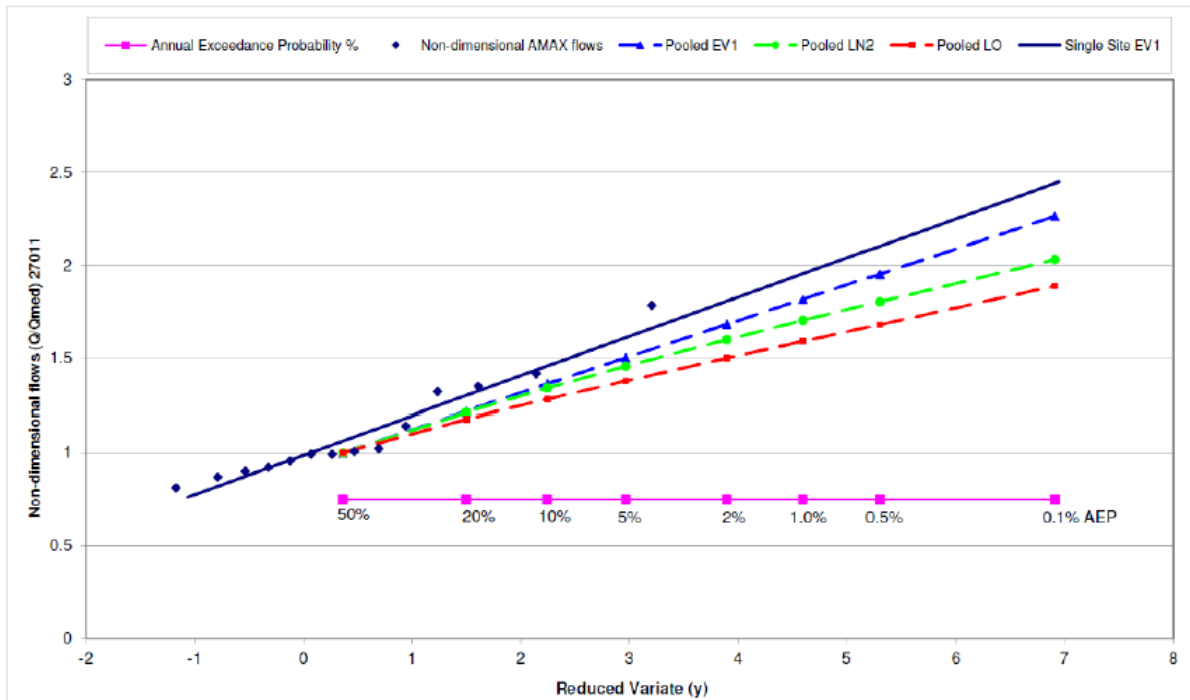


Figure 3-3: Growth curves review from Shannon CFRAMs Hydrology Report

3.4 Design Flow Hydrographs

The hydrographs for the Owenagarney river were mainly derived using the FSU method. Refer to Figure 3-4 for the typical hydrographs used in the Shannon CFRAM for the Owenagarney river.

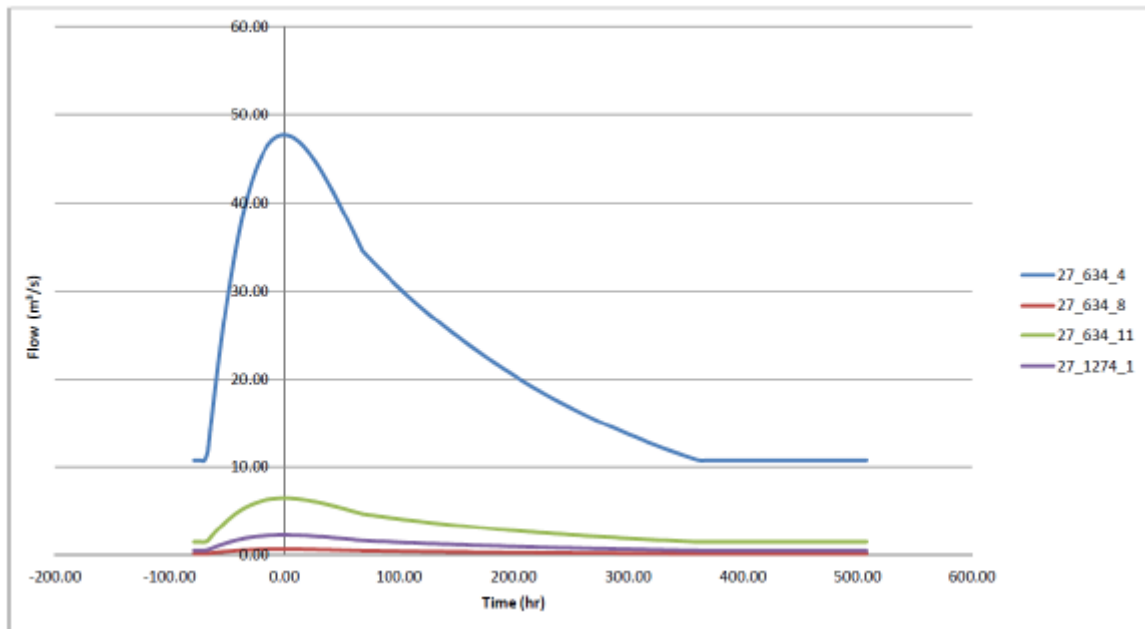


Figure 3-4: Design Inflow Hydrographs for 1%AEP from Shannon CFRAMs Hydrology Report

4 Review of the Site-Specific Flood Risk Assessment for Planning Application 18/594

JBA Consulting was appointed by Clare County Council to undertake a Site-Specific Flood Risk Assessment for a site in Sixmilebridge adjacent to the Unnamed Tributary with a detailed hydrology and hydraulic assessment.

The QMED estimation method for the Owenogarney River is same as in the Shannon CFRAM study (the FSU 7-variable equation) but it was decided not to adjust the value. The growth curve used is based on the pooled analysis and is the EV1 (the same distribution type as the Shannon CFRAM study). The pooling group used in the SSFRA is different to the pooling group used in the CFRAM. For the unnamed tributary a conservative approach was taken using the IH124 method and the Regional FSR Growth curve for Ireland. The estimated peak flows in the SSFRA are presented in Table 4-1.

Table 4-1: Peak Flows estimated in SSFRA for PA 18/594

	QMED Method	QMED (m ³ /s)	Growth Curve	10% AEP (m ³ /s)	1% AEP (m ³ /s)	0.1% AEP (m ³ /s)
27_634_10	FSU	22.18	EV1 pooled	31.62	43.40	54.96
Unnamed Tributary	IH 124	1.08	Regional FSR Ireland Growth Curve	1.57	2.24	2.97

5 Summary of the Review of Available Information

From the review of the available data and previous studies, the following key observations are noted:

- Two hydrometric gauging stations are present on the Owenagarney River: Owenagarney Br (27011) and Castle Lake (27014). Both gauges have some limitations and uncertainties in their record, however they still provide a reasonable degree of information on the nature of the catchment. The OPW have provided a separation of fluvial peak events to derive a fluvial only AMAX series of the Owenagarney River gauge record. A more detailed assessment on both gauges is required.
- The physical catchment descriptors for each identified HEP have been updated. The PCDs affected by the change are: MSL, DrainD and NETLEN. Two new HEPs added to the unnamed tributary, the PCDs for each have been estimated from the available datasets.
- Review of the flood history in the area, indicated three significant flood events in the recent years. The flood source is mainly fluvial from the Owenagarney River and the unnamed tributary overtopping its banks.
- The available flood mapping studies in the area (Shannon CFRAMS and SSFRA) have conducted hydrological review for Sixmilebridge. In both studies the Castle Lake gauge was not reviewed. The QMED estimation method for the Owenagarney River in both studies is the FSU 7-variable equation. The gauged QMED in both studies was lower than the FSU calculated and therefore the QMED in the CFRAM was adjusted to the weighted geometric mean value from hydrologically similar gauges, while in the SSFRA the QMED was used unadjusted. The growth curve used is EV1 from pooled analysis, the pooling groups has differences between the two studies. The SSFRA includes hydrological estimation for the unnamed tributary.

Taking all the above into account, the design flow estimation for the study needs to include detailed review of the available gauge records. The FSU method is reasonable for estimating the QMED for the Owenagarney River, however, given the uncertainties in the recorded AMAX, a review of more growth curves needs to be undertaken. A conservative estimate of flows for the unnamed tributary in the SSFRA were chosen. Uncertainty remains and so alternative methods need to be analysed and considered to select the most appropriate peak flow estimates for this study.

6 Design Flow Estimation

Following the review of the available data and studies in the previous sections, the design flows for the study will follow the FSU methodology used in the Shannon CFRAM and SSFRA with more detailed single-site analysis of the two available gauges. The objective of design flow estimation is to derive design event peak flows and hydrographs for HEPs at the Owenogarney River and the Unnamed tributary.

6.1 Owenogarney River Flow Estimation

The single-site analysis has been undertaken for Owenogarney Br and Castle Lake gauges. To estimate the peak flows for the 50%, 20%, 5%, 2%, 1%, 0.5% and 0.1% AEP events. The Flood Studies Update (FSU) method is considered appropriate as the best and most current available method for the Owenogarney river catchment. As some of the catchment descriptors have been re-calculated, it is expected that the QMEDs will be slightly greater than those calculated in the Shannon CFRAM. The full description of the methodology is presented in Appendix C.

6.1.1 Single-site Analysis station 27011 – Owenogarney (RL) Br

The gauge has record of 26 AMAX flows which can be used to calculate the gauged QMED. The gauge is tidally influenced, however, the tide has been manually removed from water level by OPW and flow has been estimated using the rating curve.

The gauge AMAX flow series is presented on Figure 6-1. The series has significant rising trend, which is notable by looking at the Mann-Kendall trend line in AMAX values over time (Figure 6-2). No correction for this trend has been carried out as the OPW extracted fluvial AMAX record series is considered the best available dataset. **The derived gauged QMED from the single-site analysis of station 27011 is 17.39m³/s.**

There is a minimal difference between the fitted growth curve from the EV1, GEV and LN3 distributions. The single-site growth curves are presented on Figure 6-3.

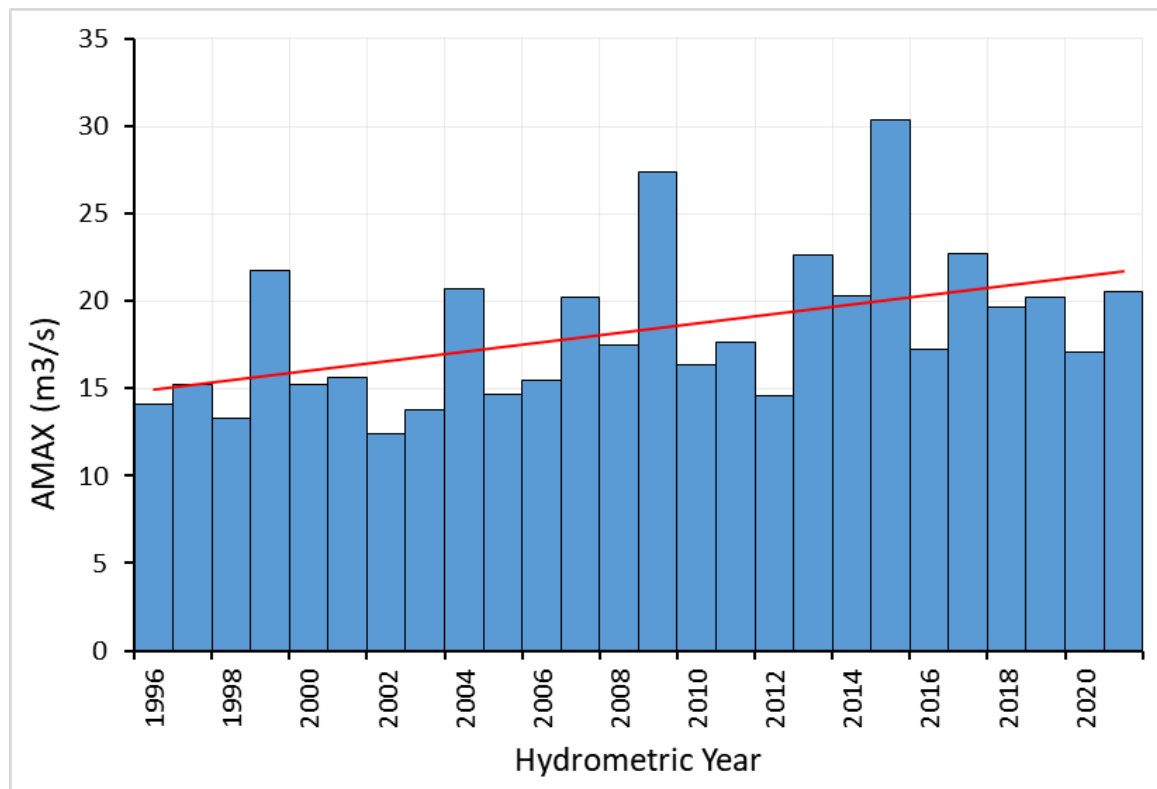


Figure 6-1: AMAX series, station 27011 - Owenogarney (RL) Br

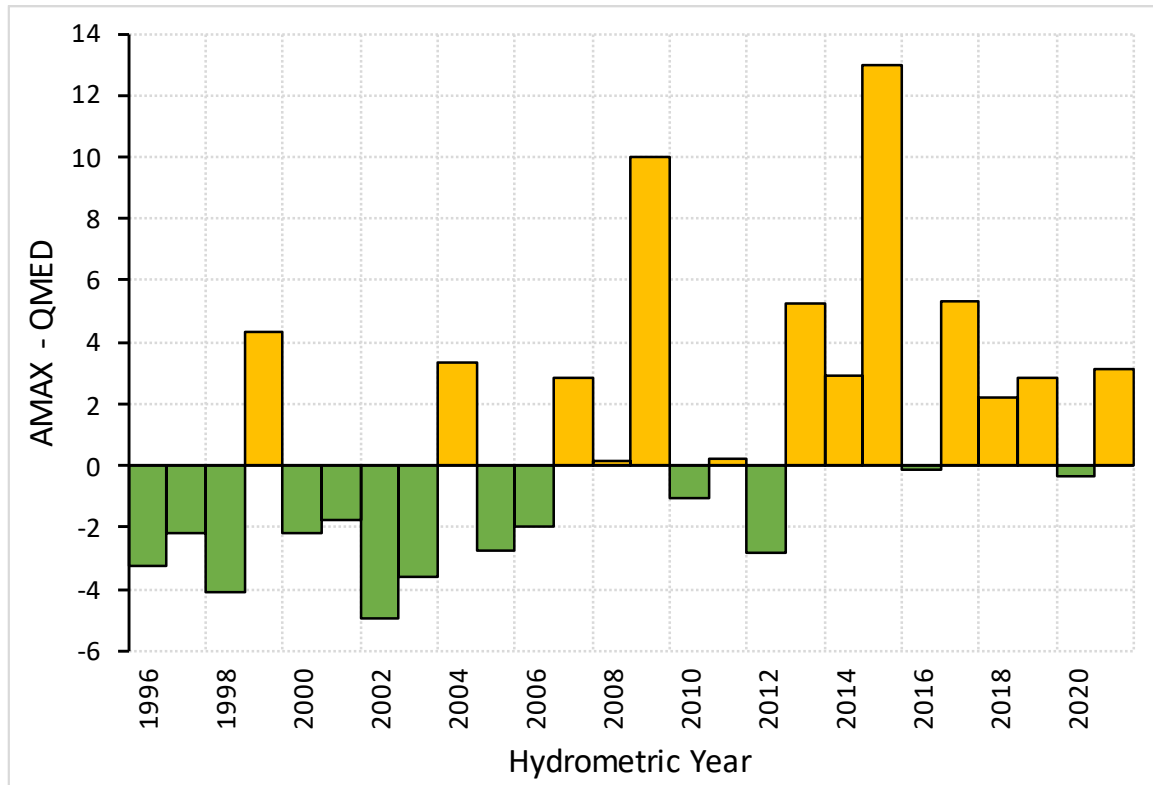


Figure 6-2: Differences between AMAX and QMED series - 27011

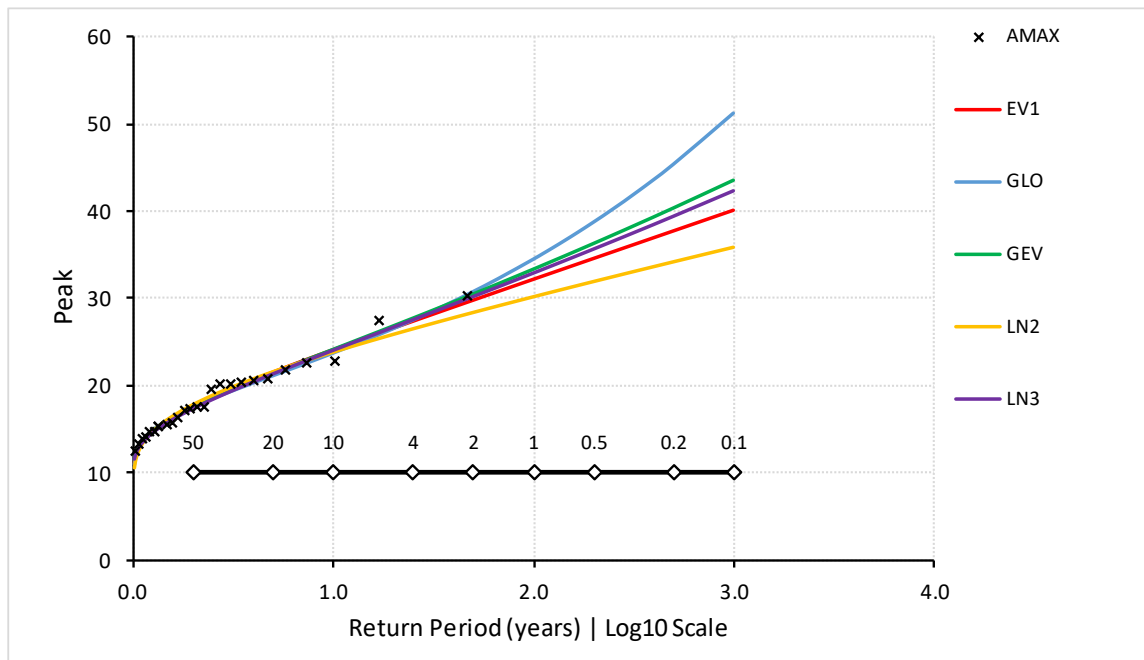


Figure 6-3: Selected flood frequency curves - station 27011

Table 6-1: Growth Curve Fitting - station 27011

Annual Exceedance Probability (%)	EV1	GLO	GEV	LN2	LN3
50% (2yr)	1.00	1.00	1.00	1.00	1.00
20% (5yr)	1.22	1.20	1.22	1.21	1.23
10% (10yr)	1.37	1.35	1.38	1.33	1.38
5% (20yr)	1.51	1.51	1.53	1.45	1.53
2% (50yr)	1.69	1.75	1.74	1.59	1.73
1% (100yr)	1.83	1.97	1.90	1.69	1.89
0.5% (200yr)	1.96	2.21	2.07	1.79	2.05
0.1% (1000yr)	2.28	2.92	2.49	2.01	2.43

6.1.2 Single-site Analysis station 27014 – Castle Lake (O/L)

The gauge has record of 11 years (2012-2023). The gauge AMAX series is presented on Figure 6-4. **The derived gauged QMED from the single-site analysis of station 27011 is 20.2m³/s.**

Some studies³ recommend not to rely on the QMED equation when FARL is below around 0.9, due to impounding reservoirs, unless they are kept permanently full and thus act like natural lakes. FARL makes no distinction between reservoirs, where the water level may be below the spillway, and lakes, which generally have a continuous outflow. FARL also does not reflect the actual attenuation effect of a specific lake, some of which could have control structures that alter downstream flow regime.

There is a minimal difference between GEV and LN3 distributions and between GLO, EV1 and LN2, them being the steepest. It should be noted, that for Owenogarney Br gauge LN2 was the shallowest. These differences imply that the statistical descriptors of the AMAX series of the two gauges are very different. This makes it hard to select one that is representative of the AMAX data that is available.

The single-site growth curves for Castle Lake gauge are presented on Figure 6-5.

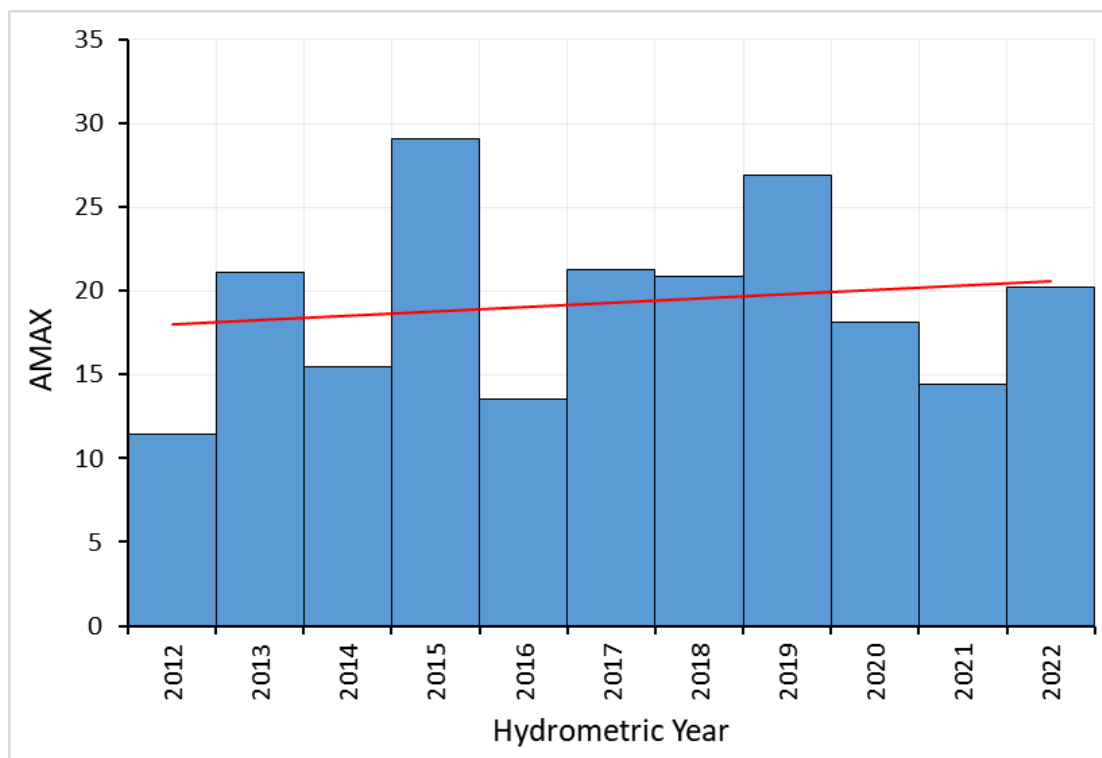


Figure 6-4: AMAX series, station 27014 – Castle Lake (O/L)

³ FEH Study - 313.7.4

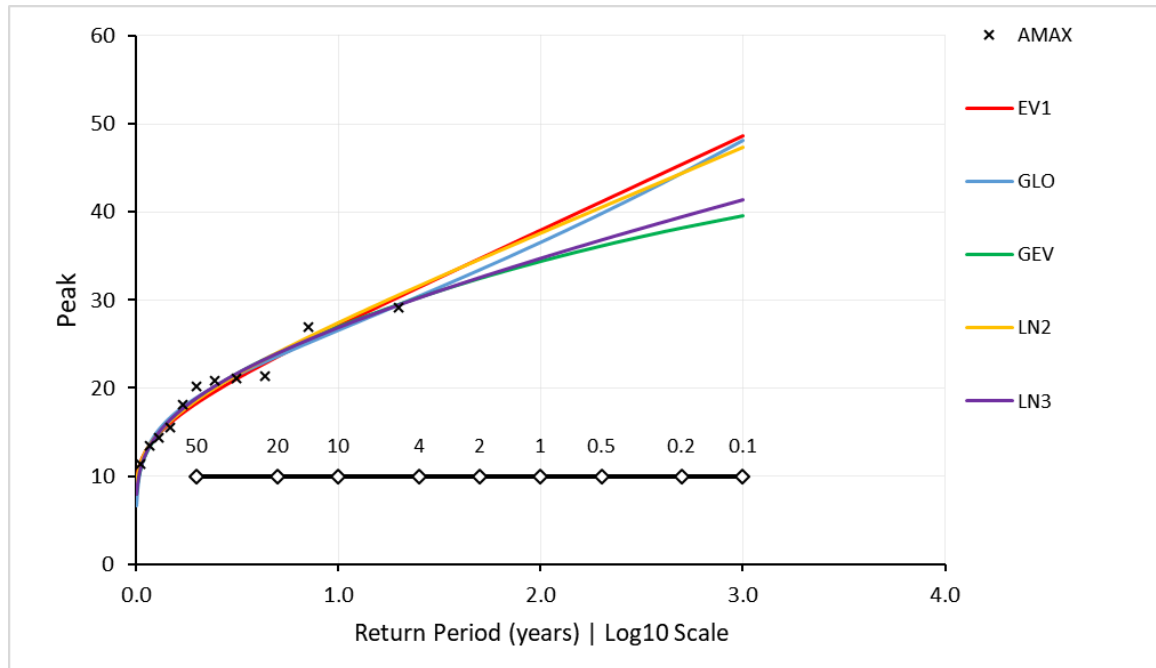


Figure 6-5: Selected flood frequency curves - station 27014

6.1.3 Peak-Over-Threshold Analysis station 27014 – Castle Lake (O/L)

A peak-over-threshold (POT) analysis has been undertaken for gauge 27014 due to the limited number of years. The peak flow data has been tested using 5 different thresholds and using two different POT equations, refer to Figure 6-6 for the reviewed thresholds. The POT equations used are shown below:

FSR RR:

$$q_0 + \beta \ln \lambda + 0.5772 \beta$$

Where q_0 – threshold (m^3/s);

β - difference between mean POT flow and q_0

λ – number of flows divided by number of years used in the analysis

FEH:

$$Q_{MED} = w Q_i + (1 - w) Q_{i+1}$$

Where w – is weight based on the Q ranking (i and $i+1$ position) (Ref to Table 2.1 of FEH Report)

The analysis shows that there is no significant difference in the estimated QMED using FSR RR equation for all thresholds. The FEH equation is based on the peak flow ranking and therefore the thresholds between $10\text{m}^3/\text{s}$ and $18\text{m}^3/\text{s}$ are equal. As it can be seen in Table 6-2, there is no big variation between all the estimated QMEDs.

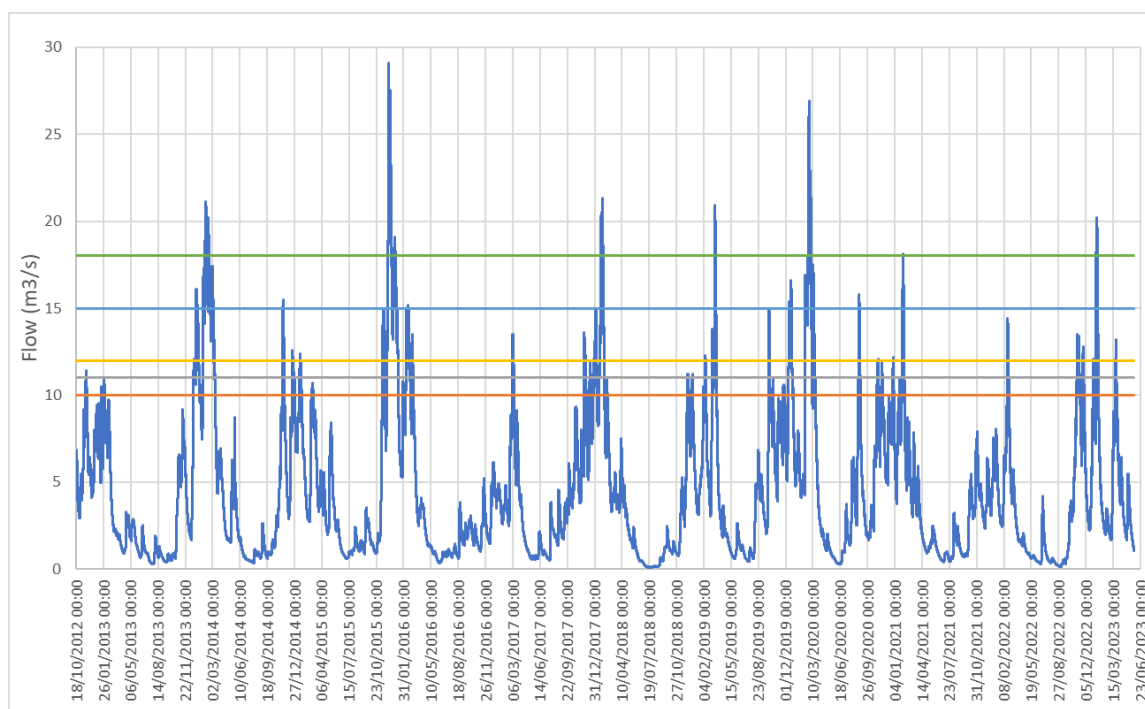


Figure 6-6: Recorded flow at 27014 and POT reviewed

Table 6-2: QMEDs derived from POT analysis -27014

Threshold (m³/s)	Number of Peaks	POT QMED(m³/s)
10 (FSR RR)	48	19.88
11 (FSR RR)	42	19.49
12 (FSR RR)	35	19.43
15 (FSR RR)	19	19.23
10 (FEH)	10	19.71
18 (FEH)	6	20.90
Single-Site QMED(m³/s)		
		20.20

6.1.4 Gauged QMED Summary

The QMED derived for Castle Lake gauge is greater than the gauged QMED for Owenogarney Br which is located 4.5km downstream and having a catchment 5% larger. This is not unexpected due to the likely attenuating effect of the river channel and floodplain, combined with the small increase in contributing catchment area between the gauges and few tributary inflows, as shown in Figure 1-1 above. There is significant uncertainty in the gauged QMED estimates for both gauges.

6.1.5 Flood Studies Update Method (FSU) – QMED estimation

QMED estimates for the Owenagarney catchment HEPs have been derived using the Flood Studies Update (FSU) method. The method is described in FSU WP 2.2 and 2.3 and uses a 7-variable regression equation based on 7 different physical catchment descriptors with an urban adjustment.

$$Q_{medRURAL} = (1.237 \times 10^{-5})(AREA^{0.937})(BFISoil^{-0.922})(SAAR^{1.306})(FARL^{2.217})(DRAIN^{0.341})(S1085^{0.185})((1 + ARTDRAIN2)^{0.408})$$

Where:

AREA is the catchment area (km²)

BFISoil is the base flow index derived from soils data

SAAR is long-term mean annual rainfall amount in mm

FARL is the flood attenuation by reservoir and lake

DRAIN is the drainage density

S1085 is the slope of the main channel between 10% and 85% of its length measured from the catchment outlet (m/km)

ARTDRAIN2 is the percentage of the catchment river network included in the Drainage Schemes

The urban extent will be taken into account using the following equation:

$$UAF = (1 + URBANEXT)^{1.482}$$

The estimated QMEDs for the Owenagarney River for each HEP are presented in Table 6-3. The QMED estimates increase only slightly downstream. This is consistent with expectations based on attenuation of flows in the floodplain and river channel and very few additional contributing tributaries downstream of Castle Lake, as shown in Figure 1-1 above.

Table 6-3: Owenagarney River QMED

HEP Label	AREA (km)	QMED (m ³ /s)
27_634_4	154.86	21.81
27_634_5	156.95	22.06
27_634_6	157.89	22.12
27_634_7	158.23	22.12
27_634_8	158.71	22.14
27_634_8a	158.90	22.27
27_634_9	160.58	22.24
27_634_10	161.13	22.43
27011	161.25	22.70

6.1.6 Review of the Adjustment Factors

The FSU recommends that use is made of donor (gauged) catchments to improve estimates of the QMED at ungauged sites as an inclusion of real data provides greater confidence in estimates.

The adjustment factor is the ratio of the gauged estimate of a variable to the physical catchment descriptor estimate. Thus, the adjustment factor for QMED is:

$$\text{AdjFac} = \text{QMED}_G / \text{QMED}_{PCD},$$

where G denotes the QMED estimate from gauged data and PCD – the estimate from physical catchment descriptors (FSU method – ref. to Section 6.1.5).

Table 6-4 presents the estimated QMED, gauged QMED and QMED adjustment factor for Owenogarney river gauging stations.

Table 6-4: Estimated and gauged QMED for Owenogarney river stations

Gauge	QMED gauged (m ³ /s)	QMED FSU (m ³ /s)	QMED Adjustment Factor
27011 – Owenogarney (Rly) Br	17.56	22.70	0.77
27014 – Castle Lake (O/L)	20.20	21.86	0.92

The FSU studies recommend that adjustment factors less than 1.0 are not applied to QMED estimates from ungauged catchment descriptors. Given that we have low confidence in the gauged QMEDs, the QMED adjustment factor will not be used in the estimation of flows for the HEPs in the study.

6.1.7 Pooling Group Growth Curve at location of Owenogarney Br gauge (27011)

The growth factors for the Owenogarney Br gauge have also been estimate using the FSU pooling group method. The AMAX record for the gauges in Ireland on the FSU web portal is only available until 2004. JBA has an internal database of the gauges with AMAX records available until 2019 that was used for generating the pooling groups for the study.

For pooled analysis within the FSU, gauges are chosen on the basis of their similarity with the subject catchment according to three catchment descriptors, i.e. AREA, SAAR and BFIsol. The report on FSU WP 2.2 presents two alternative equations for calculating the similarity of catchments according to these three descriptors. For this study, equal weight is given to each of these variables, applying the similarity distance formula given as Equation 10.2 in the report on FSU WP 2.2. two gauges additionally were included with FARL lower than 0.9. The pooling group used is presented in Appendix C.1.2. Thirteen gauging stations with 673 total years were used to derive the pooling group growth curve.

For pooled growth curves, WP 2.2 recommends considering 3-parameter distributions, because the extra data provided by the pooling group ensures that the standard error is lower than it would be for single-site analysis. The report states that either the generalised extreme value (GEV) or generalised logistic (GLO) distributions are worth considering.

The GEV and GLO pooled growth curves were compared with growth curves derived from the gauged observed data as shown on Figure 6-7. From the pooling group analysis for the Owenogarney Br gauge, the GLO pooled is steeper than the GEV pooled, and the GEV pooled is very close to the Single-Site EV1. Two additional growth curves have been added for comparison – Castle Lake 27014 Single-Site and POT. It is noted that those two are steep and close to the GLO pooled growth curve.

Due to the high degree of uncertainty between the growth curves derived from the gauged data, these are not recommended in the study. Between the GLO and GEV from pooled analysis, the GEV distribution is considered to be the most appropriate for the study, the GLO is way too steep considering the catchment attenuation.

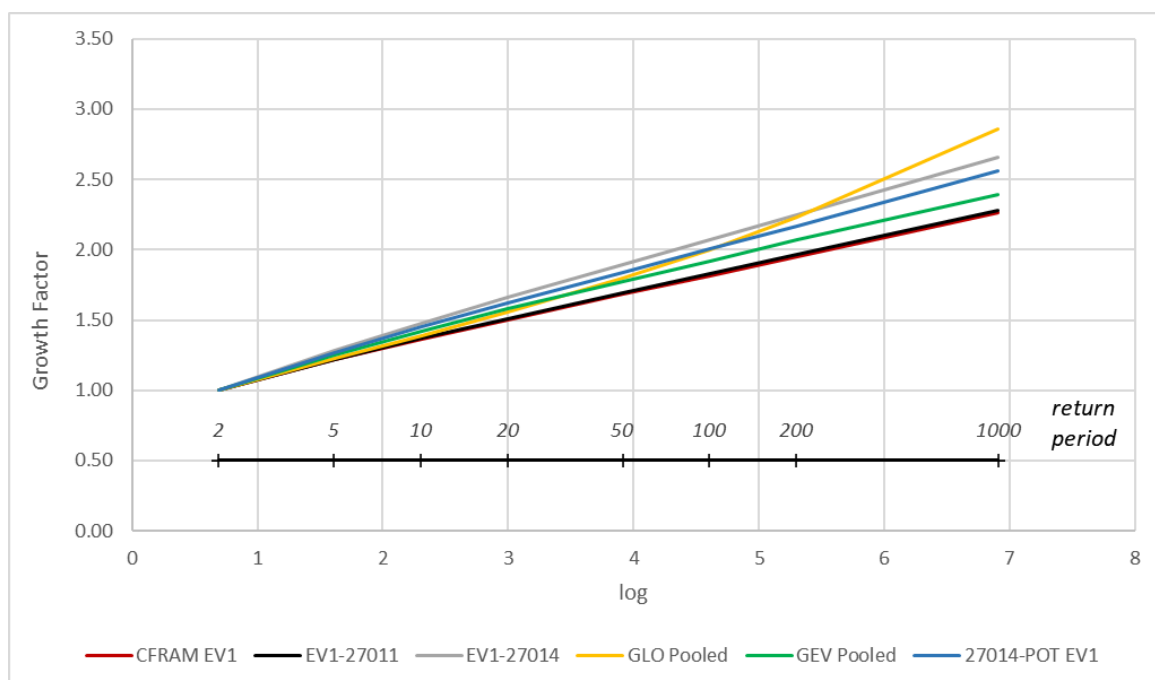


Figure 6-7: Growth Curves comparison – Owenogarney River

Table 6-5: Peak flow comparison – Owenogarney Br

Annual Exceedance Probability (%)	CFRAM EV1	EV1 - 27011	EV1 - 27014	GEV Pooled	GLO Pooled	EV1 27014 - POT
50% (2yr)	22.70	22.70	22.70	22.70	22.70	22.70
20% (5yr)	27.66	27.73	29.21	28.38	27.92	28.83
10% (10yr)	30.88	31.06	33.52	32.23	31.55	32.92
5% (20yr)	34.09	34.26	37.65	35.87	35.41	36.77
2% (50yr)	38.18	38.39	43.00	40.18	40.86	41.77
1% (100yr)	41.23	41.49	47.01	43.58	45.40	45.63
0.5% (200yr)	44.27	44.58	51.01	46.99	50.62	49.26
0.1% (1000yr)	51.31	51.73	60.26	54.25	64.92	58.11

6.1.8 Final peak flow estimates for Owenogarney River

The final peak flows calculated using the unadjusted QMED catchment descriptor FSU method with the GEV growth curve from the pooled analysis on the Owenogarney River are presented in Table 6-6.

Table 6-6: Final Peak Flow Estimates for Owenogarney River

	QMED	Annual Exceedance Probability (Return Period)							
		50% (2yr)	20% (5yr)	10% (10yr)	5% (20yr)	2% (50yr)	1% (100yr)	0.5% (200yr)	0.1% (1000yr)
Growth Curve		1	1.25	1.42	1.58	1.77	1.92	2.07	2.39
27_634_4	21.81	21.81	27.27	30.98	34.47	38.61	41.88	45.15	52.14
27_634_5	22.06	22.06	27.57	31.32	34.85	39.04	42.35	45.66	52.72
27_634_6	22.12	22.12	27.65	31.41	34.95	39.15	42.47	45.79	52.86
27_634_7	22.12	22.12	27.65	31.41	34.95	39.15	42.47	45.78	52.86
27_634_8	22.14	22.14	27.67	31.43	34.98	39.18	42.50	45.82	52.91
27_634_8a	22.27	22.27	27.83	31.62	35.18	39.41	42.75	46.09	53.22
27_634_9	22.24	22.24	27.80	31.58	35.14	39.37	42.70	46.04	53.16
27_634_10	22.43	22.43	28.04	31.85	35.44	39.70	43.06	46.43	53.61
27011	22.70	22.70	28.37	32.23	35.86	40.18	43.58	46.99	54.25

6.1.9 Hydrograph Shape - Owenogarney River

For the Owenogarney River, the hydrograph shape was generated using the recorded flows on 27011 and 27014 gauging stations. Five event records were available on both gauges, these are presented on Table 6-7. The hydrograph shape that will be adopted in this study is shown on Figure 6-8.

Table 6-7: Selected event hydrographs

Flood Event Date	Max Flow (m3/s) recorded on Owenogarney Br gauge	Max Flow (m3/s) recorded on Castle Lake gauge	Suitable for hydrograph generation
December 2015	30.34	29.1	partially
February 2014	22.65	21.1	partially
January 2018	22.70	21.3	partially
February 2020	25.26	26.9	partially
January 2023	21.78; 15.41	20.2; 12.2	yes

*Note. Two events in January 2023 have been used to help derive a hydrograph shape. One of these events has a much lower peak discharge, but the use of recent data helps to ensure the hydrograph reflects the current catchment conditions and response.

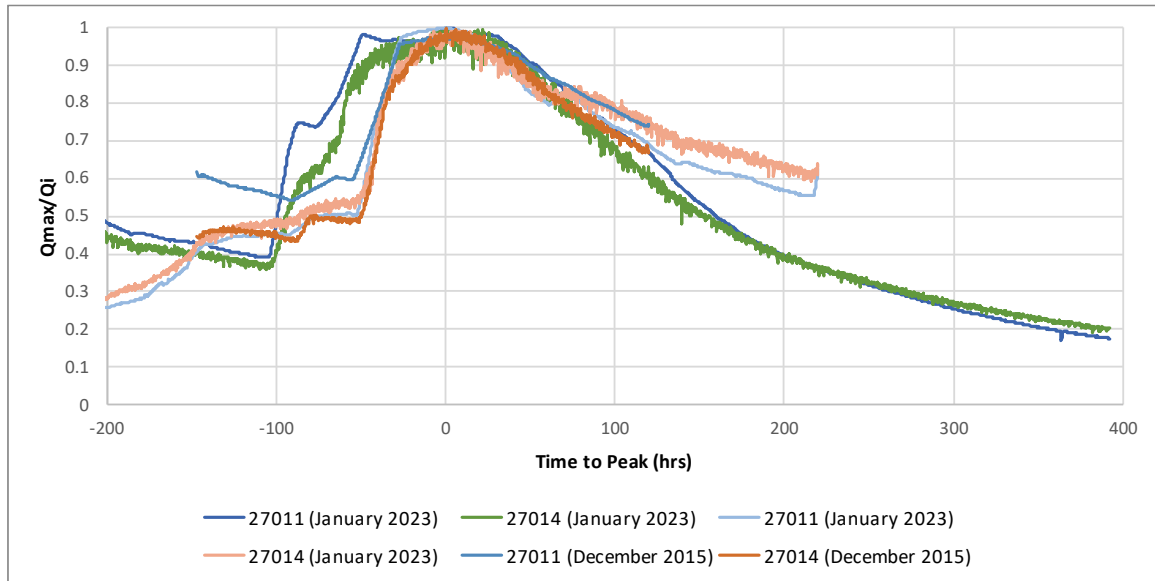


Figure 6-8: Event Hydrographs recorded at 27011 and 27014

6.2 Unnamed Tributary Flow Estimation

Two flow estimation points were considered for the unnamed tributary: one upstream of the tributary and one check-flow point downstream before discharging to the Owenagarney River. The catchment is very small, less than 2km², and therefore the typical FSU methods are generally not recommended, however it will be used for comparison with other methods. To estimate the peak flows for the unnamed tributary catchment, the following methods were considered:

- Flood Studies Update Method (FSU)
- Flood Studies Update Small Catchment Method (FSU SC)
- Flood Studies Report Rainfall-Runoff (FSR RR)
- Institute of Hydrology Report 124 Method (IH 124)

Background theory and descriptions for each method can be found in Appendices C of this Hydrology report.

6.2.1 Flow estimation summary for unnamed tributary

The estimated QMEDs for the two Hydrological Estimation Points on the tributary are shown in Table 6-8. It is recommended that the growth curve generated for the Owenagarney River be used on the small tributary.

Table 6-8: QMED estimates for the Unnamed Tributary (m³/s)

HEP Label	FSU	FSU SC	FSR RR (50% AEP)	IH124
27_1320_1	0.55	0.88	0.85	1.11
27_1320_2	0.60	0.90	1.04	1.41

The IH124 Method should only be considered for very small catchments, such as those below 5km². Generally best suited to small rural or urban catchments, which fits well with the tributary catchment in this study. For FSU SC, the acceptable range of catchment sizes for application of this method is between 1-25km². The FSR RR Method generates steep growth curves which are generally suited to smaller upland catchments where the flow regime is very flashy. The catchments in this study are extremely steep and are suitable for use of the FSR RR method. It is noted that the FSR RR method, like IH124 generates a value of QBAR not QMED and a scaling factor (0.94) is applied to obtain the correct value. All of the above methods fit well for the catchment; however it is noted that IH124 QMED produces the largest values. Given the steep nature of the catchment the FSR RR is the recommended method for the tributary along with the FSR RR generated growth curve. The final flows are presented in Table 6-9. The increase and decrease peak flow and volume will be further tested in the hydraulic model.

Table 6-9: Final Peak Flow Estimates for the unnamed tributary

FSRRR Growth Curve	Annual Exceedance Probability (Return Period)							
	50% (2yr)	20% (5yr)	10% (10yr)	5% (20yr)	2% (50yr)	1% (100yr)	0.5% (200yr)	0.1% (1000yr)
	1	1.26	1.42	1.57	1.76	1.89	2.02	2.3
27_1320_1	0.85	1.07	1.21	1.33	1.50	1.61	1.72	1.96
27_1320_2	1.04	1.31	1.48	1.63	1.83	1.97	2.10	2.39

6.2.2 Hydrograph Shape – Unnamed Tributary

For the ungauged tributary the hydrograph has been generated using the FSR method. This method comprises of the FSR triangular unit hydrograph/design storm method and estimates the design flood hydrograph, describing the timing and magnitude of flood peak and flood volume (area beneath hydrograph). This method requires the catchment response characteristics (time to peak, Tp), design rainstorm characteristics (return period, storm duration, rainfall depth and profile) and runoff / loss characteristics (percentage runoff and baseflow). The UK Natural Environmental Research Council (1975) carried out a comprehensive flood study involving a large number of catchments from throughout Britain including many Irish catchments. The unit hydrograph prediction equation was derived from 1,631 events from 143 gauged catchments (the hydrograph method only

included one Irish catchment) ranging in size from 3.5 to 500km². The final hydrographs of the unnamed tributary are shown below (Figure 6-9).

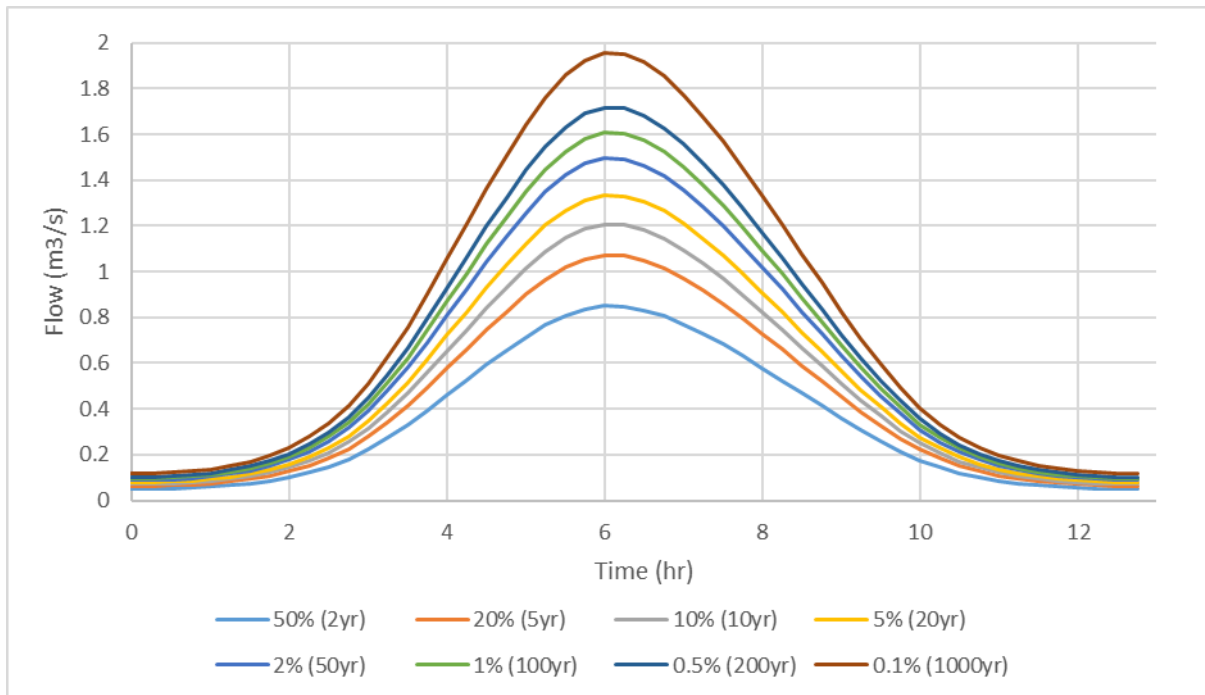


Figure 6-9: Ungauged tributary Hydrograph shape

6.3 Joint Probability

The downstream extent of the hydraulic model and study area is the Shannon Estuary. Therefore fluvial-tidal joint probability must be considered. Table 6-10 shows the proposed fluvial – tidal combinations to be considered in assessment and for sensitivity testing. Table 6-11 shows the tidal peak levels from ICWWS to be applied in the hydraulic model (South West Point S25). The hydraulic model will be calibrated to the gauged data.

Table 6-10: Fluvial-tidal joint probability combinations

Fluvial %AEP event	Tidal % AEP event
All design runs	50% AEP
10% AEP	10% AEP
1% AEP	10% AEP
10% AEP	1% AEP
10% AEP	0.5% AEP

Table 6-11: ICWWS Peak Tidal Levels for Owenogarney Catchment (S25) - OD Malin OSGM15 in meters⁴

AEP	Present Day	MRFS	HEFS
50%	3.46	3.96	4.46
20%	3.60	4.10	4.60
10%	3.71	4.21	4.71
5%	3.81	4.31	4.81
2%	3.94	4.44	4.94
1%	4.04	4.54	5.04
0.5%	4.14	4.64	5.14
0.1%	4.37	4.87	5.37

⁴ Taken from <https://www.floodinfo.ie/> coastal maps on 28 July 2023.

7 Climate Change

The relevant climate change factors have been applied to the final estimated peak flows as outlined in the tender brief for the project for the Medium Range and High-End Future Scenario (MRFS and HEFS, refer to Table 7-1). There is no change to the approach in CFRAM in applying these factors. Changes in forestation will be undertaken through adjustments to the Tp parameter for the unnamed tributary catchment. The FSU Urban Adjustment Factors for the study HEPs are all lower than 1.05 and therefore it is assumed that future changes in the urban extent will not have a significant effect on the catchment runoff response. The proportion of the forest in the study catchment is less than 25%, therefore it was assumed that changes to flow due to afforestation would not be significant. The full set of climate change flow estimates is presented in Appendix D. The tidal boundary in the hydraulic model will be set to the tidal peak levels from ICWWS (S25) as shown in Table 6-11.

Table 7-1: Climate change factors

	MRFS	HEFS
Extreme Rainfall	+20%	+30%
Flood flows	+20%	+30%
Mean sea level Rise	+500mm	+1000mm
Land Movement	-0.5mm/year	-0.5mm/year
Urbanisation	Review case-by-case	Review case-by-case
Forestation	-1/6 Tp	-1/3 Tp +10% SPR

8 Summary and Conclusion

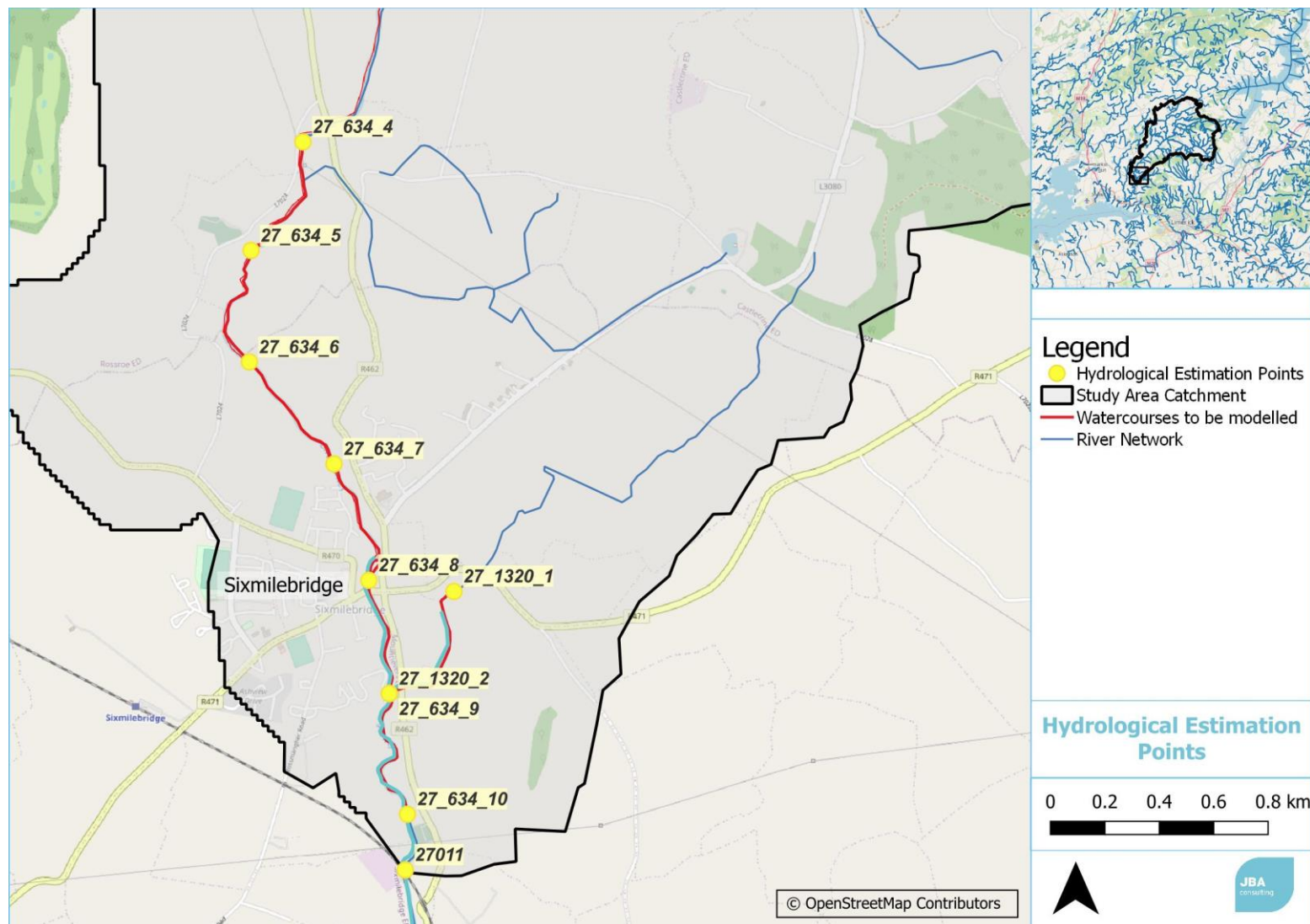
From the hydrological analysis, the estimated peak flows for the Owenogarney River are based on the Flood Study Update Method and using the GEV growth curve from pooled analysis. The hydrograph shape is based on the recorded peak flow data available on Owenogarney Br (27011) and Castle Lake (27014) gauges. For the unnamed tributary the recommended peak flow estimates and hydrographs are based on the FSR Rainfall Runoff Method.

The generated hydrographs will be used as inputs to the hydraulic model for Sixmilebridge, Co Clare to produce the hazard maps for the area. The downstream boundary will be set to the tidal peak levels from ICWWS from South West Point (S25).

The validation of the hydraulic model will need to consider whether attenuation of flood flows and hydrograph shape has been double counted in the hydrological model and hydraulic model. This will require expert judgement as it is not possible to automatically separate fluvial and tidal flows at the downstream Owenogarney gauge, and also there are insufficient flood events recorded at either gauge for reasonable average hydrograph shapes to be derived without combining the two.

Appendices

A Appendix – Hydrological Estimation Points



B Physical Catchment Descriptors at Hydrological estimation Points

Descriptor	Owenogarney River									Unnamed Tributary	
	27_634_4	27_634_5	27_634_6	27_634_7	27_634_8	27_634_8a	27_634_9	27_634_10	27011 (Owenogarney Br)	27_1320_1	27_1320_2
AREA (km ²)	154.86	156.95	157.89	158.23	158.71	158.90	160.58	161.13	161.25	1.38	1.67
SAAR (mm)	1248.31	1246.39	1245.13	1244.70	1244.01	1244.01	1243.68	1240.96	1240.96	1244.01	1244.01
FARL	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.00	0.00
BFI Soil	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.65
URBEXT	0.0011	0.0011	0.0012	0.0013	0.0028	0.0028	0.0041	0.0046	0.0046	0.00	0.02
MSL (km)	29.24	29.74	30.25	30.75	31.24	31.24	31.74	32.25	32.47	2.22	2.77
S1085 (m/km)	8.88	8.52	8.36	8.15	8.00	8.00	7.75	7.67	7.67	17.84	18.22
DrainD (km/km ²)	1.14	1.15	1.15	1.15	1.15	1.16	1.15	1.15	1.19	1.61	1.11
NETLEN	176.56	180.44	180.94	181.44	181.94	184.71	185.21	185.71	192.08	2.22	2.77
ArtDrain2	0.00	0.00	0.00	0.00	0.0004	0.0004	0.0035	0.0065	0.0065	0.00	0.00

C Description of the Hydrological Estimation Methods

C.1 The Flood Study Update method

The Flood Studies Update (FSU) method to estimate QMED as described in research reports produced from FSU work packages 2.2 and 2.3, has been used. QMED can be estimated using a regression equation based on seven different physical catchment descriptors, in conjunction with an urban adjustment, developed in FSU work package 2.3.

The multivariate regression equation was developed on the basis of data from 199 gauged catchments, linking QMED to a set of catchment descriptors.

$$Q_{medRURAL} = (1.237 \times 10^{-5})(AREA^{0.937})(BFISoil^{-0.922})(SAAR^{1.306})(FARL^{2.217})(DRAIN^{0.341})(S1085^{0.185})((1 + ARTDRAIN2)^{0.408})$$

Where:

AREA is the catchment area (km²).

BFISoils is the base flow index derived from soils data

SAAR is long-term mean annual rainfall amount in mm

FARL is the flood attenuation by reservoir and lake

DRAIN is the drainage density

S1085 is the slope of the main channel between 10% and 85% of its length measured from the catchment outlet (m/km).

ARTDRAIN2 is the percentage of the catchment river network included in the Drainage

C.1.1 The FSU small catchments method

The FSU small catchments method was created as part of FSU WP 4 and is discussed in 'WP 4.2 - Flood Estimation in Small and Urbanised Catchments'. It is designed for catchments smaller than 25 km².

The FSU small catchment equation is a 5 variable regression equation that was developed after the examination of multiple small catchments equations and regression analysis of multiple catchment descriptors. The FSU small catchment equation for QMED is:

$$Q_{med} = (2.0951 \times 10^{-5}) * (AREA^{0.9245}) * (SAAR^{1.2695}) * (BFI^{0.9030}) * (FARL^{2.3163}) * (S1085^{0.2513})$$

The urban extent can be taken into account using the same method as above for the FSU standard method.

C.1.2 Polled Growth Curve

For pooled analysis within the FSU, gauges are chosen on the basis of their similarity with the subject catchment according to three catchment descriptors, i.e. AREA, SAAR and BFIsoil. The report on FSU WP 2.2 presents two alternative equations for calculating the similarity of catchments according to these three descriptors. For this study, equal weight was given to each of these variables, applying the similarity distance formula given as Equation 10.2 in the report on FSU WP 2.2.

Not all gauges in Ireland were considered for use in pooling, because the analysis required to fit a flood growth curve makes use of the magnitude of each annual maximum flow, and thus it is necessary that even the highest flows are reliably measured. This excludes gauges where there is significant uncertainty in the high flow rating.

The final pooling group used is shown in table below. No alterations were made to the pooling groups defined using the process defined above as it was deemed appropriate for the analysis. Gauging stations had already been screened according to the quality of their flood peak data, as described above. Although there is some evidence from research on UK data⁵ that flood growth curves are affected by additional catchment descriptors such as FARL, the FSU research found that FARL was not a useful variable for selection of pooling groups (uncertainty was greater when FARL

⁵ Kjeldsen, T.R., Jones, D.A. and B06031ayliss, A.C. (2008) Improving the FEH statistical procedure for flood frequency estimation. Science Report SC050050, Environment Agency.3020

was included than when it was excluded) and therefore no attempt was made to allow for the presence of lakes in the composition of pooling groups. Similarly, no allowance was made for arterial drainage in selecting pooling groups.

For pooled growth curves, WP 2.2 recommends considering 3-parameter distributions, because the extra data provided by the pooling group ensures that the standard error is lower than it would be for single-site analysis. The report states that either the generalised extreme value (GEV) or generalised logistic (GL) distributions are worth considering. For this study, GEV has been fitted for the pooled analysis.

Table A-1: Pooling Group used in the FSU analysis

Rank	Station No.	Name	Watercourse	Years	Classification	AREA	SAAR	BFI _{SOIL}	FARL	Excluded
1	19001	Ballea	Owenboy	59	NA	103.292	1175.67	0.6399351	1	No
2	19020	Ballyedmond	Owennacurra	40	A2	73.9548	1179.07	0.664	1	No
3	25044	Coole	Kilmastulla	51	A2	92.5464	1186.86	0.583	0.997	No
4	30002	Ower Br. (Black[Shrule])	Black [Shrule]	44	NA	193.566	1143.04	0.5742748	0.992	No
5	29071	Cutra	L. Cutra	40	A2	123.8396	1212.48	0.551	0.804	No
6	30005	Foxhill	Robe	61	A1	237.82	1172.46	0.5637	0.985	No
7	16003	Rathkennan	Clodiagh	39	A2	243.2024	1192.01	0.5495	0.999	No
8	25002	Barringtons Bridge	Newport	64	A2	221.6092	1299.96	0.542	0.999	No
9	16005	Aughnagross	Multeen	43	A2	84.0004	1153.57	0.56	1	No
10	29001	Rathgorgin	Raford	53	A1	115.4796	1089.74	0.581	0.998	No
11	16006	Ballinaclogh	Multeen	46	B	75.7964	1115.82	0.5868	0.994	No
12	25030	Scarriff Bridge	Graney	61	A2	280.0208	1183.81	0.5415	0.85	Yes
13	26006	Willsbrook	Suck	66	A1	184.764	1120.64	0.5404	0.97	Yes
14	34024	Kiltimagh	Pollagh	36	A2	127.2268	1177.46	0.521	0.922	Yes
15	29011	Kilcolgan	Dunkellin	35	A1	354.1404	1079.37	0.6332	0.976	Yes
16	30007	Ballygaddy	Clare	44	A2	469.896	1115.06	0.6455	0.989	Yes
17	26010	Riverstown	Cloone	48	B	94.5268	1064.26	0.578	0.937	Yes
18	27002	Ballycorey	Fergus	64	A1	564.2711	1336.35	0.697	0.835	No
19	06070	Muckno	Muckno L	11	A1	162.0238	1046.28	0.83	0.733	Yes
20	35005	Ballysadare	Ballysadare	69	A2	639.656	1198.32	0.6086	0.898	No
21	25027	Gourdeen Bridge	Ollatrim	51	A1	118.8648	1021.15	0.6511	1	Yes

C.2 The FSR Rainfall Runoff Method

The FSR Statistical method is widely used in Ireland and the UK for ungauged catchments is the FSR triangular unit hydrograph and design storm method. This method estimates the design flood hydrograph, describing the timing and magnitude of flood peak and flood volume (area beneath hydrograph). This method requires the catchment response characteristics (time to peak, t_p), design rainstorm characteristics (return period, storm duration, rainfall depth and profile) and runoff / loss characteristics (percentage runoff and baseflow).

The unit hydrograph prediction equation was derived from 1,631 events from 143 gauged catchments (the hydrograph method only included one Irish catchment) ranging in size from 3.5 to 500km². The result was a triangular Unit Hydrograph described by the time to peak T_p of the catchment derived from catchment characteristics. The instantaneous triangular unit hydrograph is defined by a time to peak T_p , a peak flow in cumecs/100km² $Q_p = 220/T_p$ and a base length $T_B = 2.52T_p$.

The FSR rainfall-runoff method relies on rainfall frequency statistics to provide inputs to a model that converts rainfall to runoff. The rainfall-runoff model separates a flood hydrograph into a baseflow component and a rapid runoff component. The rapid runoff is found by estimating the component of rainfall that contributes to runoff (the effective rainfall) and converting the effective rainfall to flow by use of a unit hydrograph. The unit hydrograph describes the theoretical response of the catchment to an input of a unit depth of rainfall over a unit of time.

The steps in the model are:

- Determine the parameters of the unit hydrograph, either from flood event data or from catchment characteristics;
- Determine the percentage runoff to convert total rainfall to effective rainfall;
- Construct the design storm by determining its duration, depth and profile;
- Combine the effective rainfall profile with the unit hydrograph by convolution to give the flood hydrograph;
- Add baseflow to the flood hydrograph

The (T_p) equation and the calculation of percentage runoff (PR) use the following equations:

$$T_p = 283 S_{1085-0.33} SAAR^{-0.54} MSL^{0.23} \text{ (eqn 1)}$$

$$\text{and PR} = SPR + DPR_{CWI} + DPR_{RAIN} \text{ (eqn 2)}$$

where

MSL is the mainstream length (km)

$$SPR = 10S_1 + 30S_2 + 37S_3 + 47S_4 + 53S_5$$

S_1 to S_5 are the catchment fractions covered by the five winter rainfall acceptance potential (WRAP) classes and S_u is the unclassified fraction which is covered either by standing water or a paved area.

$DPR_{CWI} = 0.25(CWI - 125)$ and CWI = catchment wetness Index which is a function of SAAR.

$DPR_{RAIN} = 0.45(R - 40)^{0.7}$ for storm depth $R > 40\text{mm}$ and $= 0$ for $R < 40\text{mm}$

The design rainstorm duration is obtained from the FSR formula $D = (1 + 0.001SAAR) T_p$. Using the prescribed FSR rules for computing the storm duration, profile and percentage runoff a 140year return period design storm is required to produce the 100year design flood.

The FSR RR growth curve with initial DDF values from Met Eireann were applied to HEP catchments that falls within 60mOD.

C.3 Institute of Hydrology Report 124 Method

The IH 124 Report examined the response of small catchments, less than 25km², to rainfall and derived an improved flood estimation equation (Marshall & Bayliss, 1994). A total of 87 sites were used to develop the method. The report developed a new equation to estimate the mean annual flood, QBAR (in m³/s), for small rural and urban catchments.

$$QBAR_{Rural} = 0.00108 AREA^{0.89} SAAR^{1.17} SOIL^{2.17}$$

And

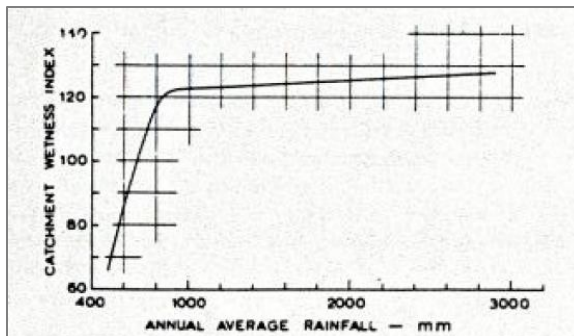
$$QBAR_{Urban}/QBAR_{Rural} = (1 + URBAN)^{2NC} [1 + URBAN\{(21/CIND) - 0.31\}]$$

$$\begin{aligned} \text{Where: } NC &= 0.92 - 0.00024SAAR, \text{ for } 500 \leq SAAR \leq 1100\text{mm,} \\ NC &= 0.74 - 0.000082SAAR, \text{ for } 1100 \leq SAAR \leq 3000\text{mm, and} \\ CIND &= 102.4SOIL + 0.28 (CWI - 125), \\ SOIL &= \frac{(0.10S_1 + 0.30S_2 + 0.37S_3 + 0.47S_4 + 0.53S_5)}{S_1 + S_2 + S_3 + S_4 + S_5} \end{aligned}$$

Where: NC is rainfall continentality factor, CIND is a catchment index defined as a function of SOIL and catchment wetness index (CWI), both as in FSR Method⁶, and QBAR (m³/s), AREA (km²), and SAAR (mm).

The QBAR computed has an estimated return period of 2^{3/1} years.

CWI is derived from SAAR, the recommended CWI values for standard design conditions are shown in figure below.



SOIL value is based on the percentage of the catchment within each WRAP (Winter Rain acceptance potential) soil class. The fraction of each soil type (S₁, S₂, etc) is given a weighting and the soil index is the weighted mean.

⁶ The Flood Studies Report (FSR), published by Natural Environment Research Council - 1975

D Climate Change Flow Estimates

MRFS	Annual Exceedance Probability (Return Period)							
	50% (2yr)	20% (5yr)	10% (10yr)	5% (20yr)	2% (50yr)	1% (100yr)	0.5% (200yr)	0.1% (1000yr)
27_634_4	26.18	32.72	37.17	41.36	46.33	50.26	54.19	62.56
27_634_5	26.47	33.09	37.59	41.82	46.85	50.82	54.79	63.26
27_634_6	26.54	33.18	37.69	41.94	46.98	50.96	54.94	63.44
27_634_7	26.54	33.18	37.69	41.94	46.98	50.96	54.94	63.44
27_634_8	26.56	33.21	37.72	41.97	47.02	51.00	54.99	63.49
27_634_8a	26.72	33.40	37.94	42.22	47.30	51.30	55.31	63.86
27_634_9	26.69	33.36	37.90	42.17	47.24	51.24	55.25	63.79
27_634_10	26.92	33.64	38.22	42.53	47.64	51.68	55.71	64.33
27011	27.24	34.05	38.68	43.04	48.21	52.30	56.38	65.10
27_1320_1	1.02	1.28	1.45	1.61	1.81	1.96	2.11	2.44
27_1320_2	1.25	1.56	1.77	1.97	2.21	2.40	2.58	2.98
HEFS	Annual Exceedance Probability (Return Period)							
	50% (2yr)	20% (5yr)	10% (10yr)	5% (20yr)	2% (50yr)	1% (100yr)	0.5% (200yr)	0.1% (1000yr)
27_634_4	28.36	35.45	40.27	44.81	50.19	54.45	58.70	67.78
27_634_5	28.68	35.84	40.72	45.31	50.76	55.06	59.36	68.53
27_634_6	28.75	35.94	40.83	45.43	50.90	55.21	59.52	68.72
27_634_7	28.75	35.94	40.83	45.43	50.89	55.21	59.52	68.72
27_634_8	28.78	35.97	40.87	45.47	50.94	55.25	59.57	68.78
27_634_8a	28.95	36.18	41.11	45.74	51.24	55.58	59.92	69.18
27_634_9	28.91	36.14	41.06	45.68	51.18	55.52	59.85	69.10
27_634_10	29.16	36.45	41.40	46.07	51.61	55.98	60.36	69.69
27011	29.51	36.89	41.90	46.62	52.23	56.66	61.08	70.53
27_1320_1	1.11	1.38	1.57	1.75	1.96	2.12	2.29	2.64
27_1320_2	1.35	1.69	1.92	2.14	2.39	2.60	2.80	3.23

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