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LIFEDeeRiver
Restoration of Freshwater Features

Concept Design - Modelling Report

Natural Resources Wales

March 2021

FINAL

Quality information

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1. Introduction

1.1 Overview

This report summarises modelling that has been undertaken in support of the LIFEDeeRiver project. The overall project contributes to a number of the LIFE project's key aims, including improving longitudinal connectivity for fish and restoring or improving natural physical processes, features and habitats. Specifically, investigation into fish passage solutions at six obstructions (weirs) were required as part of the Restoration of Freshwater Features project: Horseshoe Falls, Llangollen Upstream and Downstream, Morlas Ford, Erbistock and Chester. At the start of the project, Natural Resources Wales (NRW) identified a number of preferred solutions for fish passage:

- Horseshoe Falls weir: Nature-like by-pass channel on right hand bank (RHB);
- Llangollen Upstream weir: Creation of at least three notches at bed level within the weir crest (~8-10 m wide);
- Llangollen Downstream weir: Remove remains to bed level and create a natural river channel;
- Morlas Ford weir: Ford removal and river channel restoration, access to the opposite bank via a clear span bridge;
- Erbistock weir: Partial removal to bed level of ≥ 50% of the weir's width;
- Chester weir: Either: 1) Improvements to the existing fish pass wall and notch in crest for smolt passage downstream; 2) Notch the weir crest for downstream smolt passage, or; 3) Bypass channel on the left hand bank (LHB).

The objectives of the current study were to collate pre-construction information from a variety of environmental disciplines for each of the six weirs, including ecology, geomorphology, hydrology, heritage, topographic survey, utilities and contaminated land to assess the preferred options at each site, determine a recommended option and produce conceptual designs.

1.2 Report Structure

A standard modelling methodological approach was subsequently applied to each weir and is described in Section 2. Sections 3 to 8 detail the modelling undertaken at each weir itself.

2. Modelling Methodology

2.1 Background

The River Dee is designated as Special Area of Conservation (SAC) due to it harbouring notable fish species including Atlantic salmon (*Salmo salar*) sea lamprey (*Petromyzon marinus*), river lamprey (*Lampetra fluviatilis*), brook lamprey (*Lampetra planeri*) and bullhead (*Cottus gobio*). Other species of conservational interest include sea/brown trout (*Salmo trutta*) and European eel (*Anguilla anguilla*). However, the river has been historically modified and among other features, it has 14 weirs along the flow path considered to impact fish passage. Consequently, some protected species and habitats have been categorised as unfavourable-bad or unfavourable-inadequate. To improve longitudinal connectivity for fish and restore or improve natural physical processes, features and habitats, specific preferred restoration options have been selected for six weirs. Figure 2.1 show the location of the six weirs selected.

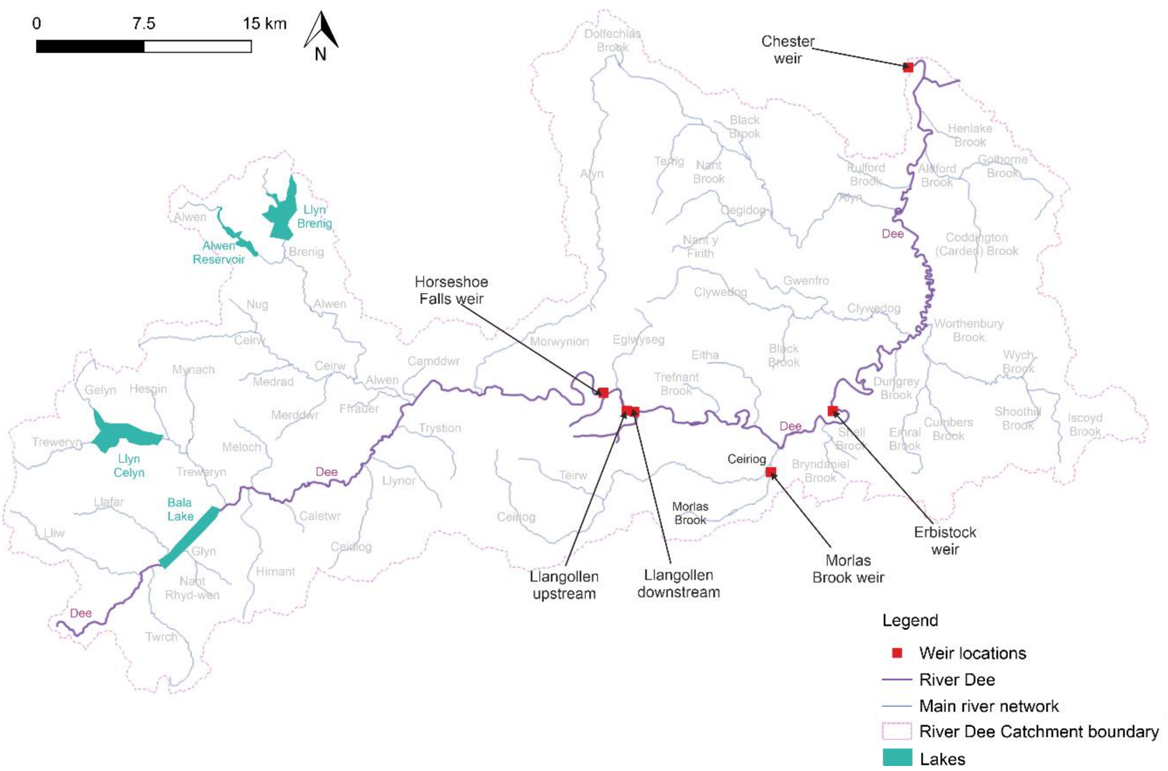

Figure 2.1 Location of weirs selected to develop Preferred Restoration Options

2.2 Modelling Tools

2.2.1 Hydraulic Modelling

To simulate the baseline and preferred option scenario, a TUFLOW 2D hydraulic model was developed for each weir, covering the local study area and suitable lengths of river upstream and downstream such that the full impact of the options would be included in the models. In order to construct the hydraulic models, two key inputs are required: a Digital Elevation Model (DEM) and hydrological inputs.

The DEM for the models were constructed by integrating the topographic survey data collected for the project into freely available LiDAR data, which was used to represent the wider model extent not covered by topographic surveys. To represent the channel profile, the topographic point data was interpolated to create Triangular Irregular Networks (TINs). The TIN surfaces were reviewed against LiDAR data, the topographic survey sections and aerial imagery to ensure good representation of channel morphology. When necessary, interpolated points between topographic survey cross sections were integrated into the TIN until the surface performance was considered suitable. The remaining areas of the model surface were represented by freely available 1 m resolution LiDAR data. To represent the preferred option scenario, modifications on the baseline DEM were developed to represent the options in the model surface. Adjustments were carried out by using GIS and TUFLOW software until the scenario performance at each weir was considered suitable.

Hydrological inputs for the model were calculated by using data from the EA network of hydrological monitoring points throughout the River Dee catchment. For each model, the following events were calculated:

- Flow events:
  - Q_{95};
  - Q_{50}; and
  - Q_{10}
- Flood events:
  - 1 in 2 years; and
  - 1 in 100 years plus 30% Climate Change

Baseline and preferred options were then run for each event. To evaluate if the hydraulic design of the preferred option allows passage for the fish species and to examine if there were any potentially issues associated with the proposed options, water depth, velocity and shear stress results were assessed.

Scenario Modelling

For each of the weirs, the baseline situation and the preferred restoration options scenarios were modelled. The preferred restoration options were defined after discussion with NRW. Table 2.1 shows the Grid Reference and scenarios modelled for each weir.

Table 2.1 Preferred Restoration Options selected for each weir.

Weir	NGR	Scenarios
Horseshoe Falls	SJ 19541 43352	• Baseline • Nature-like by-pass channel on right
Llangollen Upstream	SJ 21367 42132	• Baseline • Creation of one notch 3m wide and 0.3m deep
Llangollen Downstream	SJ 21594 42108	• Baseline • Partial removal
Morlas Brook	SJ 31190 38319	• Baseline • Ford removal and river channel restoration, access to the opposite bank via a clear span bridge
Erbistock	SJ 35444 42161	• Baseline • Partial removal to bed level of ≥ 50% of the weir's width
Chester	SJ 40823 65854	• Baseline • Creation of two notches 2m wide and 0.3m deep

2.2.2 Flow Estimates at the Weirs

Typical Flows

Flow statistics for each of the weirs have been estimated for five of the weirs (Horseshoe Falls, Llangollen Upstream and Downstream, Erbistock, Chester), and using LowFlows software for the ungauged Morlas Brook. These were then used in the hydraulic modelling.

The six weirs are outlined in Table 2.2 below along with information detailing the respective catchment sizes, closest flow gauging station and rationale for flow statistic estimates to be used. Flow estimates for the sites are provided in Table 2.3.

Table 2.2 Fish pass sites: Grid reference, catchment size and flow statistics estimation

Site	Grid Reference	Catchment size at fish pass[1] (km^2)	Flow statistic estimate for this gauge
Horseshoe	SJ1954143352	752.7	The site is reasonably close to the Manley Hall (River Dee) flow gauge and so statistics at the site can be estimated from the gauge
Llangollen Upstream	SJ2136742132	785.4	The site is reasonably close to the Manley Hall (River Dee) flow gauge and so statistics at the site can be estimated from the gauge
Llangollen Downstream	SJ2159442108	785.8	The site is close to the upstream weir and those values estimated for that site will be used here.
Morlas	SJ3119038319	21.1	Flow statistics to be determined through LowFlow Estimate software
Erbistock	SJ3544442161	1,033.5	The site is close to the Manley Hall (River Dee) flow gauge and so statistics at the site can be estimated from the gauge
Chester	SJ4082365854	1,801.0 (site is actually downstream of the gauge though catchment size reported by FEH is lower than that reported at the gauge on the NRFA)	This site is very close to the gauge and the statistics essentially apply to this site too.

Table 2.3 Flow (statistic) estimates for each fish pass site

Flow Statistic	Flow at Fish Pass sites (m^3/s)					
	Horseshoe Falls	Llangollen Upstream	Llangollen Downstream	Morlas (from LowFlow software)	Erbistock	Chester
Q_{99}	5.6	5.9	5.9	0.04	7.7	4.5
Q_{95}	6.3	6.6	6.6	0.06	8.7	4.9
Q_{70}	8.6	8.9	8.9	0.12	11.8	9.0
Q_{50}	14.5	15.1	15.1	0.19	19.9	18.5
Q_{30}	25.9	27.0	27.0	0.32	35.5	37.5
Q_{15}	42.4	44.2	44.2	0.56	58.2	70.9
Q_{10}	52.4	54.7	54.7	0.74	72.0	91.0
Q_5	69.1	72.1	72.1	1.06	94.9	120.6

[1] UK Centre for Ecology and Hydrology. Flood Estimation Handbook - https://fehweb.ceh.ac.uk/GB/map

Flood Flows

Flood flows for each of the weirs have been estimated for 1 in 2 year and 1 in 100 year plus 30% Climate Change return periods.

In agreement with NRW, return periods and flood flows were output by the Flood Estimation Handbook (FEH) software. This method was chosen due to the catchment and associated sites are applicable with the approaches. Further details on flood estimation derivation is detailed in Annexes 1 to 3. In the case of Morlas, due to the hydraulic model encompassing two watercourses, this method was applied at Afon Morlas to get the flood inflows for this watercourse. Flood inputs at Afon Ceiriog watercourse were subsequently calculated through catchment apportioning, acknowledging the relative catchment sizes. Flood peaks for 1 in 2 year and 1 in 100 year plus 30% Climate Change event hydrographs for each weir are shown in Table 2.4 below.

Table 2.4 Flood peak (m^3/s) for required return periods for each fish pass site

Location and Model	1 in 2 year peak flow (m^3/s)	1 in 100 year plus 30% CC peak flow (m^3/s)
Horseshoe Falls	191.1	518.5
Llangollen Upstream and Downstream	193.9	526.1
Morlas Brook (Afon Morlas)	5.1	21.5
Morlas Brook (Afon Ceiriog)	27.8	117.1
Erbistock	228.8	663.1
Chester	268.0	682.5

2.2.3 Other

Other sources of information were used as part of the modelling or to inform it. These included topographic surveys and site observations.

3. Horseshoe Falls Weir

3.1 Baseline

3.1.1 Site

Site around the weir is illustrated on Figure 3.1. It is situated on the River Dee in Denbighshire, north east Wales, approximately 2.3km north west of Llangollen town centre. The site comprises a weir spanning the majority of the width of the River Dee, falling short from the eastern bank to allow for the navigation of the Shropshire Union Canal Llangollen Branch. The site lies approximately 85m above ordnance datum (AOD).

The weir is a large horseshoe shaped structure that has an existing abstraction associated with it for the Llangollen canal. The weir is reported to be flow sensitive. The offtake of the canal is on the LHB and has existing screening in place to prevent fish entrainment. The weir is part of a World Heritage Site. It extends for approximately 145m, has a vertical downstream face with a head difference of approximately 1.5-2m.

At project onset the preferred fish passage option was a nature like bypass channel on the RHB.

The site is found in a river setting environment and flows from west to east, where the river then meanders and flows from north to south. To the west of the weir, adjacent to the River Dee, is an area of land, where a fish bypass channel could be constructed. The area of land is presumably used for agricultural purposes, specifically for pasture grazing. To the north of the site, land use is predominately areas of open field, presumably used for arable agriculture. A hotel and church are also found to the north. Downstream of the weir and to the east, an area of public open green space lies directly adjacent to the site. The B5103 is beyond this that eventually leads to a bridge that crosses the River Dee downstream from the site. An outdoor education centre is also present. To the south, the site is bounded by the A5 road that cuts through an extensive patch of woodland. Directly adjacent to the site is the Llangollen Railway, which spans the entirety of the southern site boundary at notably higher elevations than the river with a steep topography between the two.

The Canal and Rivers Trust (CRT) commissioned a review of the condition of the weir[2]. The findings were that the weir itself is in a fair condition, with minor uneven flow over the crest of the weir as a result of small voids under the breadth of the crest. Recommendations to improve the overall condition included repairing weir crest masonry, removing debris and vegetation caught on or adjacent to the crest and removing potentially unsafe platform attached to west of the training wall (the latter appears to have been actioned). While seemingly homogeneous from a distance, the review indicated that the weir is comprised of a number of components including metals plates, bars, flanges etc indicating that it has been modified and repaired since originally constructed (completed by 1808).

[2] Principal Inspection Report, LA-075-006 Horseshoe Falls Weir (ARCADIS 2018). Report Ref: UA007159-ARC-XX-XX-RP-CE-0006.

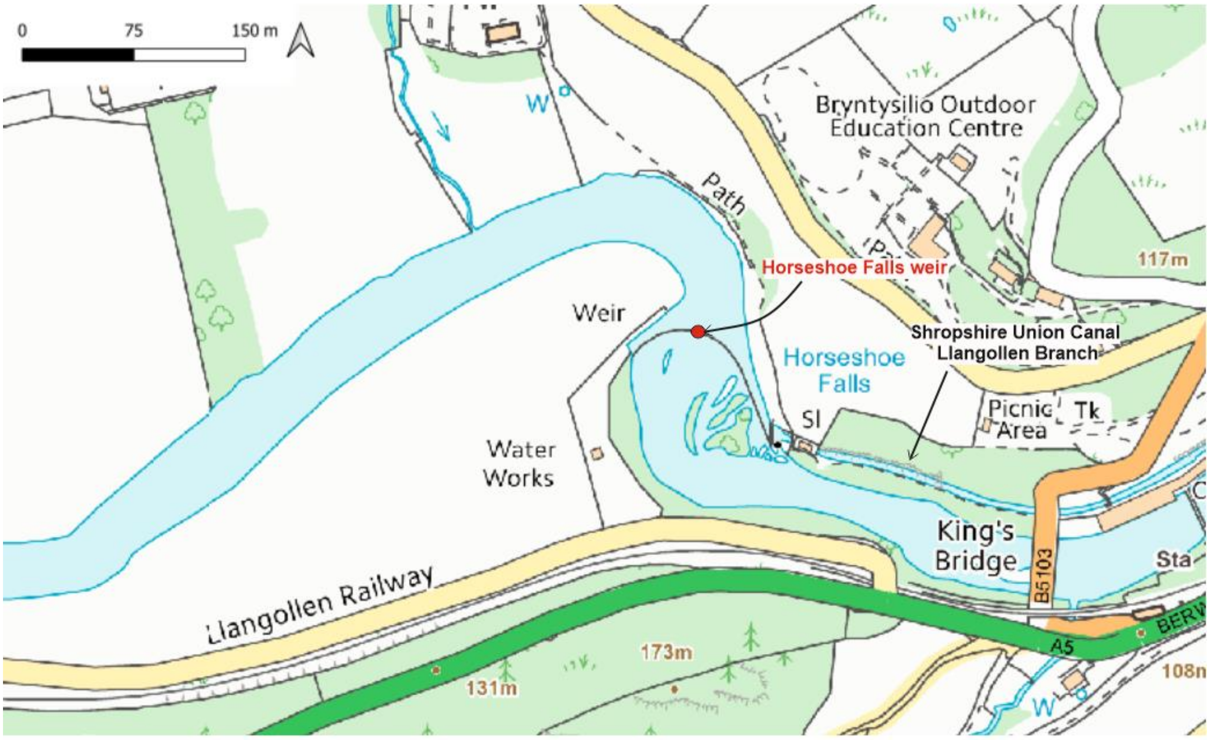

Figure 3.1 Horseshoe Falls weir study area

3.1.2 Site Visit

The site was visited on 16 July 2020. Images from this, complimented by imagery from the topographic survey, are presented in Plates 3-1 through to 3-20.

The historic value of the site was evident, particularly with a number of visitor information boards located in the vicinity of the weir.

CRT have an abstraction for Shropshire Canal just upstream of the weir (RHB). Existing screens are located between the main river channel and the canal feeder stream. At the start of the canal feeder stream there is a sluice that enables some flow to return to the river while flow that continues down the feeder stream must flow through a culvert under an ops building (which contains a flow monitor measuring flow into the canal). Maximum annual abstraction at the site amounts to 18,300 Ml/yr while the daily maximum abstraction amounts to 82.96 Ml/d (0.96 m^3/s).

United Utilities who are currently investigating the provision of new screens (to prevent fish and eel entrainment) on behalf of the CRT at the abstraction from the River Dee into the canal feeder stream, with the existing screens not complying to current regulations. It is understood that the current preferred screening option also entails notching the weir just downstream of these screens to help prevent trash building up at the screens.

On the opposite bank, Hafren Dyfrdwy own a building, believed to be a pump house, and a small plot of land through which a bypass could otherwise flow through.

Plate 3-1. Horseshoe Falls weir (looking from LHB)- panoramic view

Plate 3-2. Structure in channel at weir (one end of where stoplogs can be inserted to dry up channel to canal for when that is maintained)

Plate 3-3. Screens at entrance to canal feeder stream

Plate 3-4. LHB end of the weir/ screens at entrance to CRT on the left

Plate 3-5. Screens from canal feeder stream side

Plate 3-7. Screens and surrounding structures

Plate 3-8. Looking along the weir crest from LHB

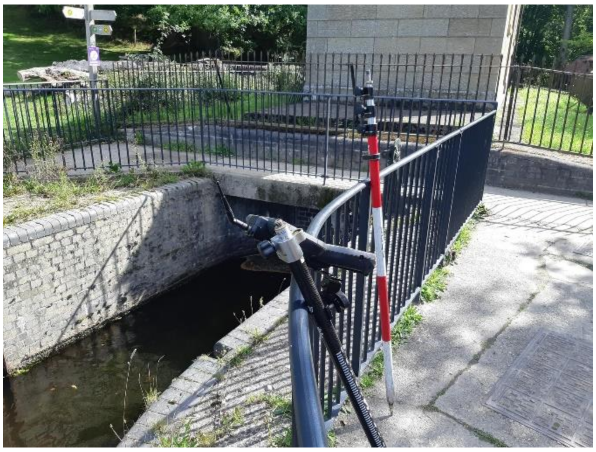

Plate 3-9. Building above the channel feeder stream, under which channel is culverted and flow is monitored

Plate 3-10. Start of canal feeder stream

Plate 3-11. Horseshoe Falls weir from RHB- panoramic view

Plate 3-12. Horseshoe Falls weir from RHB (central section including rock outcrop)

Plate 3-13. Horseshoe Falls weir from RHB

Plate 3-14. Downstream of Horseshoe Falls weir (RHB) looking upstream

Plate 3-15. RHB end of Horseshoe Falls weir

Plate 3-16. Downstream of Horseshoe Falls weir (RHB) looking across the channel- panoramic view

Plate 3-17. Downstream of Horseshoe Falls weir (RHB) looking downstream

Plate 3-18. Deposited materials downstream of Horseshoe Falls weir (RHB)

Plate 3-19. Field through which bypass channel may flow (looking towards the west/ general upstream direction)

Plate 3-20. Hafren Dyfrdwy land and pump house

3.1.3 Topographic and Bathymetric Survey

A topographic survey of the site and surrounding area was undertaken. This included a survey at and around the weir, cross sections in the River Dee upstream and downstream extending approximately 500m and 350m from the weir, respectively. The survey was undertaken to refine representation of the system within the hydraulic modelling (see section 3.2.3).

All surveyed points are indicated in Figure 3.2 below while those around the weir itself are indicated in Figure 3.3, which shows a reasonable coverage with some gaps where surveying could not be undertaken safely.

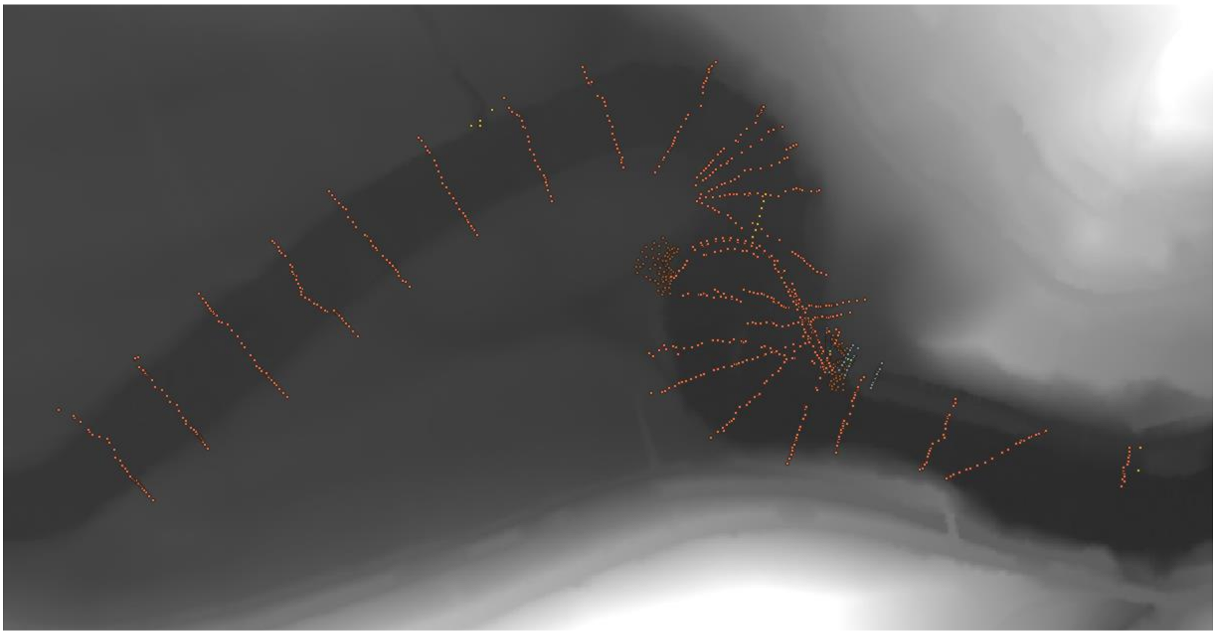

Figure 3.2 Horseshoe Falls Weir xyz data received

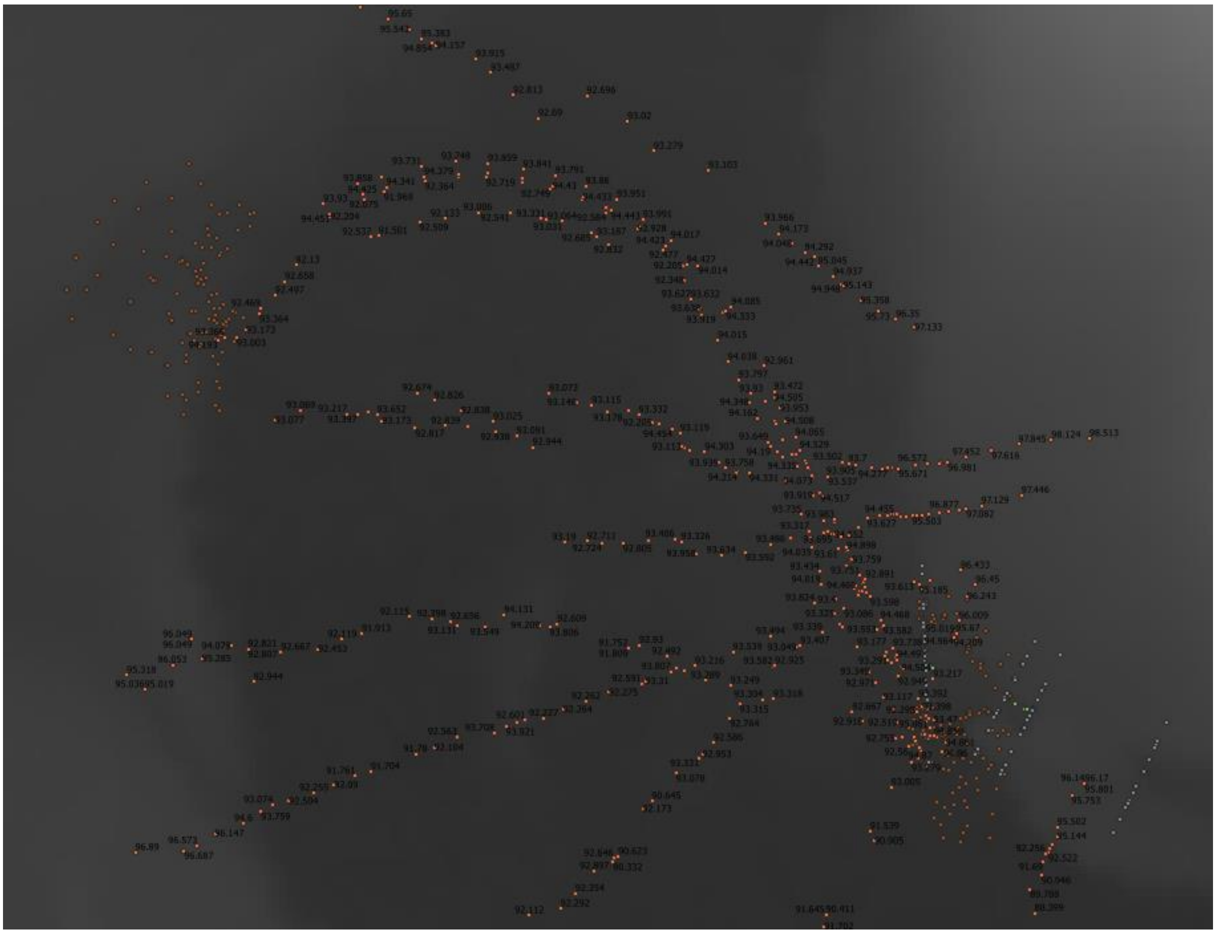

Figure 3.3. Horseshoe Falls Weir xyz data received (zoomed into area around the weir)

3.2 Model Build

3.2.1 Modelling overview

A 2D hydraulic TUFLOW model was constructed covering the area of interest at Horseshoe Falls Weir. The model was constructed using available hydrological data, freely available LiDAR, topographic data and information gathered during the site visit. For the baseline and Preferred option scenario, the model was run for the events described in section 2.2.2. In addition, flow sensitivity runs were carried out.

The baseline surface was constructed integrating the topographic data from the survey carried out at the study area, on free available 1 m resolution LiDAR data. Then, the baseline DEM was modified including a bypass channel and a notch on the weir crest to simulate the preferred option.

3.2.2 Model area

The area to be modelled encompasses the area where the topographic survey was undertaken. It extends 500m upstream and 300m downstream of the weir. These limits are illustrated on Figure 3.4 below.

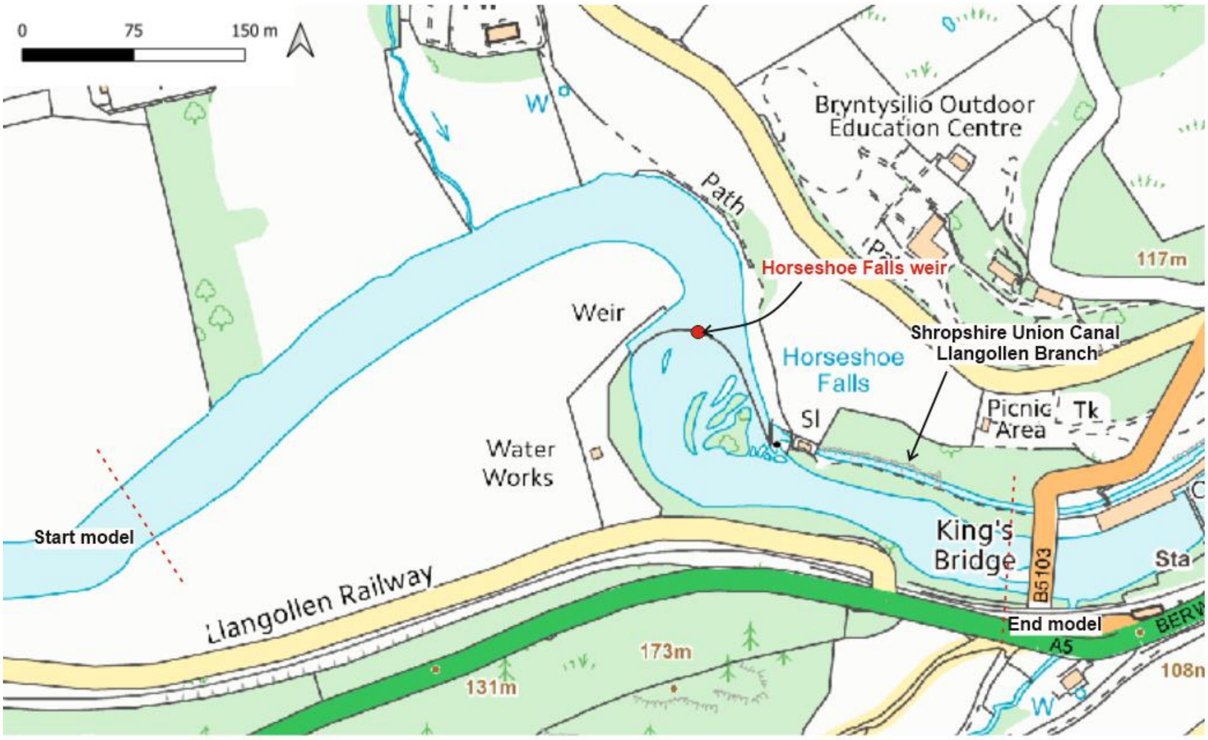

Figure 3.4 Horseshoe Falls weir modelling area (note model was extended further upstream and downstream to avoid effects of boundary conditions within the study area)

3.2.3 Baseline Model

Construction of the DEM

Initial data to build the modelling surface included free available 1m resolution LiDAR data and the topographic survey. The topographic survey included the following data:

- Survey at and around the weir;
- Survey on the leat channel
- Cross section along river Dee,

A DEM was constructed by integrating the topographic survey data on the available LiDAR data. The topographic survey data was interpolated using GIS software to create a TIN surface by. No iterations between cross sections were needed as the river morphology was well represented by the TIN surface.

A decision was made to exclude the canal from the model, as these do not function like normal rivers (i.e. gradients can be minimal). To exclude it the elevation of the area between the river and feeder stream were raised. An approximation of flow into the canal was made by revieing its abstraction licence and reducing the overall flow in the model, accordingly.

The TIN was created for wetted areas and merged with LiDAR, using TuFlow software, for non-wetted areas to create the final DEM. Figure 3.5. shows, the location of the topographic survey data points and LiDAR data followed by an illustration of the final baseline DEM.

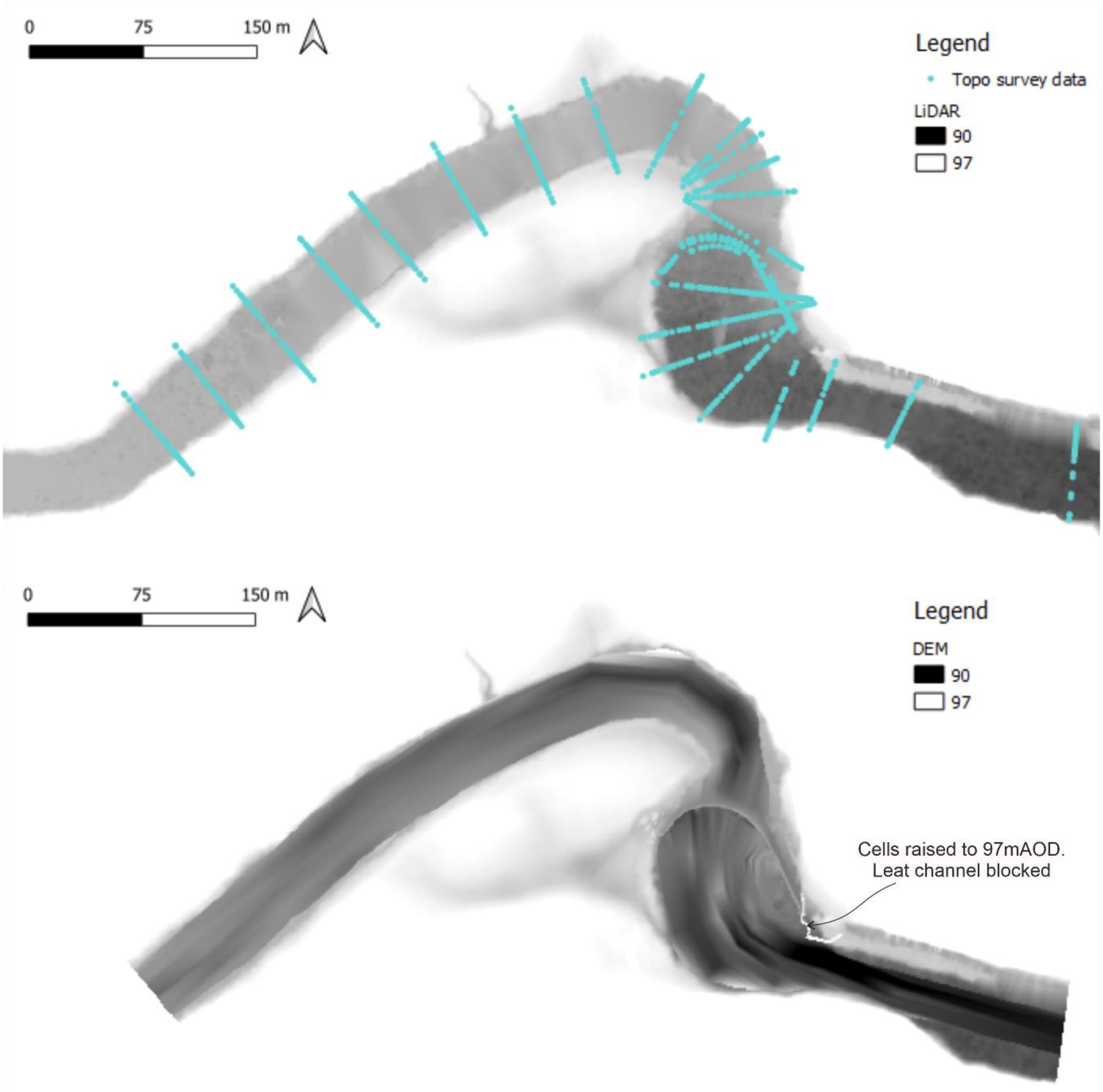

Figure 3.5 Baseline LiDAR and survey point locations and final baseline DEM for Horseshoe Falls weir model

Hydrological assessment of inflows for the model

Flow estimates for the site were as outlined in Section 2.2.2. Figure 3.6 shows the hydrograph associated with the flood events that were modelled.

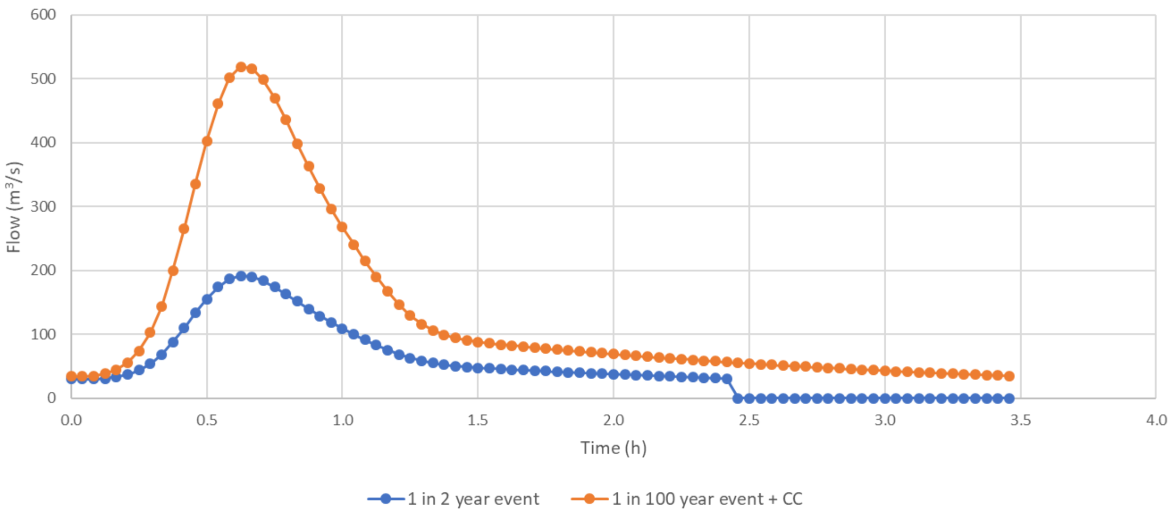

Figure 3.6 Flood events hydrographs for Horseshoe Falls weir model.

3.2.4 Design Scenario

Overview

A design to improve connectivity at Horseshoe Falls weir was developed. The preliminary preferred option under consideration by NRW was a nature-like by-pass channel in the RHB to improve ecological connectivity. Optioneering was undertaken to assess the suitability of this option and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Horseshoe Falls weir fish passage improvement design included:

- Improved passage primarily for upstream and downstream migrating salmonids, lamprey and eel across a range of flows
- Historic importance and designations
- Limited visual impact including avoidance of weir drying at low flows
- Canal abstraction (LHB) must not be impacted
- No increase to public safety risk or flood risk
- Maintenance

CRT and partners are currently endeavouring to provide new screens just upstream of the weir and on the LHB side, to prevent fish entering the canal feeder stream from the River Dee, via the offtake. Their current plans are to include a small notch downstream of the weir (1.0 m wide and 0.2 m deep) to reduce trash that may gather at the screen.

Bypass and notch design

Channel location and dimensions were iterated. The final bypass route was designed to follow a paleo-channel observed on LiDAR data, had a channel length of around 220m and a width of approximately 1.5m. The bypass top connection with the River Dee was designed for 94.1m AOD while the level at the bottom was 92.8m AOD. The bypass has an overall gradient of 0.72%. In order to increase velocities and attractant flow, the width along the first 20m of the bypass from the entrance was reduced to 1m and the gradient increased to 1.7%.

To design the notch, baseline DEM cells where reduced in 0.2m to represent 1m wide notch on the weir crest.

Figure 3.7 below represents the bypass channel and weir notch location and long sections.

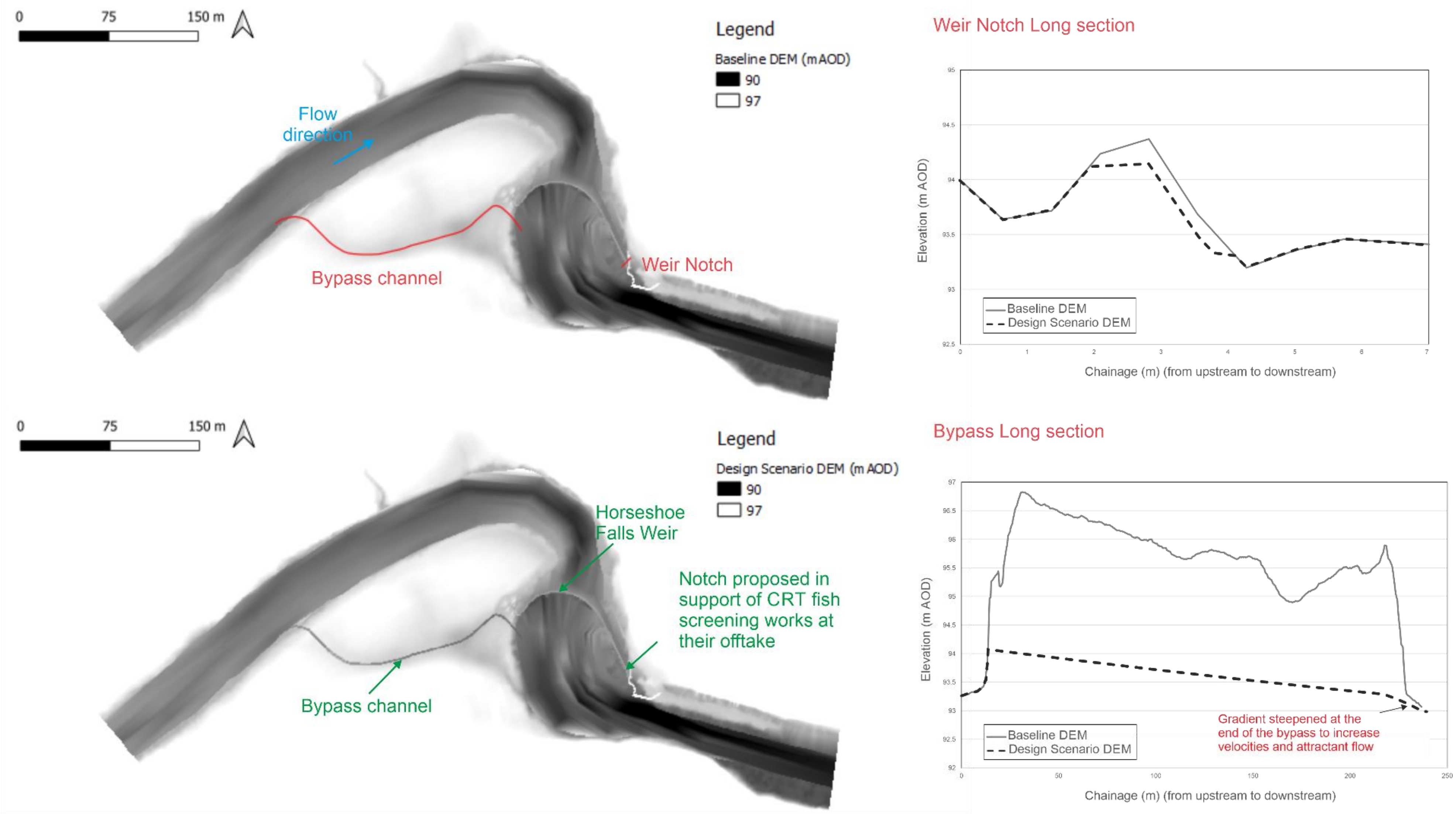

Figure 3.7. Design scenario for Horseshoe Falls weir model

3.2.5 Sensitivity Runs

In order to assess the potential impact of the abstraction to the feeder stream for the Llangollen Canal and noting that abstraction quantities are not know to us at present, river flows were reduced by the maximum abstraction that is permitted (0.96m3/s). Table 3.1 shows the revised river flows (for low, median and high flows) with this removed.

Table 3.1 Horseshoe flow (statistic) estimates considering Llangollen Canal abstraction

Flow (statistic) estimates (m³/s)		
Q_{95}	Q_{50}	Q_{10}
5.4	13.5	51.3

Results of the sensitivity analyses are presented in Figure 3.8 and Figure 3.9. Results for low flows have been presented as the site is considered to be flow sensitive. Results indicate that with maximum abstraction to the Llangollen canal offtake, the bypass should enable fish passage opportunities for multiple species. This is also the case with a small notch included, as planned by the CRT in tandem with their screen replacement works. This size of their notch may not considerably improve smolt passage, particularly given its location, but does demonstrate that both schemes should be compatible.

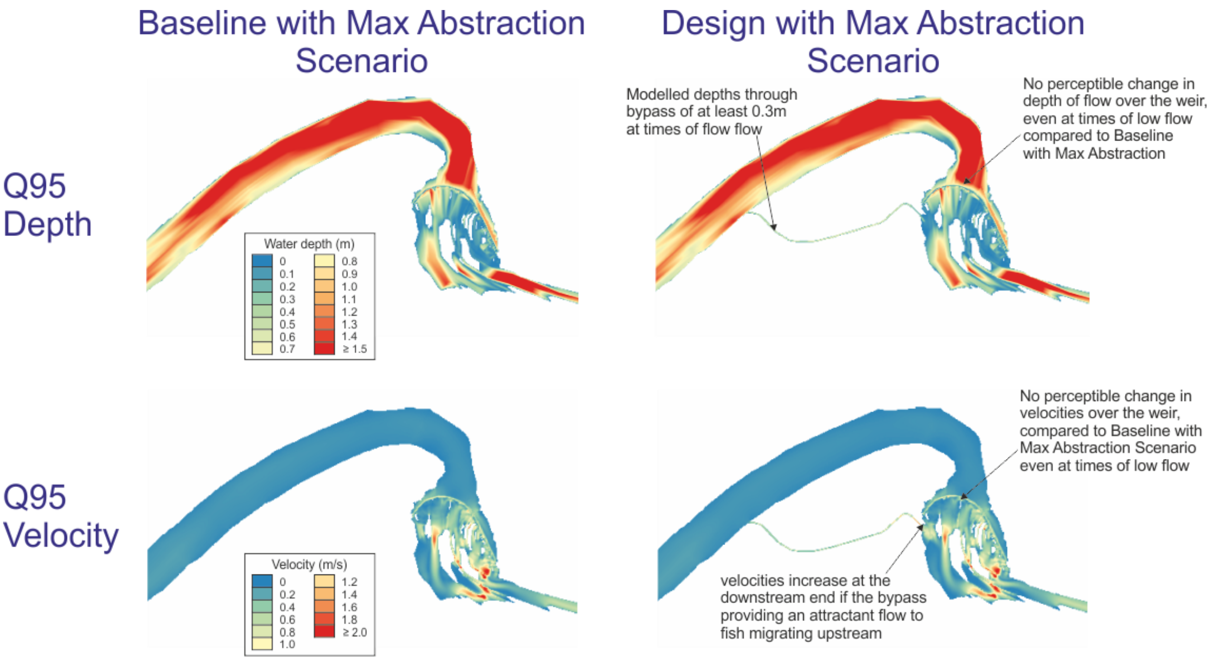

Figure 3.8 Comparison of Baseline and Design Max Abstraction Scenarios (Q_{95} depth and velocities)

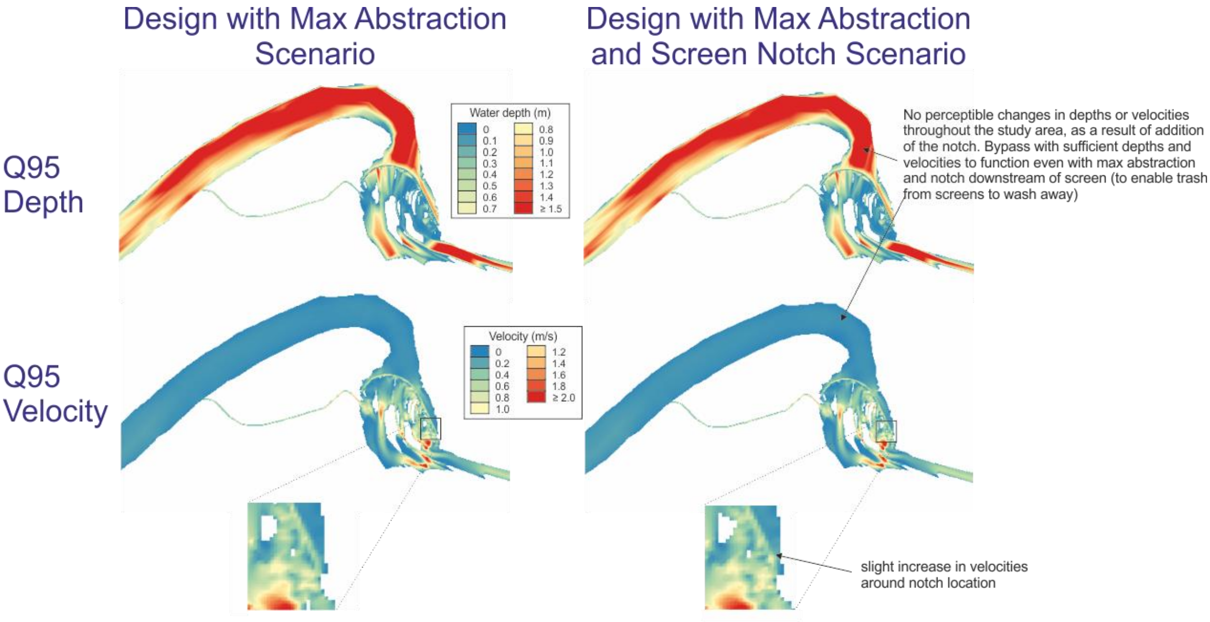

Figure 3.9 Comparison of Design Max Abstraction and Design Max Abstraction with CRT Notch Scenarios (Q_{95} depth and velocities)

3.3 Results

Plots of velocities and depths for various flows under the baseline, bypass and notch scenarios are presented in Figure 3.10 and Figure 3.11, respectively. Approximate maximum water depth and velocity ranges at Q_{95} - Q_{10} (Table 3.2) in the bypass was sufficient for a range of target species and life stages[3]. Bypass substrate should be suitably selected based on geomorphological considerations and requirements for fish passage for all target species and life stages. Should the notch option also be taken forward, detailed design should ensure that flow accelerates gradually and smoothly into a bell mouthed notch entrance to assist the passage of downstream migrating smolts, and that canal intake screening complies with best practice guidance[4]. Discharge from the bypass constituted 2.5%, 3.3% and 7.3% of Annual Daily Flow under Q_{95}, Q_{50} and Q_{10} scenarios, respectively. Best practice guidance suggests a minimum target discharge of 5% ADF for fish attraction to a route of passage, however, the guidance also notes that there can be no prescriptive definition of discharge requirements and that these will be dependent on site specific factors[3].

[3] Armstrong, G.S., Aprahamian, M.W., Fewings, G.A., Gough, P.J., Reader, N.A. and Varallo, P.V., 2010. *Environment Agency Fish Pass Manual: Guidance notes on the legislation, selection and approval of fish passes in England and Wales.* Environment Agency, Rio House, Bristol http://publications. environment-agency. gov.uk.

[4] Environment Agency, 2005. *Screening for intakes and outfalls: a best practice guide*. Science Report CS030231. Environment Agency, Bristol, 153 pp.

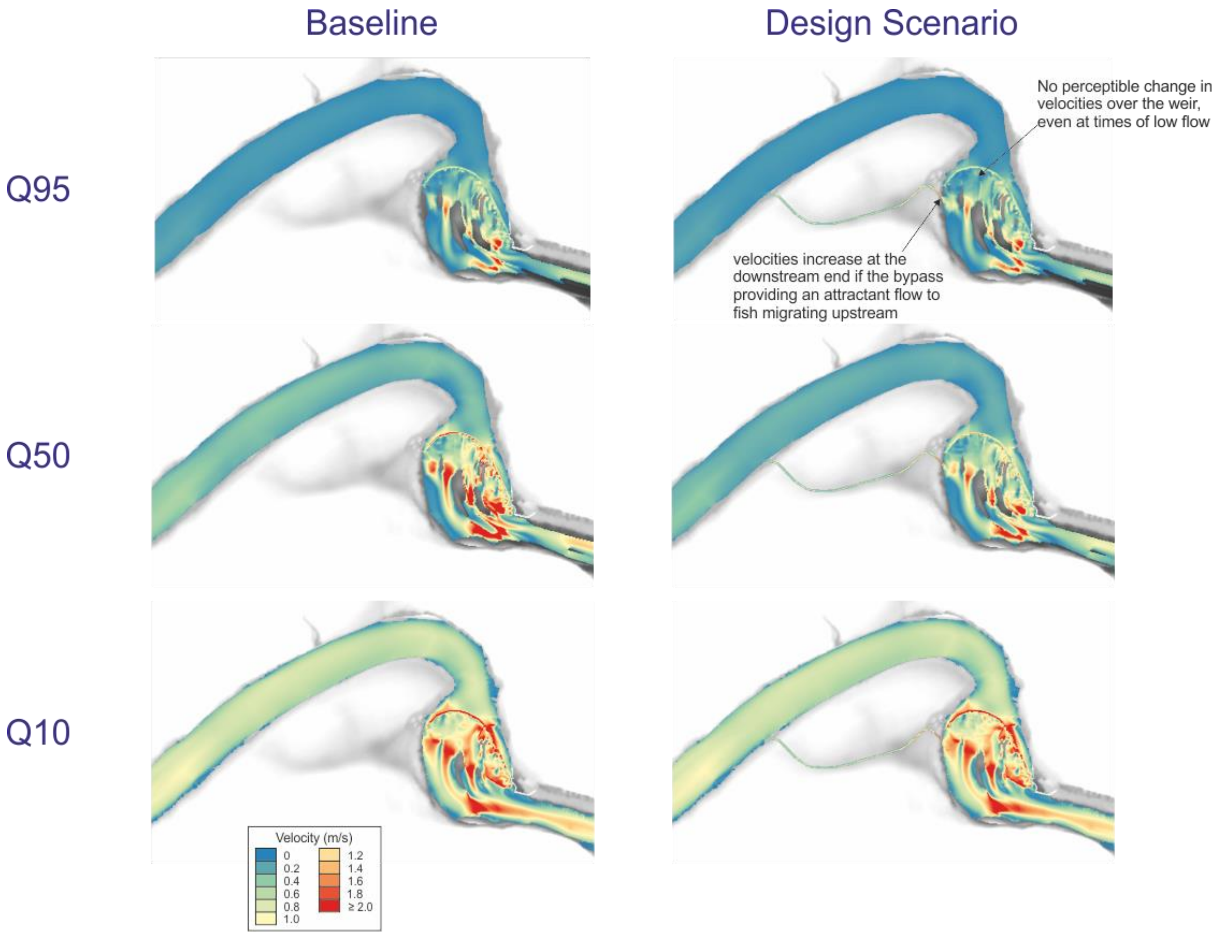

Figure 3.10 Velocity results for various flows under the baseline and design (bypass and 0.2 m deep x 1.0m wide notch) scenarios

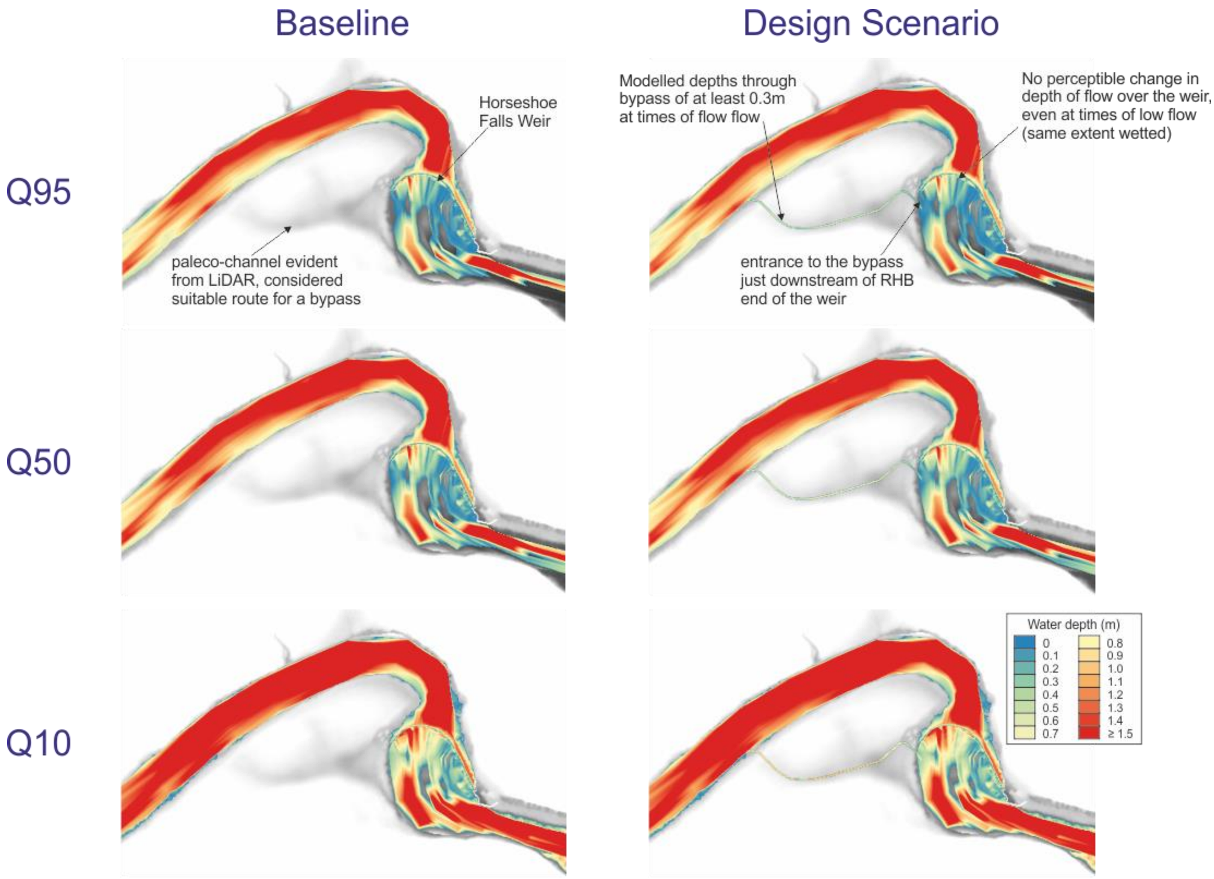

Figure 3.11 Depth results for various flows under the baseline and design (bypass and 0.2 m deep x 1.0 wide notch) scenarios

Table 3.2 Approximate maximum design (RHB bypass and LHB 0.2 m deep x 1.0 wide notch) scenario velocity and depth ranges through the bypass channel at Q_{95}, Q_{50} and Q_{10} flows

Flow statistic	Velocity (m/s)	Depth (m)
Q_{95}	1.6	0.2 – 0.6
Q_{50}	1.8	0.3 – 0.8
Q_{10}	2.0	0.4 – 1.2

No change in the extent or depths of flooding is predicted other than where the dimensions of the weir itself are altered and along the proposed bypass (see Figure 3.12).

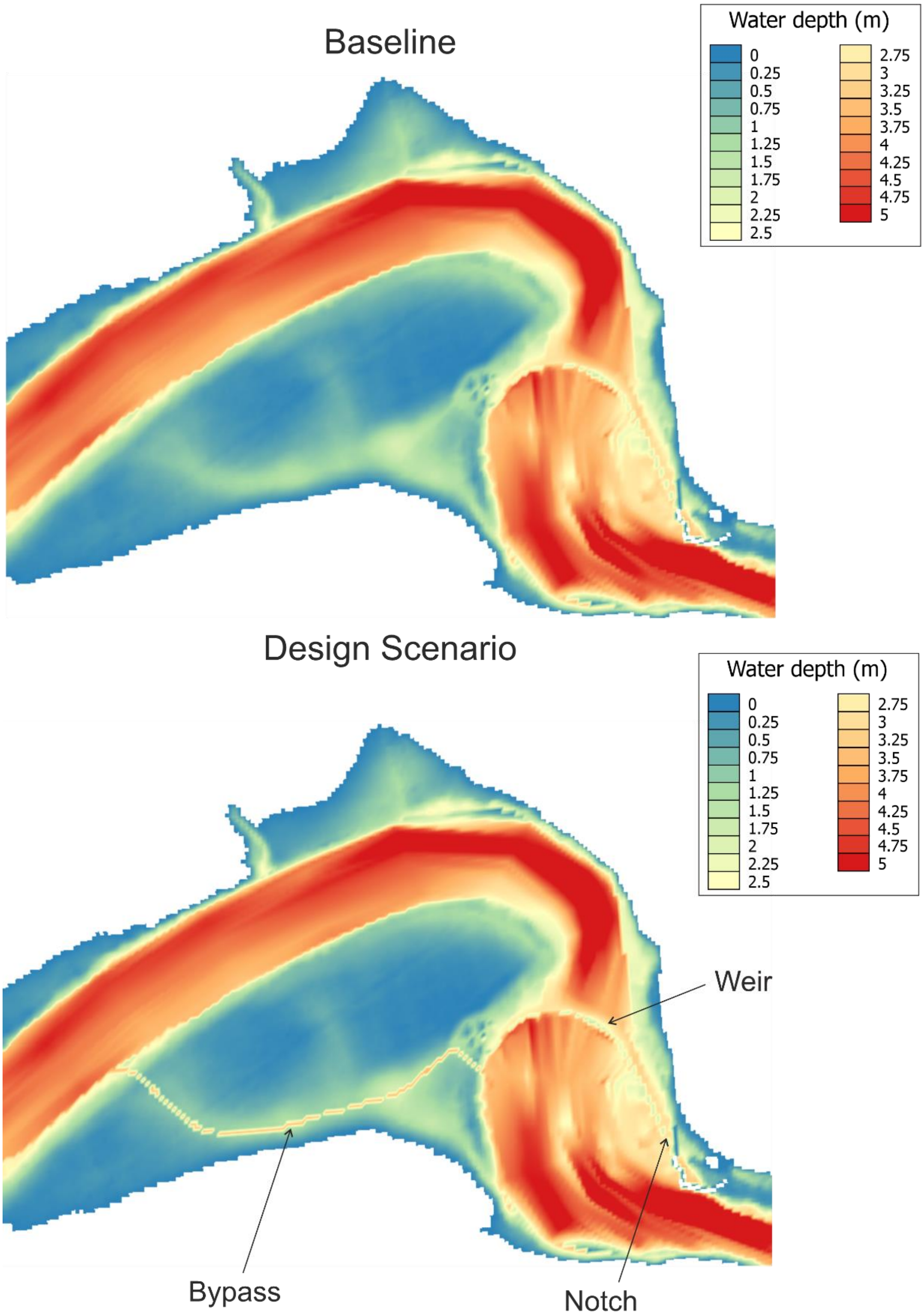

Figure 3.12 1 in 100 year flood extent (plus climate change) for Baseline and Final Scenario

Results of the sensitivity analyses are presented in Figure 3.13 and Figure 3.14. Results for low flows have been presented as the site is considered to be flow sensitive. Results indicate that with maximum abstraction to the Llangollen canal offtake, the bypass should enable fish passage opportunities for multiple species. This is also the case with a small notch included, as planned by the CRT in tandem with their screen replacement works. This size of their notch may not considerably improve smolt passage, particularly given its location, but does demonstrate that both schemes should be compatible.

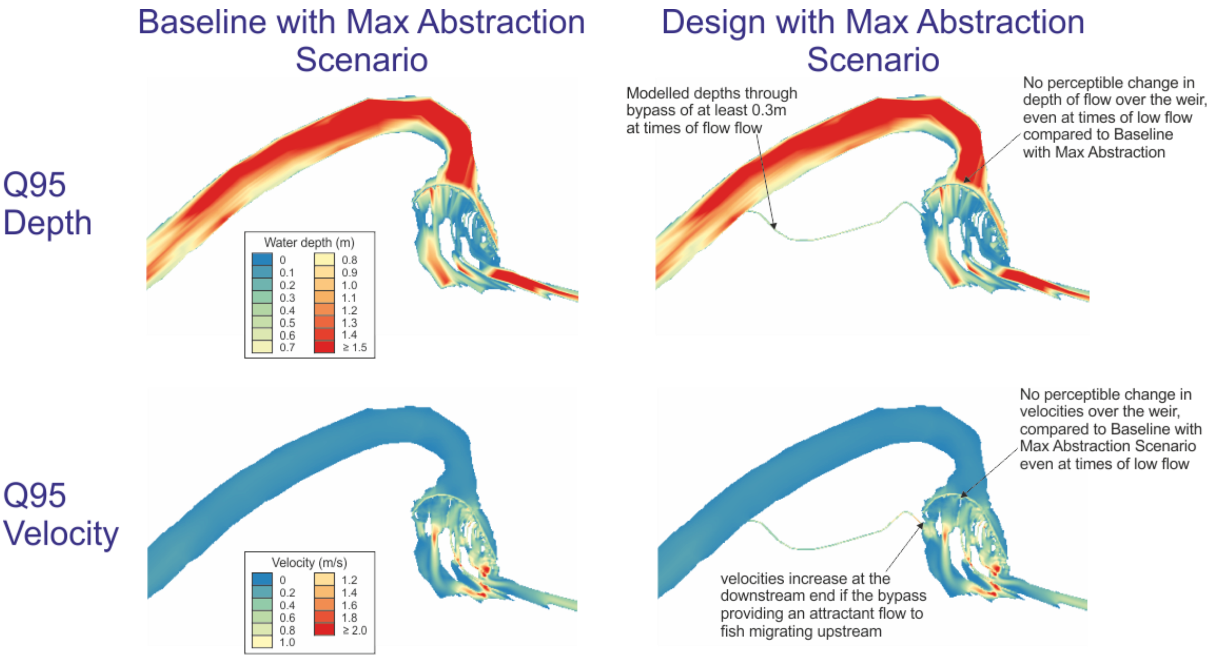

Figure 3.13 Comparison of Baseline and Design Max Abstraction Scenarios (Q_{95} depth and velocities)

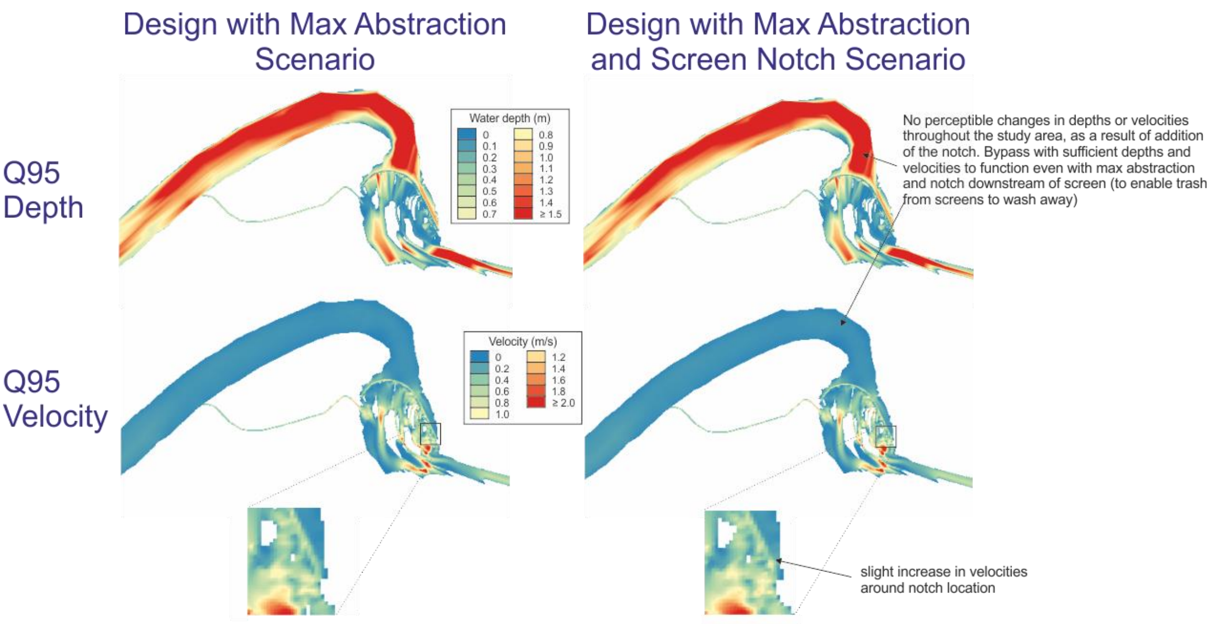

Figure3.14 Comparison of Design Max Abstraction and Design Max Abstraction with CRT Notch Scenarios (Q_{95} depth and velocities)

4. Llangollen Upstream Weir

4.1 Baseline

4.1.1 Site

Site around the weir is illustrated on Figure 4.1. Llangollen upstream weir is situated within the town of Llangollen, set within the valley of the River Dee, on an elevation of approximately 85m AOD. It is composed of concrete with a head of approximately 1m and is formed of two upstream arches that span the width of the river (approximately 115m). The stretch of the river around the weir is a popular kayaking area.

Historic mapping indicates that in the 19th century through to at least 1959 the weir only extended across half of the river. By 1970-1975 mapping the weir had been extended the whole length of the river.

Upstream of the weir, the river is comprised of a deep channel coupled with large areas of exposed bedrock, with patches of trees. A mixture of exposed bedrock and trees are present downstream of the weir.

To the north of the weir and river are the Llangollen Heritage Railway and railway station primarily used for steam powered trains is present. Beyond this lies a mixture of residential and commercial properties (those properties neighbouring the river appear to have private fishing rights adjacent to their properties). Abbey Road (A542) runs parallel to site and a marina is present further north, connected to the Shropshire Union Canal Llangollen Branch.

South of the weir and river there are commercial and residential properties, along with places of worship are present. Large area of open green space beyond this. To the south-east of the weir there is a former Corn Mill, now a restaurant is present on the southern bank with the Llangollen Bridge further east. The outside area of the restaurant is decked and water flows underneath this area. Towards the downstream end there is a former water wheel which is inactive.

The preferred fish passage option at project onset was creation of three notches to bed level in the weir crest (8-10m wide).

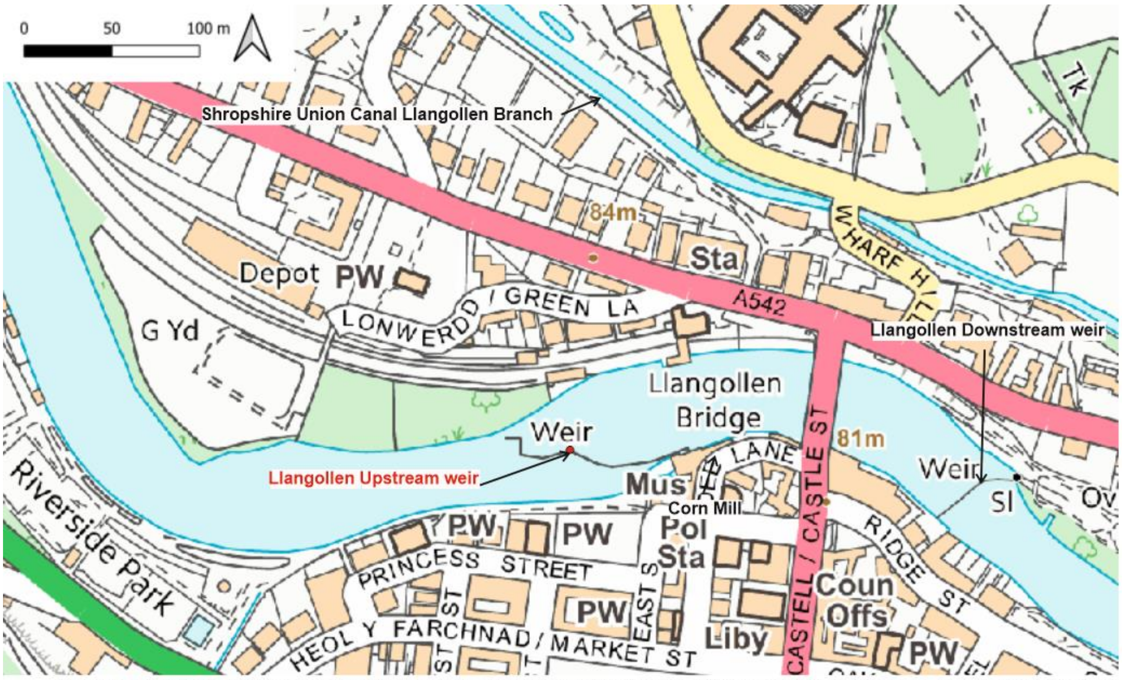

Figure 4.1 Llangollen upstream weir study area

4.1.2 Site Visit

A site was visited on 17 July 2020. Images from this, complimented by imagery from the topographic survey, are presented in Plates 4-1 through to 4-11.

Plate 4-1. River Dee upstream of the weir

Plate 4-2. River Dee upstream of the weir (looking upstream)

Plate 4-3. River Dee upstream of the weir (looking downstream)

Plate 4-4. Weir looking across the channel from RHB

Plate 4-5. Weir looking across the channel from RHB

Plate 4-6. Weir looking across the channel from RHB

Plate 4-7. Weir looking across the channel from RHB

Plate 4-8. Weir looking across the channel from RHB (Corn Mill pub decking apparent)

Plate 4-9. Private Fishing berth (RHB River Dee upstream of the weir)

Plate 4-10. River Dee looking upstream from Castle Street Bridge/ downstream of the weir

Plate 4-11. River Dee looking upstream from Castle Street Bridge/ downstream of the weir- bridge pier apparent

The site was visited once more on the 2 September 2020. Further notable images from this visit are presented in Plates 4-12 to 4-20.

Plate 4-12. Panoramic view of area downstream of the weir, from decked area of restaurant on RHB

Plate 4-13. Looking along the weir crest from restaurant on the RHB

Plate 4-14. Looking at area just downstream of the weir crest from restaurant on the RHB

Plate 4-15. Inactive water wheel located in the Corn Mill Pub

Plate 4-16. Looking upstream to area just downstream of the weir, from Corn Mill Pub (RHB)

Plate 4-17. Looking downstream from Corn Mill Pub (RHB) Rapid area popular with kayakers is evident

Plate 4-18. Looking upstream to weir and area just downstream of it, from Corn Mill Pub (RHB)

Plate 4-19. Another view of the weir and wider area from entrance to Llangollen Railway Station

Plate 4-20. View of the Corn Mill Pub from Llangollen Railway Station (closed sluice is apparent)

4.1.3 Topographic and Bathymetric Survey

A topographic survey of the site and surrounding area was undertaken. This included a survey at and around the weir, cross sections in the River Dee upstream and downstream extending approximately 400m upstream of the weir and continuing downstream beyond the Llangollen Downstream weir (see Section 5.1.3). The survey was undertaken to refine representation of the system within the hydraulic modelling (see Section 4.2.3).

All surveyed points are indicated in Figure 4.2 below while those around the weir itself are indicated in Figure 4.3, which shows a reasonable coverage with some gaps where surveying could not be undertaken safely.

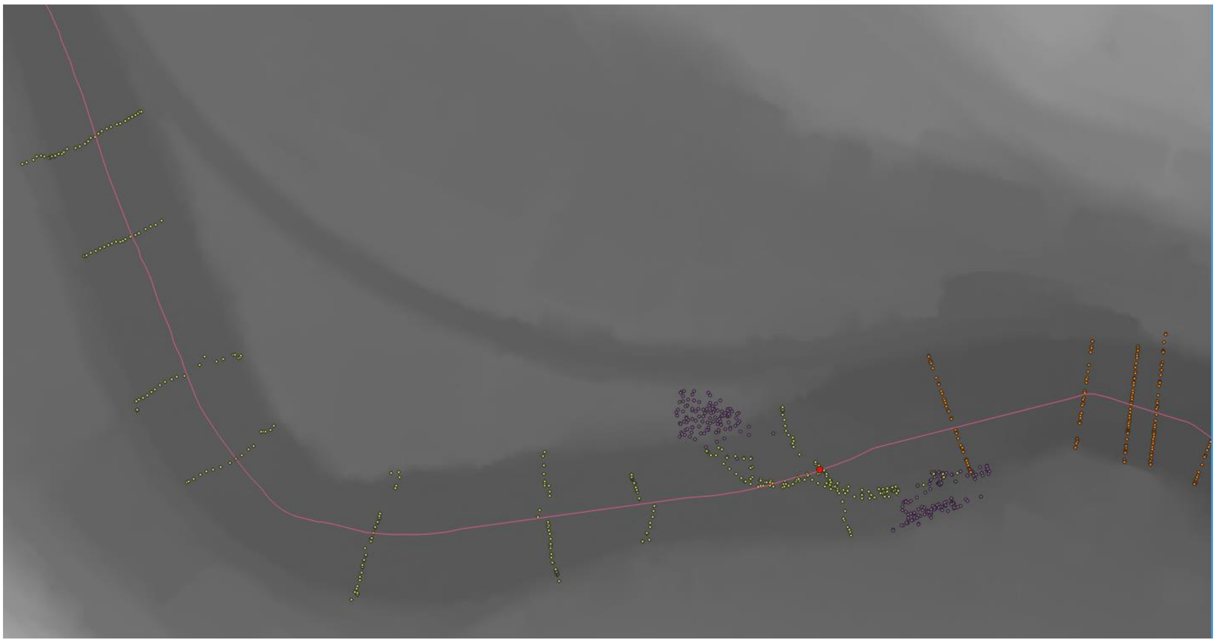

Figure 4.2 Llangollen Upstream weir xyz topo data received (different colours showing different areas)

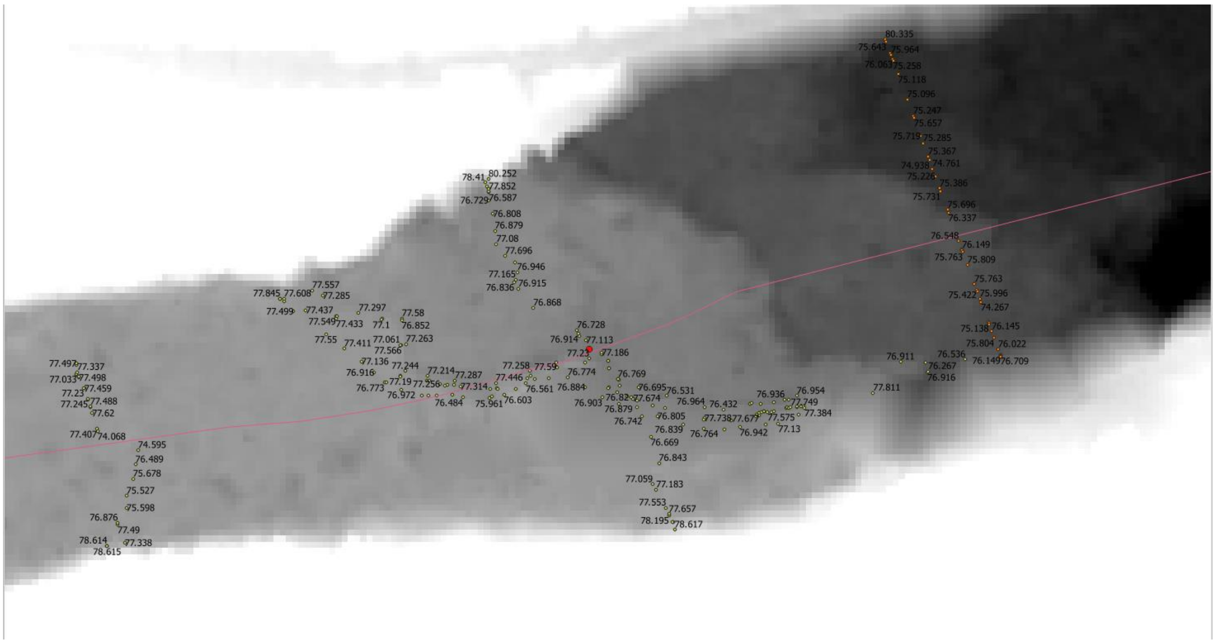

Figure 4.3 Llangollen Upstream Weir xyz data received (zoomed into area around the weir)

The difference between all surveyed river points and LiDAR data is shown in Figure 4.4. This suggests that differences were generally within 1m, though upstream of the weir a deeper central channel was picked up by the survey but not the LiDAR (which is as expected, with water refracted LiDAR rays), again showing the value of undertaking the survey.

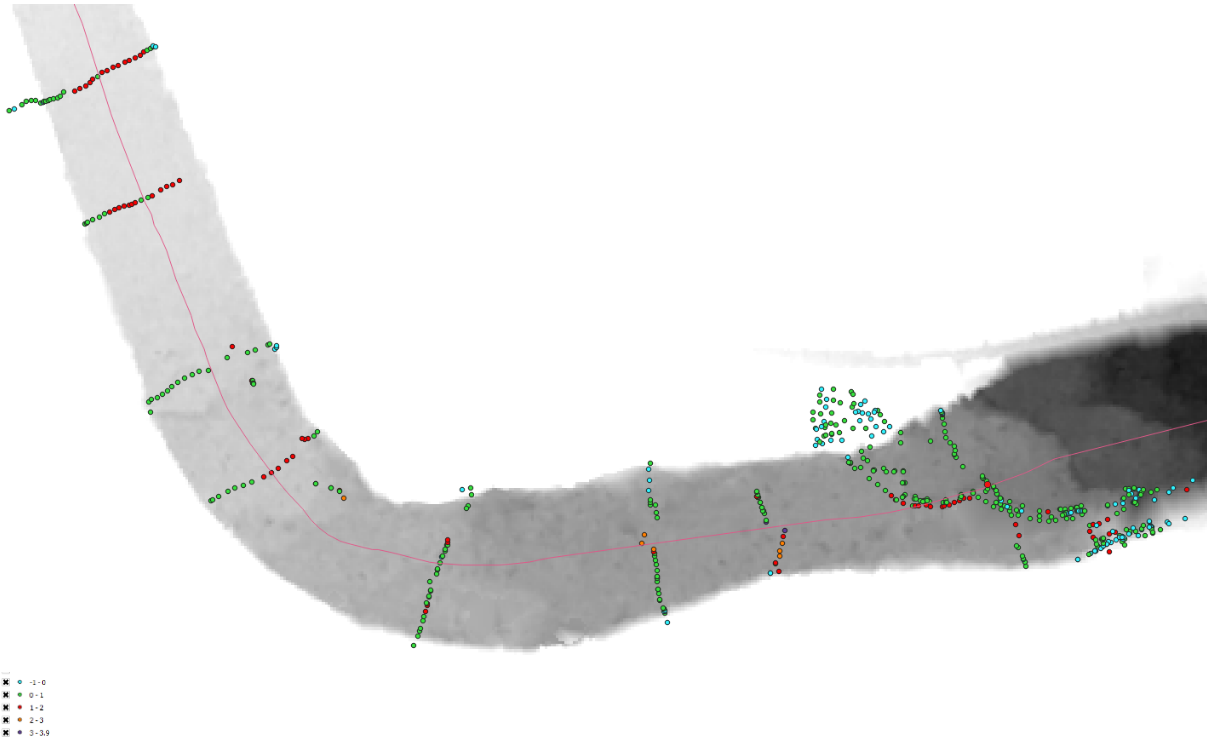

Figure 4.4. Differences between LiDAR and survey xyz data (LiDAR minus survey)

4.2 Model Build

4.2.1 Modelling overview

Due to the proximity of both Llangollen weirs, a 2D hydraulic TUFLOW model was constructed encompassing both the upstream and downstream weirs (as well as upstream and downstream of them). The model was constructed using available hydrological data, freely available LiDAR, topographic data and information gathered during the site visit. For the baseline and preferred option scenario, the model was run for the events described in section 2.2.2.

The baseline surface was constructed integrating the topographic data from the survey carried out at the study area, on free available 1 m resolution LiDAR data. Then, the baseline surface was adjusted simulating weir crest notch.

4.2.2 Model area

As mentioned before, the model area was built for upstream and downstream weirs together. The area to be modelled encompasses the area where the topographic survey was undertaken. These limits are illustrated on Figure 4.5 below.

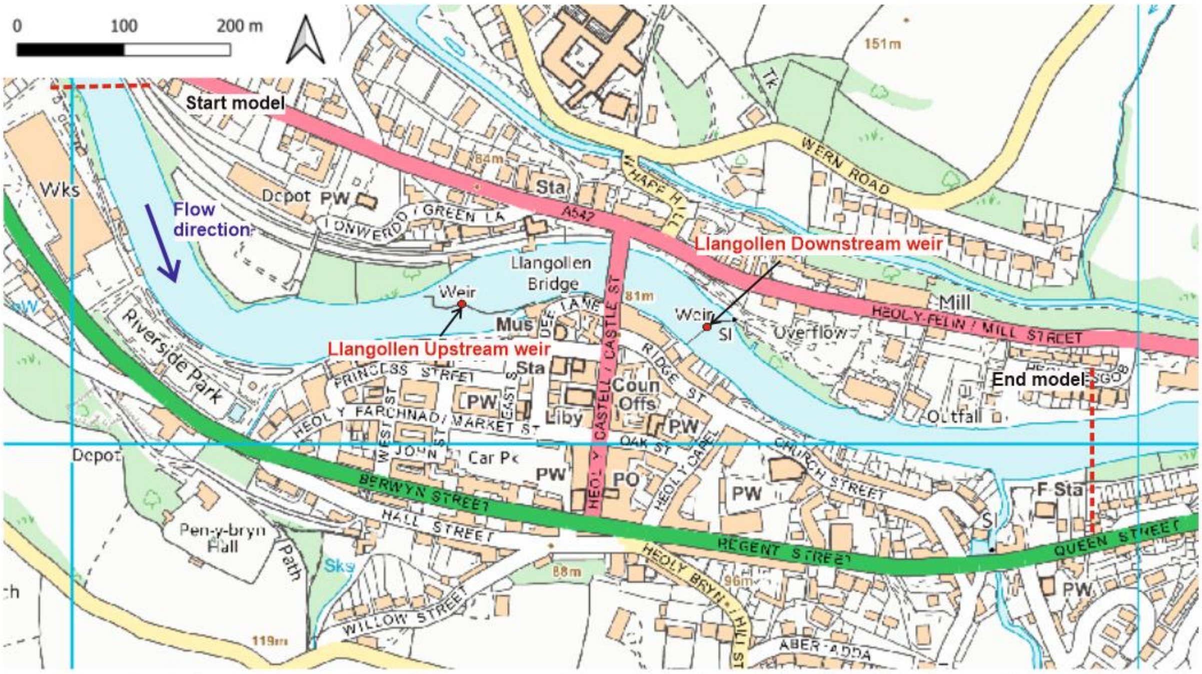

Figure 4.5 Llangollen upstream and downstream weirs study area

4.2.3 Baseline Model

Construction of the DEM

Initial data to build the modelling surface included free available 1m resolution LiDAR data and the topographic survey. The topographic survey included the following data:

- Survey at and around the upstream and downstream weirs;
- Cross sections along river Dee.

A DEM was constructed by integrating the topographic survey data with the available LiDAR data. A number of survey iterations were created using GIS software. An initial review of the TIN created with the data available indicated that iterations around Llangollen upstream weir were needed to match the River morphology, as observed on aerial imagery. Figure 4.6 shows the topographic survey data and the location of interpolated sections that were created to construct the surface of the DEM.

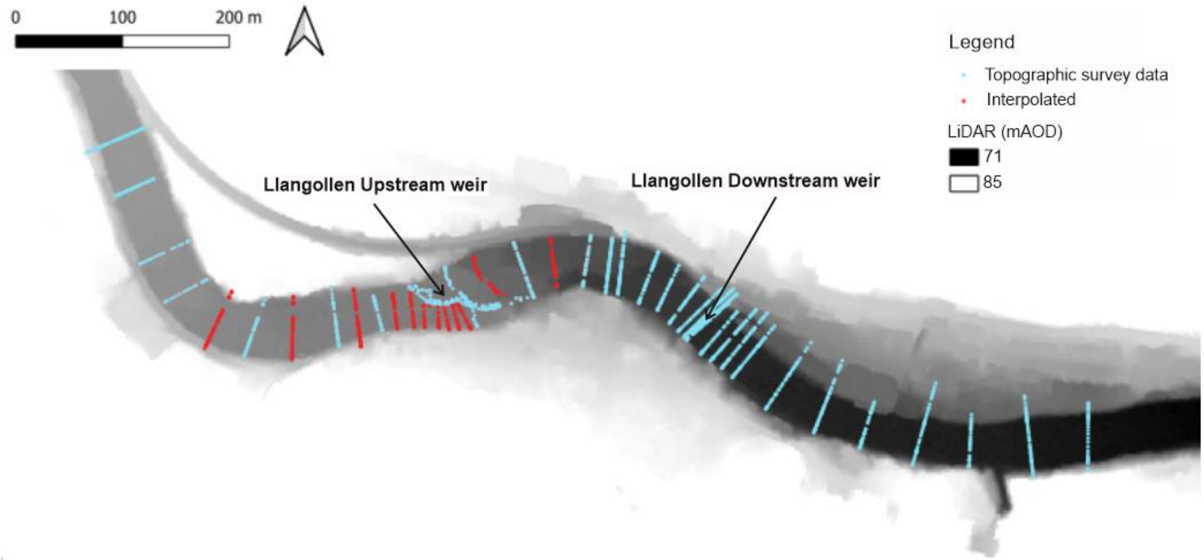

Figure 4.6 LiDAR information and Location of surveyed and interpolated sections

Once the TIN surface performance was considered suitable it was merged with the LiDAR using TuFlow software. Figure 4.7 shows the final baseline DEM around the Llangollen Upstream weir.

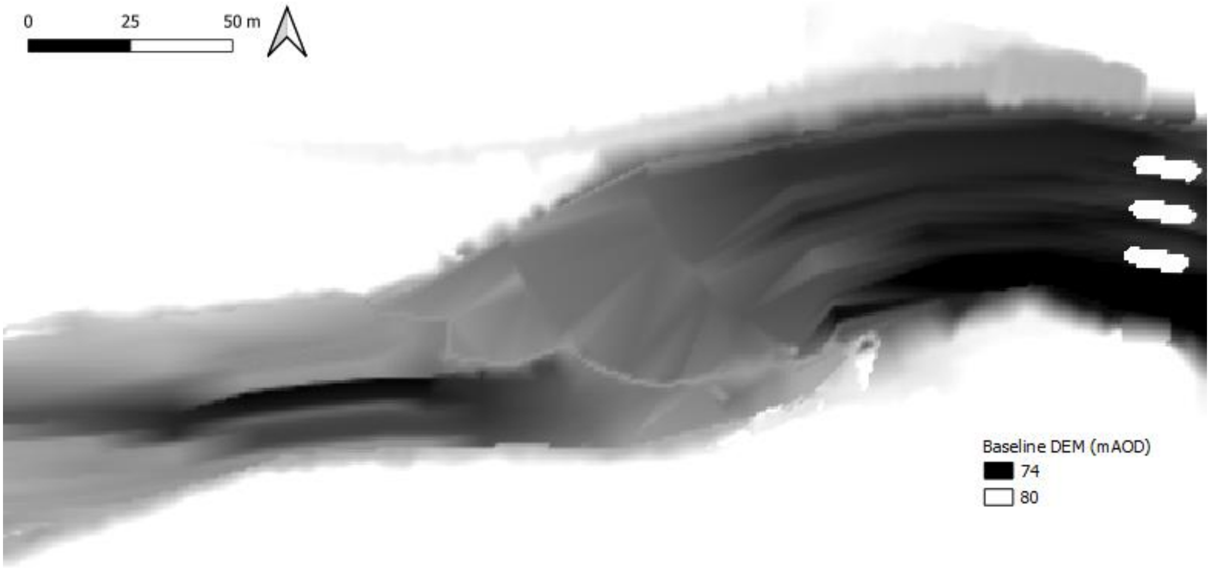

Figure 4.7 Final Baseline DEM around Llangollen upstream weir

Hydrological assessment of inflows for the model

Flow estimates for the site were as outlined in Section 2.2.2.

Figure 4.8 shows the hydrograph associated with the flood events that were modelled.

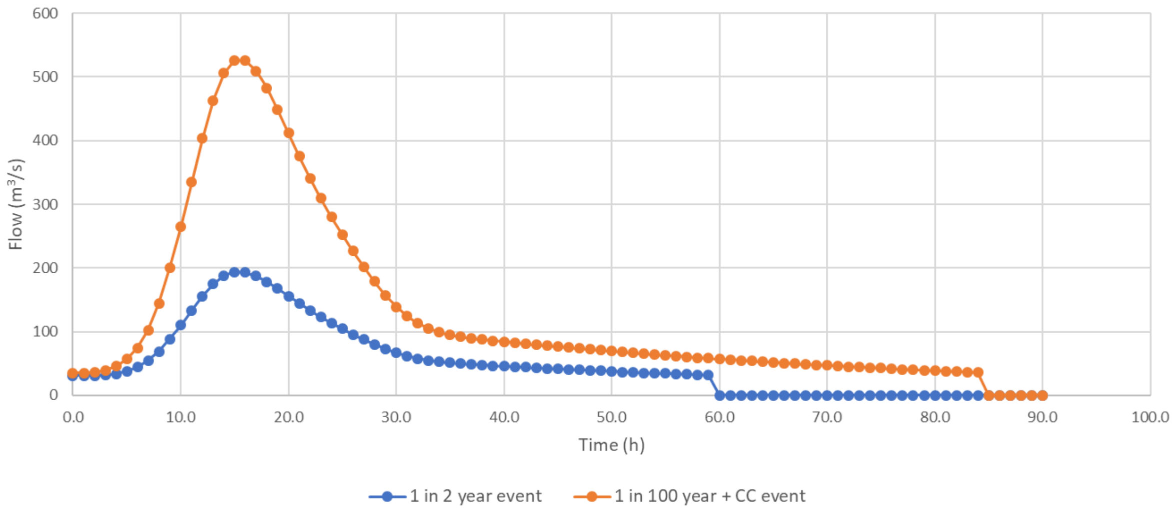

Figure 4.8 Flood events hydrographs for Llangollen upstream and downstream weirs model

4.2.4 Design Scenario

Overview

The preliminary preferred option under consideration by NRW was creation of at least three notches within the weir crest, approximately 8-10m wide. The notches were to be to bed level at intervals across the structure, in a way that did not increase confusion for fish passage upstream and improved longitudinal ecosystem connectivity. Optioneering was undertaken to assess the suitability of this option and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Llangollen Upstream weir fish passage improvement design included:

- Improved passage primarily for downstream migrating salmonids, lamprey and eel across a range of flows

- Use of the area by kayakers and canoeists
- Area of historical interest
- Limited visual impact including avoidance of weir drying at low flows
- No increase to public safety risk or flood risk
- Maintenance

The preliminary preferred option would improve fluvial connectivity for salmonids, enable passage by canoeists and kayakers and cause no increase to public safety risk or maintenance. Notching the weir was considered preferable to other fish passage options e.g. technical and other non-technical options, particularly for the provision of downstream passage for smolts. However, initial hydraulic assessments identified that the preliminary option would cause drying of the weir crest at low to medium flows, in part due to the large notch sizes and wide, zig-zag nature of the weir. Therefore, the preliminary option was refined to prevent weir drying at low flows whilst maintaining all the benefits.

The final recommended option was a single notch (0.3m deep x 3.0m wide) positioned in the centre of the weir, at a location consistent with the thalweg enabling easy discovery by downstream migrating smolts[5]. A notch width of 3.0m was selected to assist canoe passage

Notch design

The notch design includes a 3.0m wide and 0.3m deep channel along the weir face towards the riverbed, avoiding glacis drying, sudden drops and rough surfaces[6].

To represent the notch, baseline DEM cells were reduced by 0.3m around the area indicated on Figure 4.9 below.

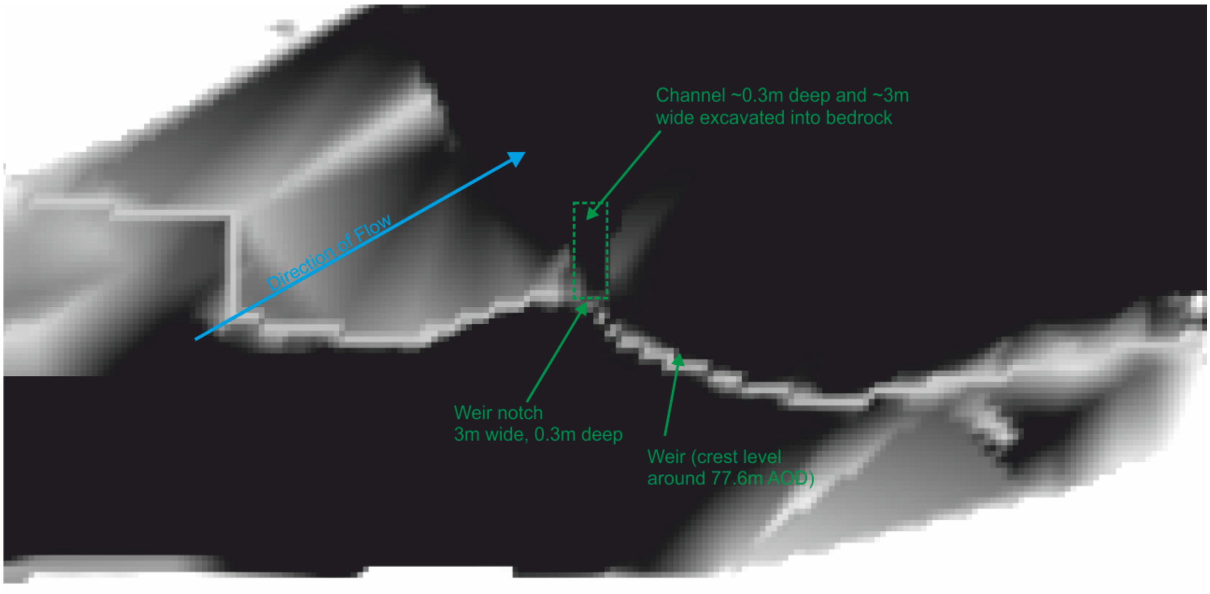

Figure 4.9 DEM for the partial removal of Llangollen Upstream weir scenario

4.3 Results

Plots of velocities and depths for various flows under the baseline and notch scenarios are presented in Figure 4.10 and Figure 4.11, respectively. Average water depths and approximate maximum

[5] Thorstad, E.B., Whoriskey, F., Uglem, I., Moore, A., Rikardsen, A.H. and Finstad, B., 2012. A critical life stage of the Atlantic salmon *Salmo salar*: behaviour and survival during the smolt and initial post-smolt migration. *Journal of Fish Biology*, 81(2), 500-542.

[6] Environment Agency, 2005. *Screening for intakes and outfalls: a best practice guide*. Science Report CS030231. Environment Agency, Bristol, 153 pp.

velocities at Q_{95} - Q_{10} in the notch were sufficient for downstream migrants (Table 4.1), providing that detailed design ensures gradual and smooth flow acceleration into a bell mouthed notch entrance. A pool immediately downstream of the notch and excavated channel may also be required. A minimum depth of 0.9m for head differences of <3.6m is suggested[7], however refinement based on considerably lower head differential at Llangollen Upstream weir and local constraints may be possible. Upstream migrating adult salmonids, as well as some coarse fish species undertaking local migrations, are anticipated to be able to ascend through the notch at low – medium flows[8] (Section 3.3.1 of the main report), however another route of passage exists at higher flows for these species and life stages along the LHB. Approximate maximum velocity along the LHB at Q_{10} was 1.2m/s with a depth of 0.4m (Table 4.1).

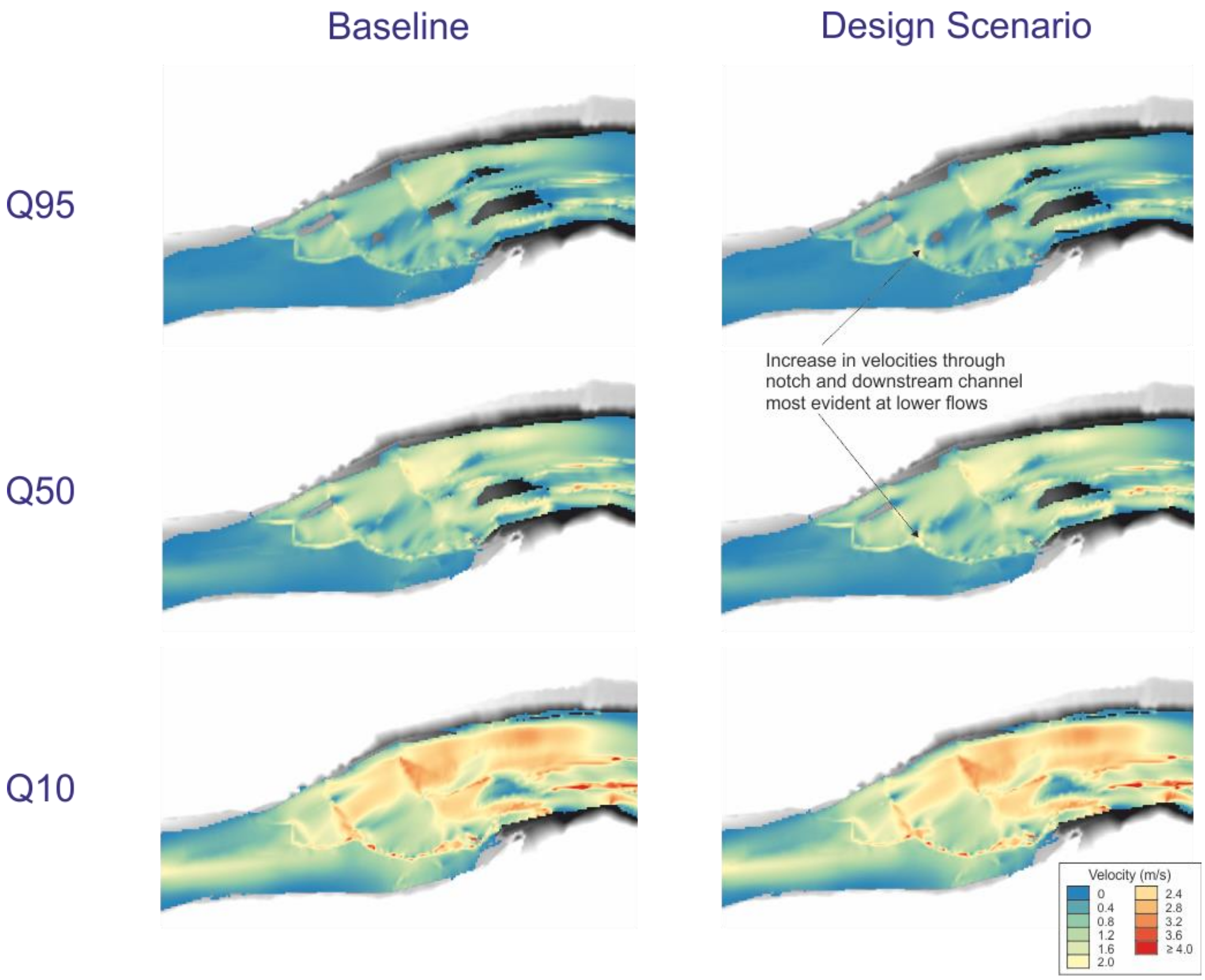

Figure 4.10 Velocity results for various flows under the baseline and design (0.3 m deep x 3.0 wide notch) scenarios

[7] Odeh, M and Orvis, C., 1998. Downstream fish passage considerations and developments at hydroelectric projects in North-east USA. In: *Fish Migration and Fish Bypasses* (Ed. Jungwirth, M., Schmutz, S. and Weiss, S.). Fishing News Books, Oxford, Blackwell: 267-280.

[8] Armstrong, G.S., Aprahamian, M.W., Fewings, G.A., Gough, P.J., Reader, N.A. and Varallo, P.V., 2010. *Environment Agency Fish Pass Manual: Guidance notes on the legislation, selection and approval of fish passes in England and Wales*. Environment Agency, Rio House, Bristol http://publications. environment-agency. gov.uk.

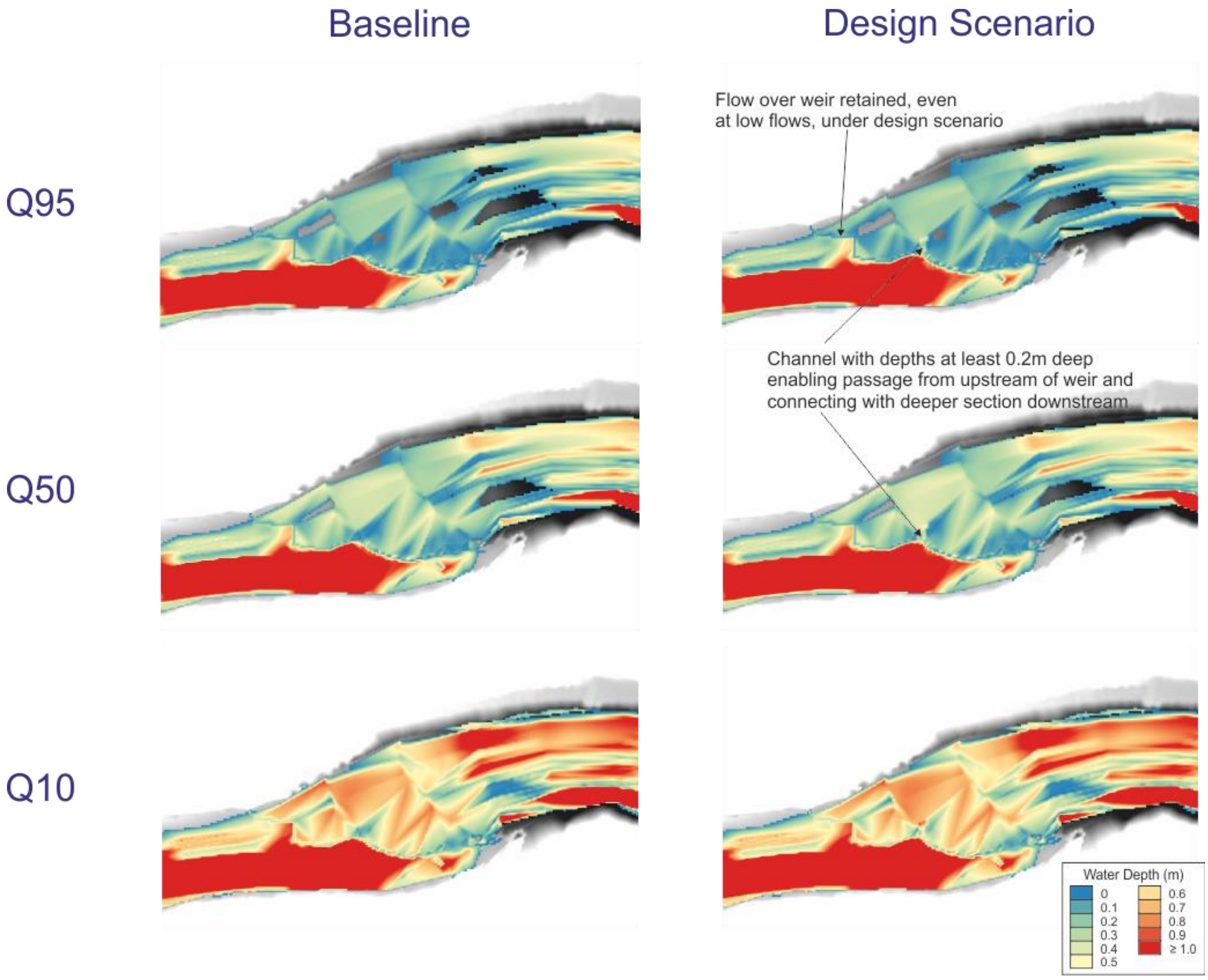

Figure 4.11 Depth results for various flows under the baseline and design scenarios

Table 4.1 Approximate maximum baseline and design (0.3m deep x 3.0m wide notch) scenario velocities and depths at Q_{95}, Q_{50} and Q_{10} flows

Flow	Velocity (m/s)		Depth (m)	
	Baseline	Design Scenario	Baseline	Design Scenario
Q_{95}	2.0	2.4	0.1	0.3
Q_{50}	2.4	2.4	0.2	0.4
Q_{10}	3.6	3.6	0.4	0.6

No change in the extent or depths of flooding is predicted other than where the dimensions of the weir itself are altered (see Figure 4.12).

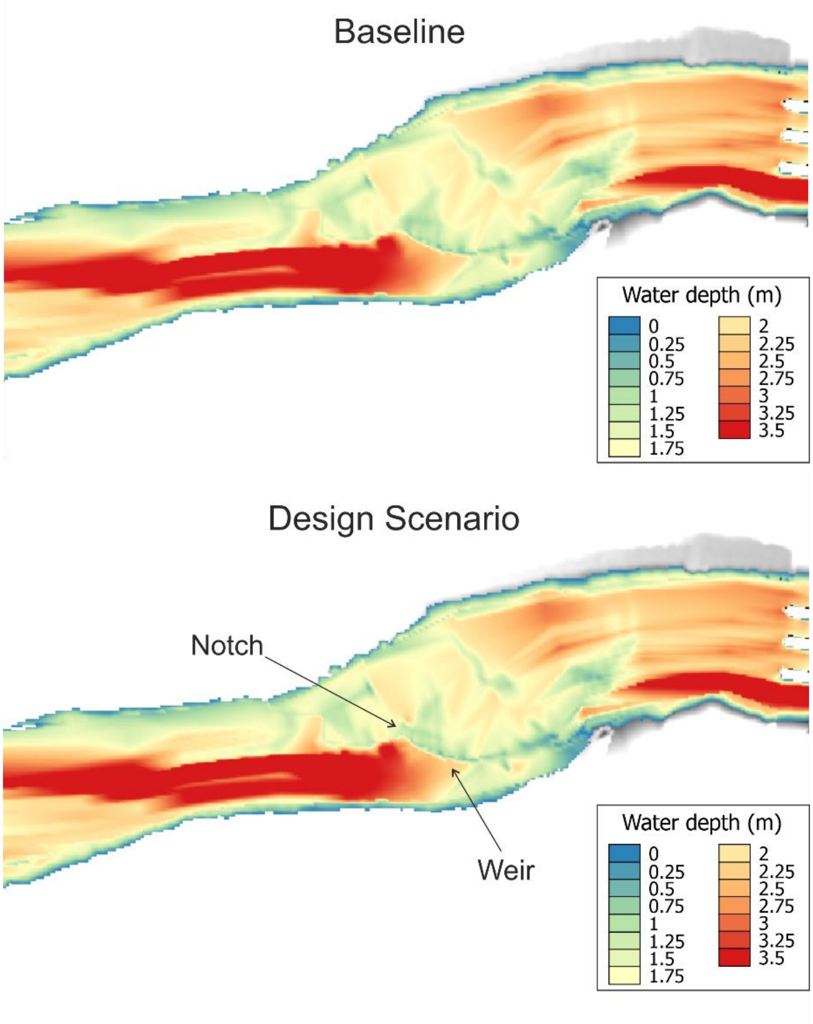

Figure 4.12 1 in 100 year flood extent (plus climate change) for Baseline and Final Scenario

5. Llangollen Downstream Weir

5.1 Baseline

5.1.1 Site

Site around the weir is illustrated on Figure 5.1. Llangollen downstream weir is situated in Llangollen, set within the valley of the River Dee, at an elevation of approximately 78m AOD. It lies around 300m downstream of Llangollen Upstream weir (Section 4). It has already been partially lowered, however remnants of the structure, approximately 53m wide and 0.8m high, remain.

To the north of the site, is a path and beyond this there is a mixture of residential and commercial properties occupy the landscape to the north with the A539 running parallel to the site. An old leat channel flows from just upstream of the weir and continues to the north of the river.

Upstream of the weir, there are areas of exposed bedrock and shallower water are found here with intermittent tree cover. Llangollen Bridge is around 100m upstream of the weir.

Historic mapping indicates that a weir has been present at the site since at least 1875.

The preferred fish passage option at project onset was to remove the remains of the structure to river bed level and create a natural river channel (partial removal would be a secondary option).

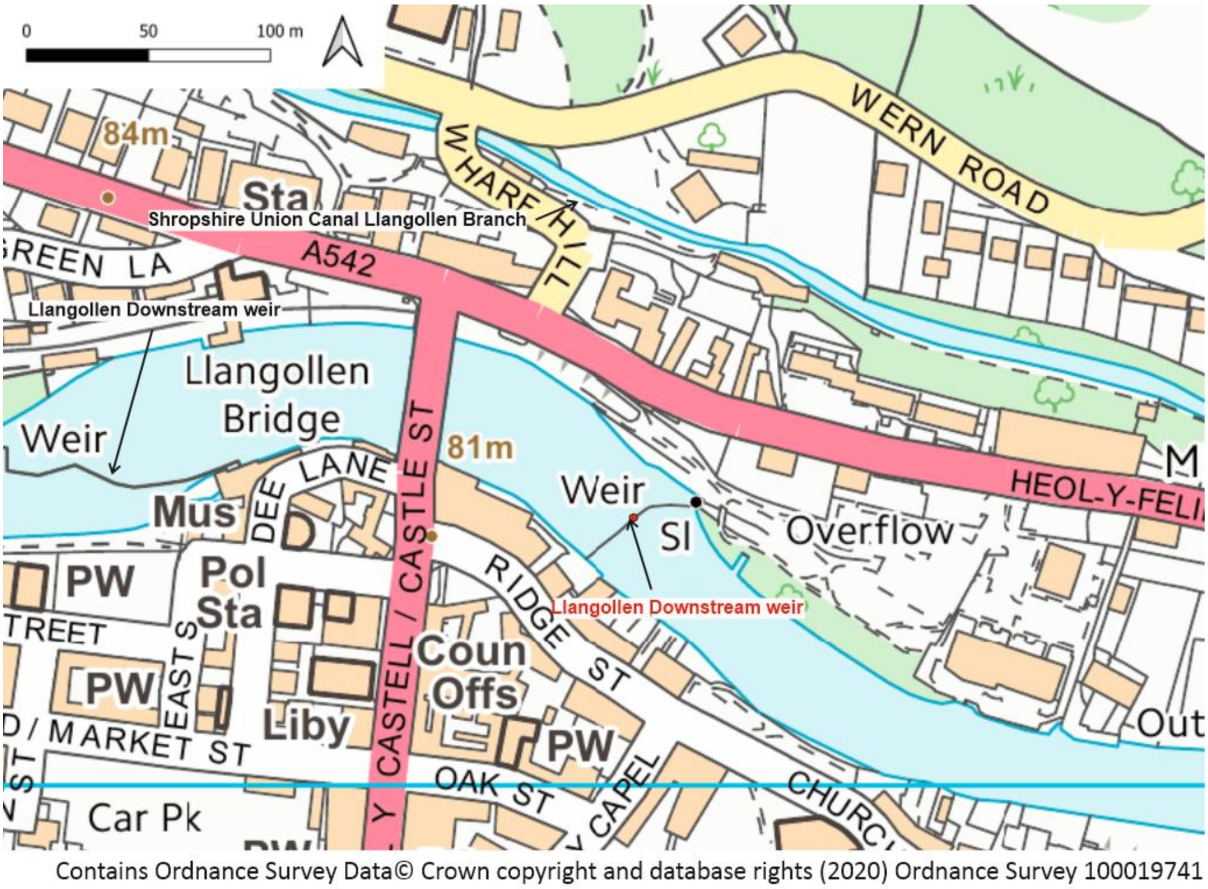

Figure 5.1 Llangollen downstream weir study area

5.1.2 Site Visit

The site was visited on the 17 July 2020. Images from this, complimented by imagery from the topographic survey, are presented in Plates 5-1 through to 5-14.

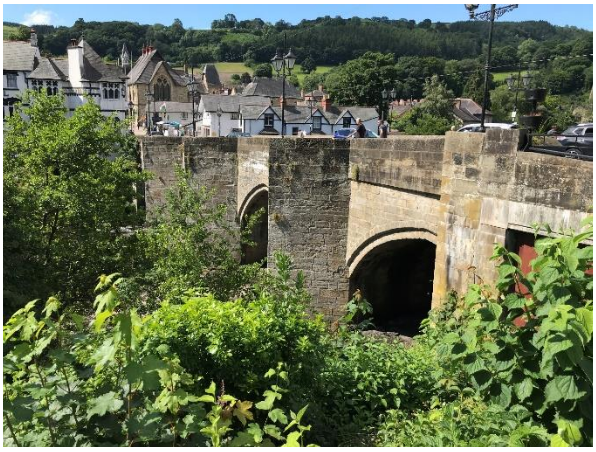

Plate 5-1. Llangollen (Castle Street) Bridge upstream of the weir

Plate 5-2. Bed immediately downstream of Castle Street bridge

Plate 5-3. River Dee looking downstream from Castle Street Bridge

Plate 5-4. Weir from LHB looking across the channel

Plate 5-5. Lower Weir from LHB, looking downstream

Plate 5-6. Weir from RHB

Plate 5-7. Weir from RHB

Plate 5-8. River Dee looking downstream (from RHB)

Plate 5-9. River Dee looking upstream (from RHB)

Plate 5-10. Weir from RHB

Plate 5-11. Weir from RHB (turbulent flows apparent at RHB end/ weir broken up at this location)

Plate 5-12. Weir from RHB

Plate 5-13. Entrance to LHB leat channel (looking from upstream)

Plate 5-14. Leat channel flow control structure (looking from downstream / showing closed footbridge)

5.1.3 Topographic and Bathymetric Survey

A topographic survey of the site and surrounding area was undertaken. This included a survey at and around the Upstream and Downstream weirs, cross sections in the River Dee extending approximately 400m upstream and downstream of each weir (see also Section 4.1.4). The survey was undertaken to refine representation of the system within the hydraulic modelling (see Section4.2.3).

All surveyed points are indicated in Figure 5.2 below while those around the weir itself are indicated in Figure 5.3, showing a reasonable coverage with some gaps where surveying could not be undertaken safely.

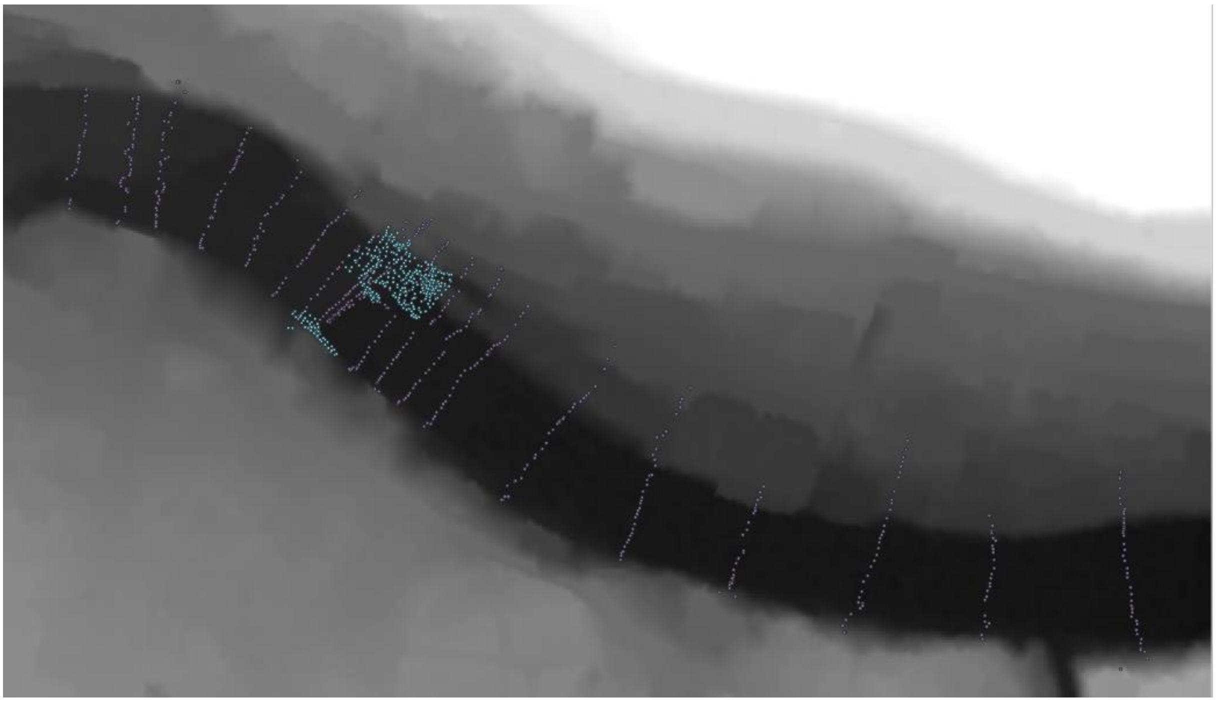

Figure 5.2 Llangollen Downstream Weir xyz topo data received (different colours showing different areas)

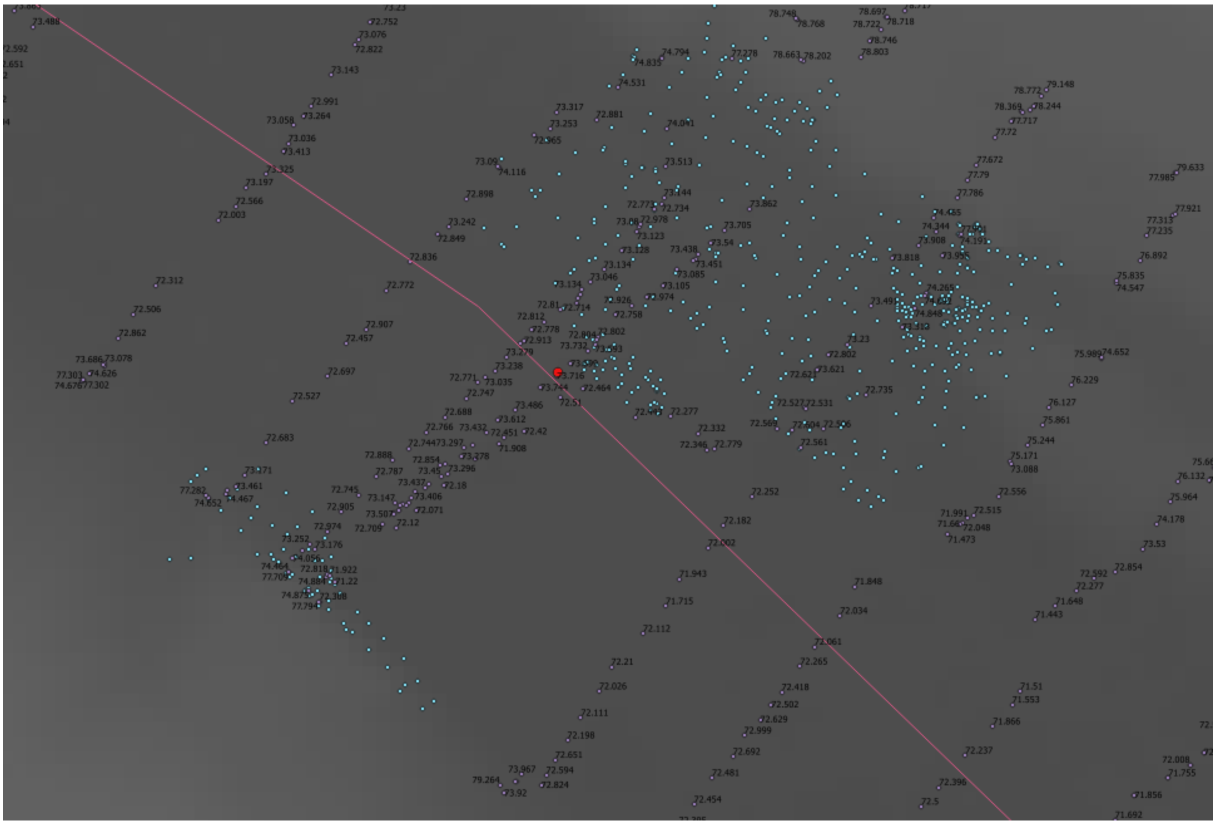

Figure 5.3 Llangollen Downstream Weir xyz data received (zoomed into area around the weir)

5.2 Model Build

5.2.1 Modelling overview

Due to the proximity of both Llangollen weirs, a 2D hydraulic TUFLOW model was constructed encompassing both the upstream and downstream weirs (as well as upstream and downstream of them). The model was constructed using available hydrological data, freely available LiDAR, topographic data and information gathered during the site visit. For the baseline and preferred option scenario, the model was run for the events described in section 2.2.2.

The baseline surface was constructed integrating the topographic data from the survey carried out at the study area, on free available 1 m resolution LiDAR data. Then, the baseline DEM was modified simulating the partial removal of the weir. The model area is described in Section 4.2.1.

5.2.2 Baseline Model

Construction of the DEM

The initial DEM was as described in Section 4.2.3. In order to integrate the topographic survey data into the DEM at and around the downstream weir, the topographic survey data was interpolated using GIS software to create a TIN surface. No iterations between cross sections were needed around Llangollen downstream weir as coverage was considered to be reasonable. Figure 4.6 in section 4.2.3 shows the topographic survey data and the iterations included to construct the surface of the DEM.

The TIN was restricted into the bed channel and merged with the LiDAR by using TuFlow software. Figure 5.4 shows the final baseline DEM around the Llangollen downstream weir.

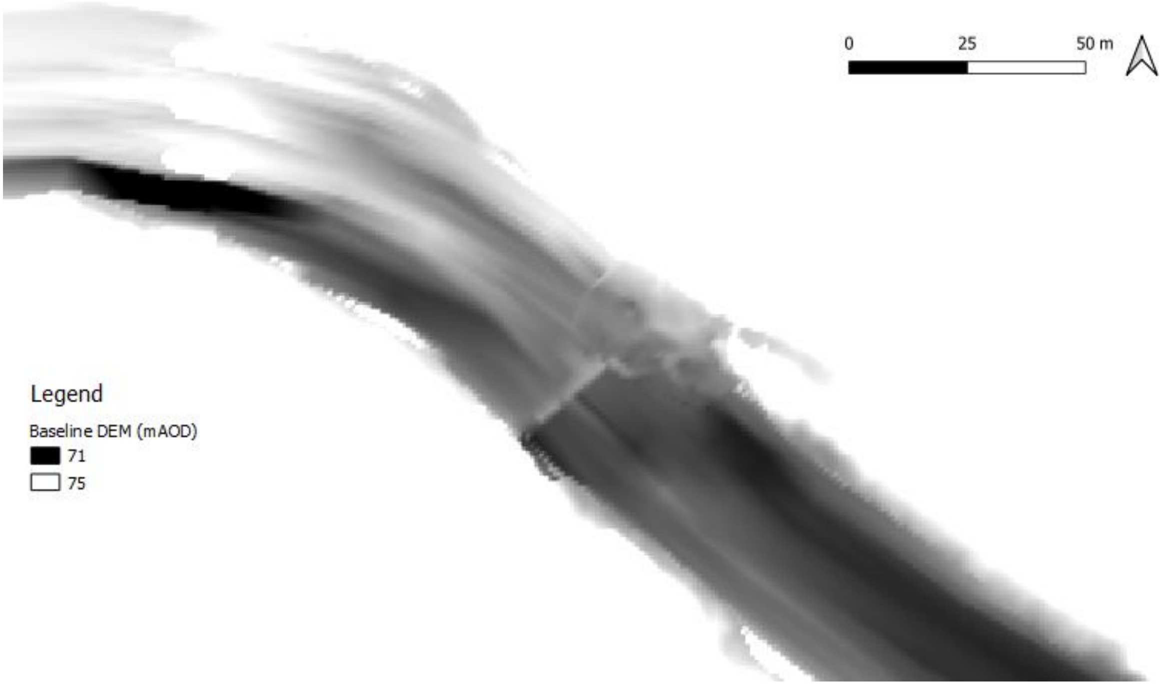

Figure 5.4 Final Baseline DEM around Llangollen downstream weir.

Hydrological assessment of inflows for the model

Flow estimates for the site were as outlined in Section 4.2.3.

5.2.3 Design Scenario

Overview

A design to improve fluvial connectivity at Llangollen Downstream weir was developed. The preliminary preferred option under consideration by NRW was removal of the remains of the structure to the riverbed and creating a natural river channel. Optioneering was undertaken to assess the suitability of this option and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Llangollen Downstream weir fish passage improvement design included:

- Improved passage for primarily for downstream migrating salmonids, lamprey and eel across a range of flows
- Use of the area by kayakers and canoeists, including a deep pool located in immediately upstream of Llangollen Bridge at Town Falls
- Area of historical interest
- Limited visual impact including avoidance of any remaining weir drying at low flows
- No increase to public safety risk or flood risk
- Maintenance

Partial removal of 8m, 4m and 2m and full removal of the weir were assessed to evaluate the effect of the preferred option. After this assessment a model with 4m partial removal of Llangollen Downstream weir was selected.

Partial removal design

To design the partial removal of the weir, the DEM was modified in a 4m wide section on the downstream weir. This section was regarded to 72.2m AOD matching the levels upstream and downstream of the weir. Partial weir removal representation in the model DEM is shown in Figure 5.5 below.

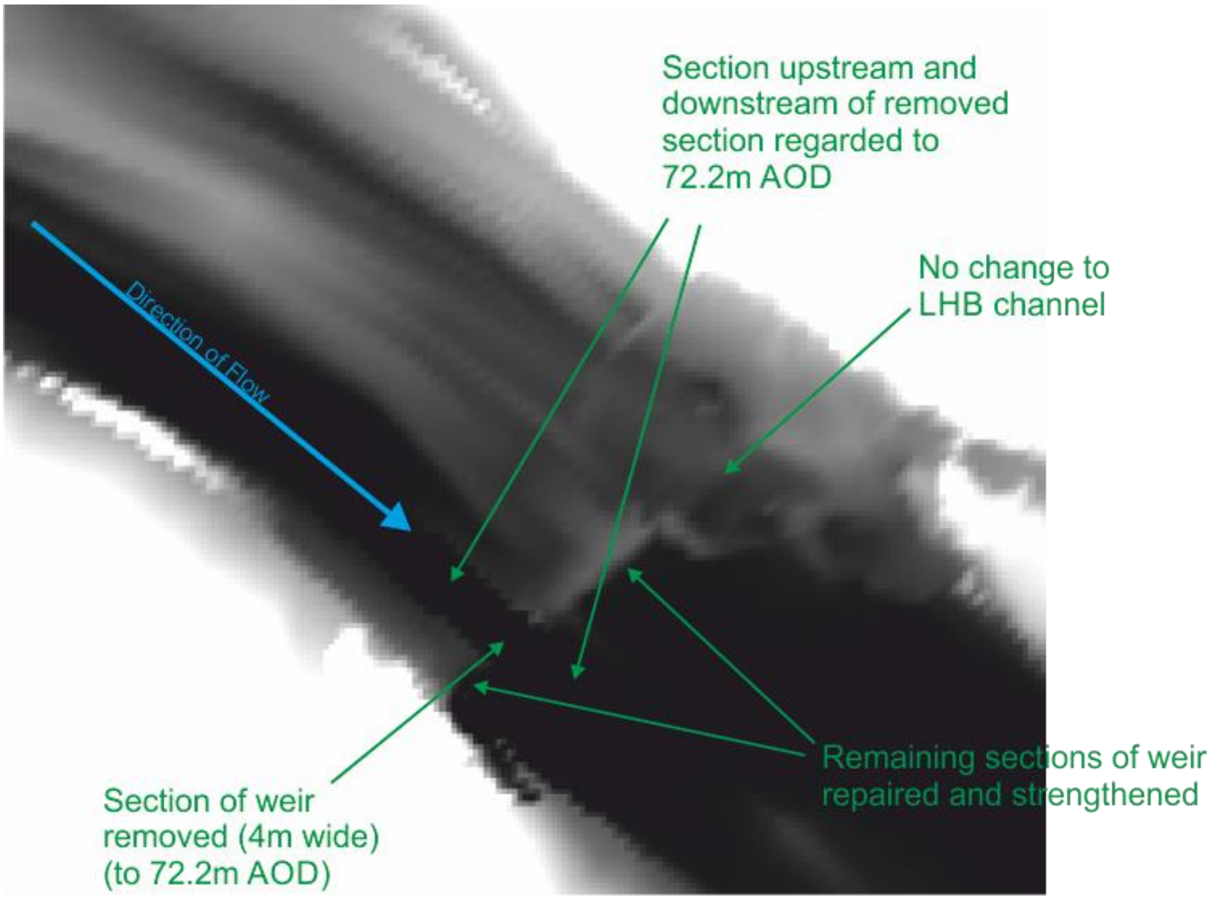

Figure 5.5 DEM for the partial removal of Llangollen Downstream weir scenario

5.3 Results

Plots of velocities and depths for various flows under the baseline and 4m partial removal scenarios are presented in Figure 5.6 and Figure 5.7, respectively. Average water depth and approximate maximum velocities at Q_{95} - Q_{10} in the area of partial removal were sufficient for downstream migrants, providing that detailed design ensures gradual and smooth flow acceleration towards the area. Upstream migrating adult salmonids, as well as some coarse fish species undertaking local migrations, are anticipated to be able to ascend through the area of weir removal at Q_{95} and Q_{10}[9], however another route of passage exists along the LBH during flows where velocities exceed maximum swimming capabilities of individuals, such as potentially at Q_{50}.

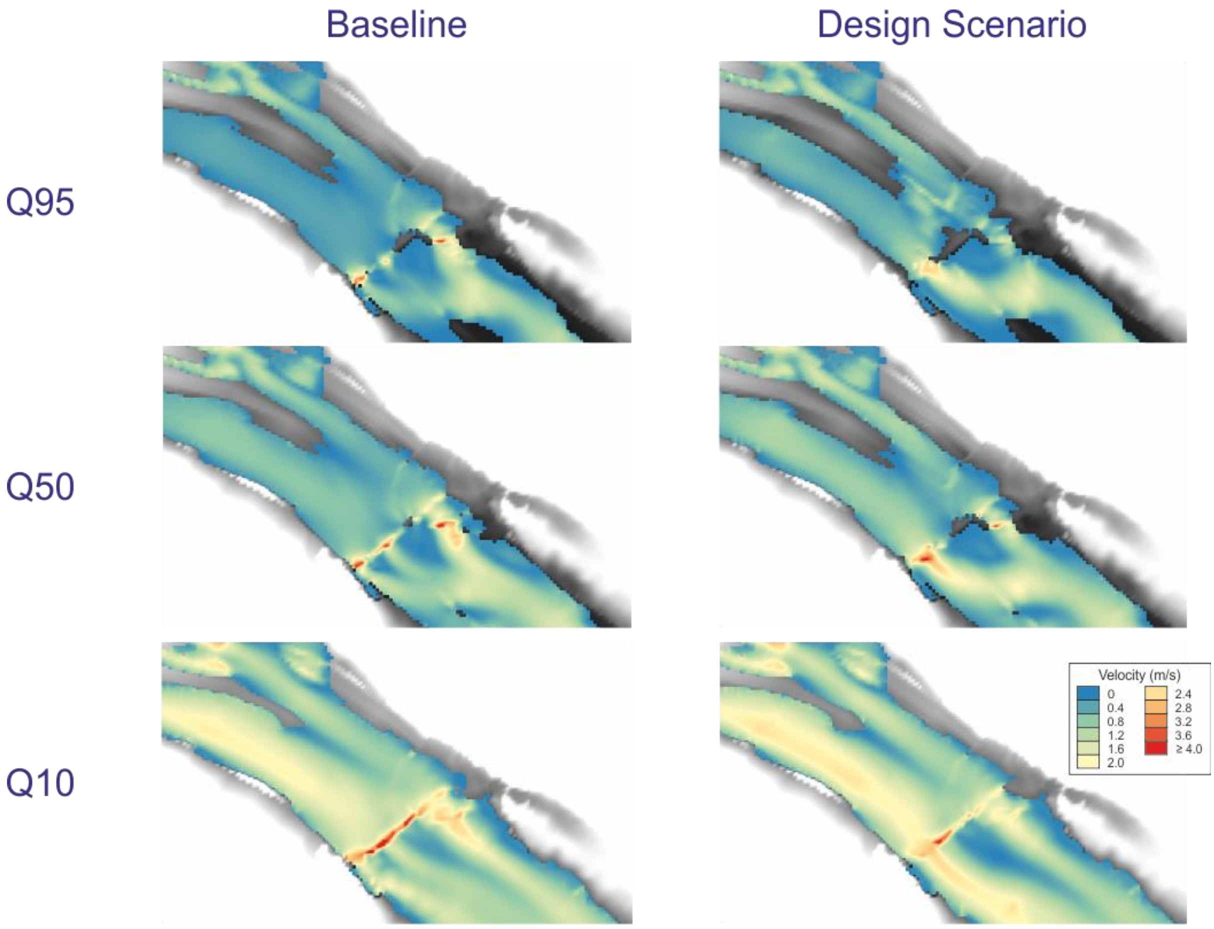

Figure 5.6 Velocity results for various flows under the baseline and design scenarios

[9] Armstrong, G.S., Aprahamian, M.W., Fewings, G.A., Gough, P.J., Reader, N.A. and Varallo, P.V., 2010. *Environment Agency Fish Pass Manual: Guidance notes on the legislation, selection and approval of fish passes in England and Wales.* Environment Agency, Rio House, Bristol http://publications. environment-agency. gov.uk.

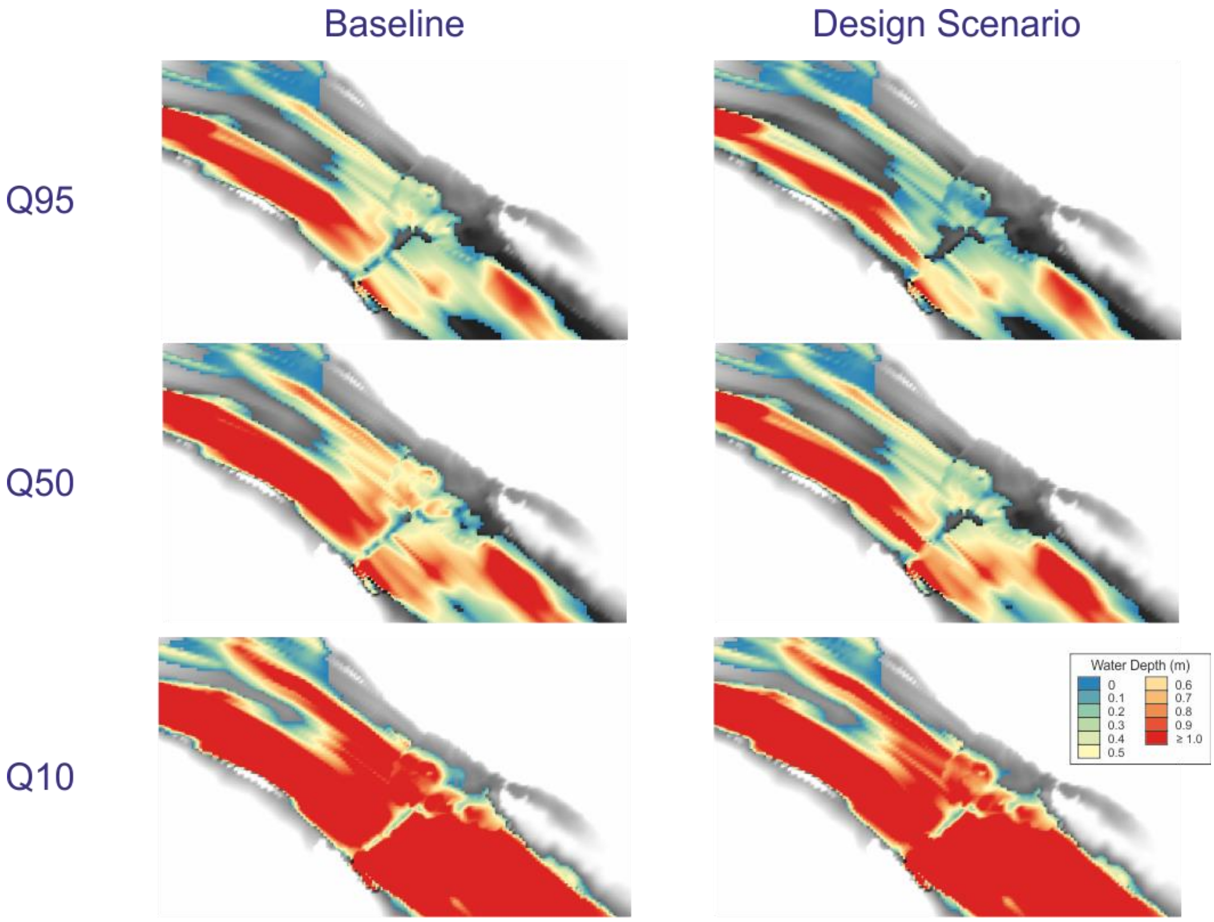

Figure 5.7 Depth results for various flows under the baseline and design scenarios

No change in the extent or depths of flooding is predicted other than where the dimensions of the weir itself are altered (see Figure 5.8).

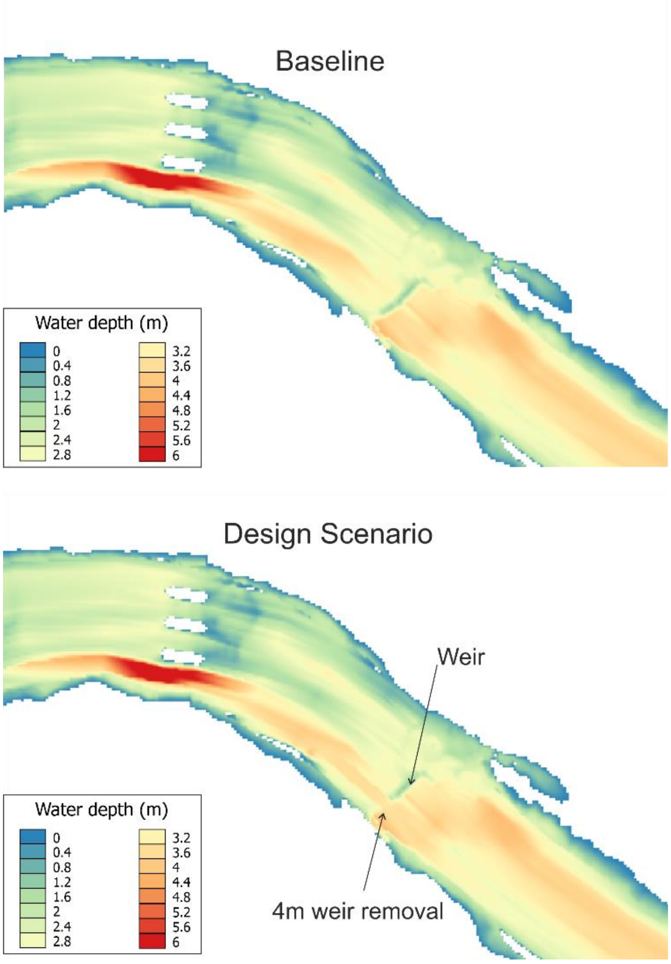

Figure 5.8 1 in 100 year flood extent (plus climate change) for Baseline and Final Scenario

6. Morlas Ford Weir

6.1 Baseline

6.1.1 Site

Site around the weir is illustrated on Figure 6.1. The weir is located on the Morlas Brook, a tributary of the Afon Ceiriog. It is located approximately 2km to the west of the town of Chirk and at an elevation of approximately 55m Above Ordnance Datum (AOD). It is centred on National Grid Reference (NGR) SJ31190 38319.

The brook and river through the study area form the border between Wales and England, while the site in the vicinity of the Denbighshire hamlet of Pont-y-blew. The nearest significant settlement to the site is the small Denbighshire town of Chirk, approximately 1.8km to the west.

The weir is a concrete plinth that forms a ford across the river. Some disrepair at its downstream end has been reported. The weir is approximately 5m wide and has a head of about 1m (with downstream perching also reported). The riparian area on the left bank is agricultural while the right bank is woodland.

At project onset, the preferred option was removal of the weir with river restoration also to occur. Access was to be provided through the replacement of the ford with a clear span bridge.

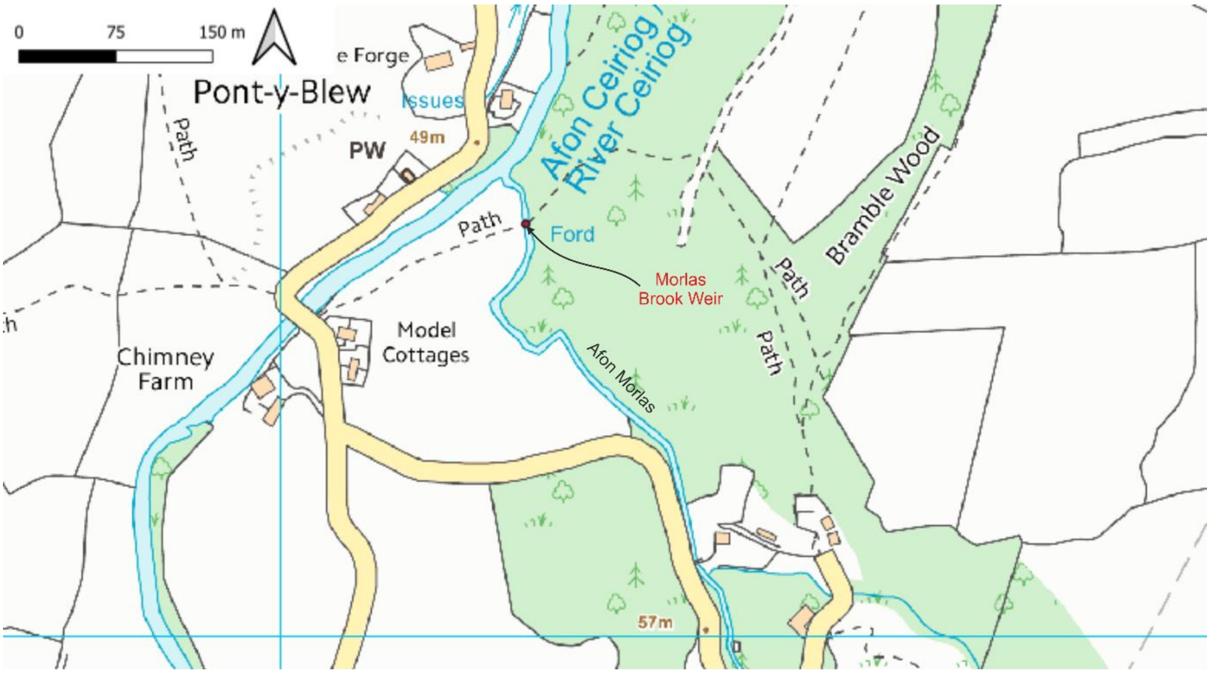

Figure 6.1 Morlas ford weir study area

6.1.2 Site Visit

The site was visited on the 16 July 2020. Images from this are presented in Plates 6-1 through to 6-10. Site measurements suggest the head difference at the downstream end of the structure was around 0.5 - 0.7m, while depths above it on the day of the survey were less than 0.1m, which would inhibit fish passage.

Plate 6-1. Afon Ceiriog upstream of the confluence with the Afon Morlas

Plate 6-2. Afon Ceiriog upstream of the confluence with the Afon Morlas

Plate 6-3. Afon Morlas upstream of the ford

Plate 6-4. Bottom RBH edge of the ford

Plate 6-5. Morlas Ford looking downstream

Plate 6-6. Upstream of the ford, showing sediment, gravels and stone build up

Plate 6-7. Level measurement downstream of the ford (also showing depth of structure)

Plate 6-8. Level measurement downstream of the ford (also showing depth of structure)

Plate 6-9. Afon Ceiriog downstream of the confluence with Afon Morlas

Plate 6-10. Afon Ceiriog downstream of the confluence with Afon Morlas (view from road bridge)

During the visit we were advised of, or observed, the following:

- The estate put the ford in at least 20 years ago.
- The weir is considered to be a major barrier to all species.
- Ford removal is considered to be the preferred option at the site, and this will enable multiple species passage upstream and downstream.
- The estate still want access across the river and installation of bridge across the brook anticipated.
- The ford is part of a Public Right of Way (PRoW) and an alternative route should be sought and confirmed. Local authorities across the border are Shropshire and Wrexham, and NRW are to consult them on the scheme and PRoW.
- NRW expected that there would be no contaminants in the materials that have built up behind the ford, which generally appeared to be gravels interspersed with fines.
- Flood risk is not likely to be an issue at the site with nearby property being at a notably higher elevation. Regardless of this, removal of the weir could possibly reduce flood risk.
- There is uncertainty over the adjustment period of the river following removal. The prospect of having a design that helps to control this in the short term, with components being removed in the future, was mentioned. This would necessitate the inclusion of an afterlife / maintenance plan.
- There is reportedly a water main around the weir that would need to be accounted for.

6.1.3 Topographic and Bathymetric Survey

A topographic survey of the site and surrounding area was undertaken. This included a survey at and around the weir, cross sections in Morlas Brook upstream and downstream of the weir as well as sections in the Afon Ceiriog. The survey was undertaken to refine representation of the system within the hydraulic modelling (see Section 6.3.3). An overview of the topography at and around the site is provided in Figure 6.2 below. This confirms the estimate of head drop at the weir as measured coarsely during the site visit and also the side channel on the left-hand side downstream of the weir, with gravel deposit in between.

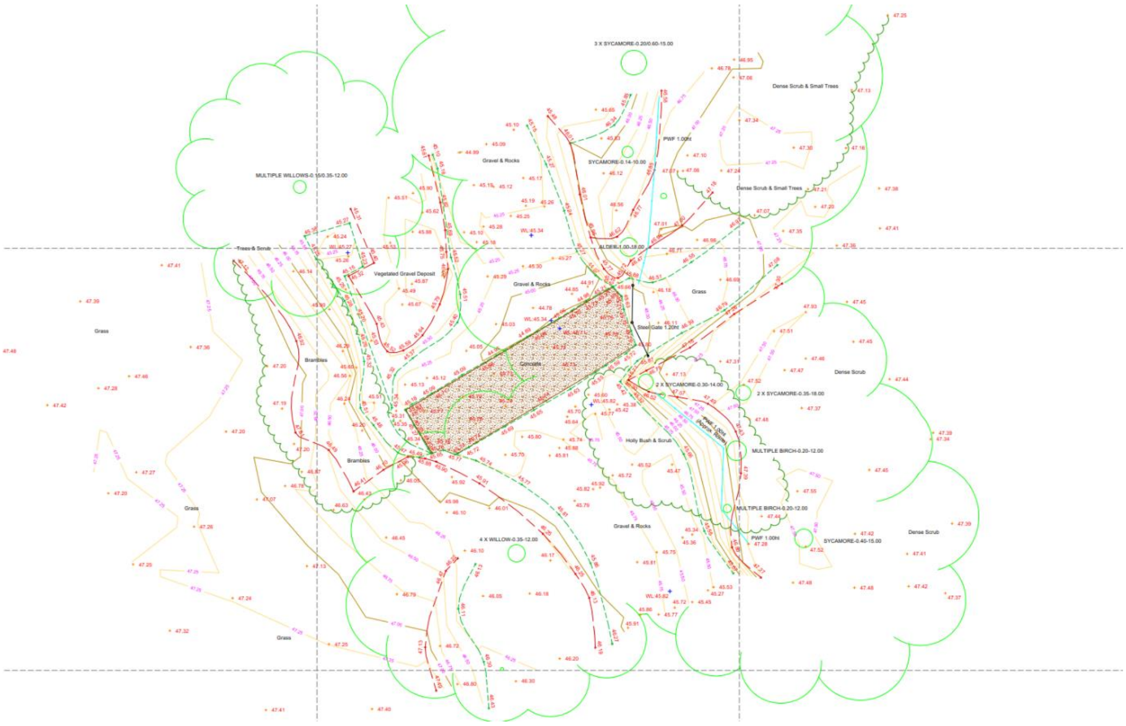

Figure 6.2 Topographic Survey at and surrounding Morlas Brook weir

The collected topography layer was integrated into the LiDAR data of the surrounding area to improve representation of Morlas Brook and Afon Ceiriog in the Digital Elevation Model (DEM) that was to be used in the hydraulic modelling. A summary of the final DEM is provided in Figure 6.3 below. This shows levels in the area around 44 to 50m AOD. Levels rise quicker on the right-hand bank of Morlas Brook.

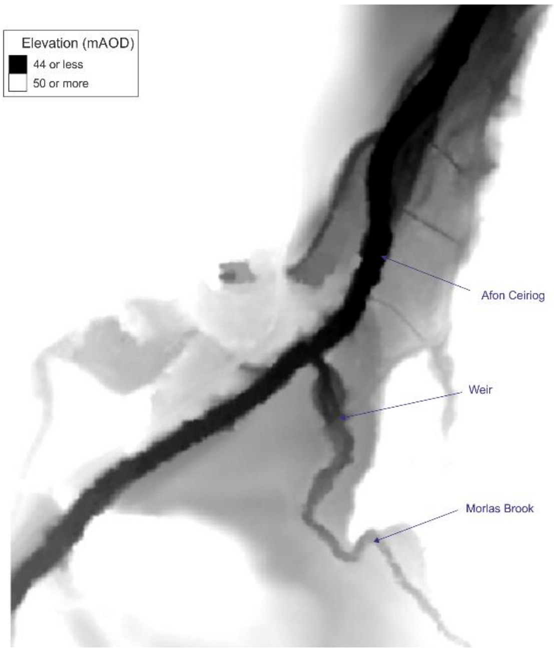

Figure 6.3 Topographic survey at and surrounding Morlas Brook weir

6.2 Design Scenario

A design to improve connectivity at Morlas weir was developed. The preliminary preferred option under consideration by NRW was removal of the ford and restoration of the river channel with access to the opposite bank continuing through replacement with a clear span bridge. Optioneering was undertaken to assess the suitability of this option and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Morlas weir fish passage improvement design included:

- Improved passage for upstream and downstream migrating salmonids, lamprey and eel across a range of flows
- Use of the area by kayakers and canoeists
- No increase to public safety risk or flood risk
- Maintenance

The preliminary option was considered the best and recommended option for improving ecological connectivity including for all target species and life stages.

6.3 Model Build

6.3.1 Modelling overview

A 2D hydraulic TUFLOW model was constructed covering the area of interest at Morlas weir. The model was constructed using available hydrological data, freely available LiDAR, topographic data and information gathered during the site visit. For the baseline and Preferred option scenario, the model was run for the events described in section 2.2.2.

The baseline surface was constructed integrating the topographic data from the survey carried out at the study area, on free available 1 m resolution LiDAR data. Then, the baseline DEM was modified simulating the ford removal and channel regrading.

6.3.2 Model area

The area to be modelled encompasses the area where the topographic survey was undertaken. The boundaries upstream of the model are defined by SJ 31350 37950 at Afon Morlas, upstream of Morlas Brook weir and SJ 31050 38250 at Afon Ceiriog, upstream of the confluence with Afon Morlas.

The downstream limit of the modelling area is defined by SJ 31269 38566, at Afon Ceiriog, downstream with the confluence with Afon Morlas. These limits are illustrated on Figure 6.4 below.

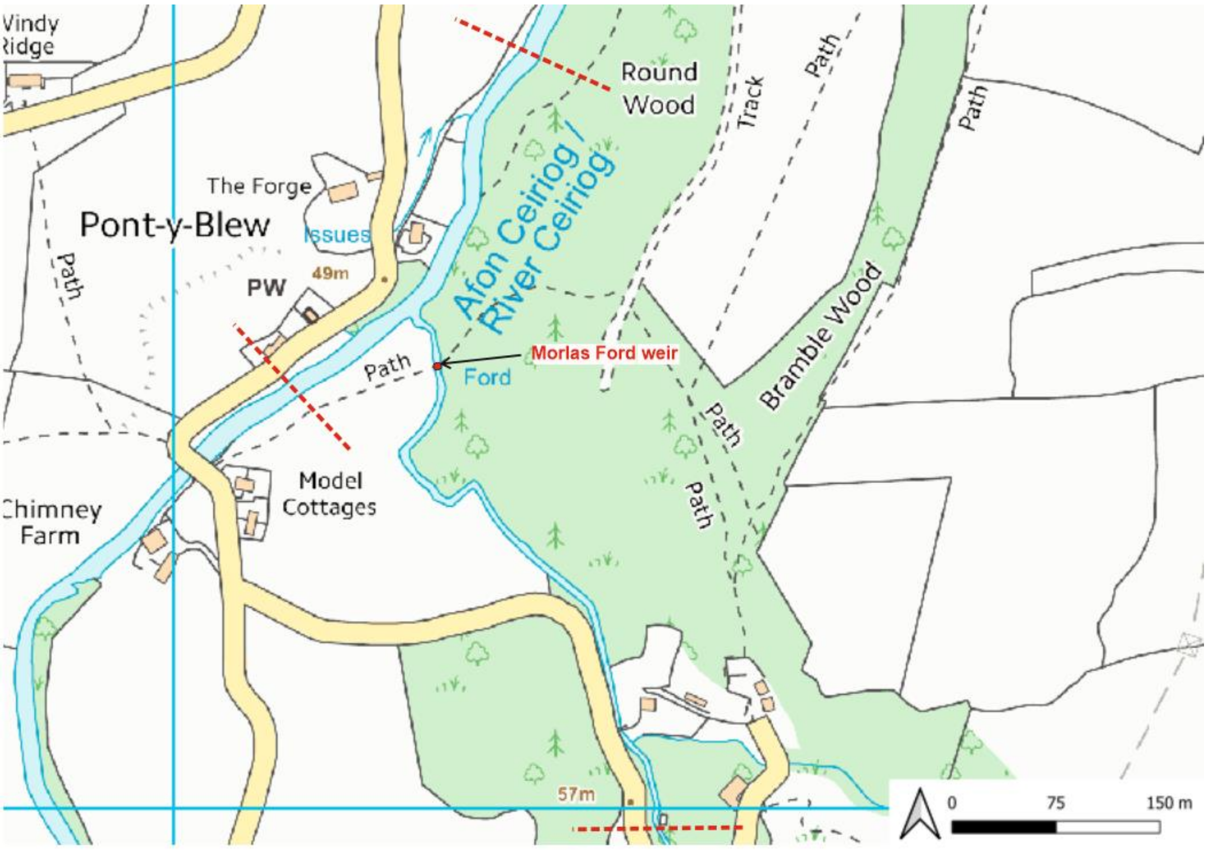

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Figure 6.4 Morlas Ford Weir model area.

6.3.3 Baseline Model

Construction of the DEM

Initial data to build the modelling surface included free available 1m resolution LiDAR data and the topographic survey. The topographic survey included the following data:

- Survey at an around the weir;
- Cross section in Morlas Brook, upstream and downstream of the weir; and
- Cross Section in Afon Ceiriog, upstream and downstream of the confluence with Afon Morlas.

A DEM was constructed by integrating the topographic survey data on the available LiDAR data. To integrate the topographic survey data into the DEM, the topographic survey data was interpolated using GIS software to create a TIN surface. An initial review of the TIN created with the data available indicated that, due to the looped morphology of both watercourses, iterations between cross sections were needed to ensure the surface matched the morphology (bends and curves) along Afon Ceiriog and Afon Morlas. Once the TIN constructed from topographic survey data and the iterations matched the watercourse morphology (bends and curves were well represented), the TIN was restricted into the bed channel and merged with the LiDAR by using TuFlow software. Figure 6.5 shows, on the left, the LiDAR and the location of the topographic cross sections and the iterations included to construct the surface; and on the right, the final baseline DEM.

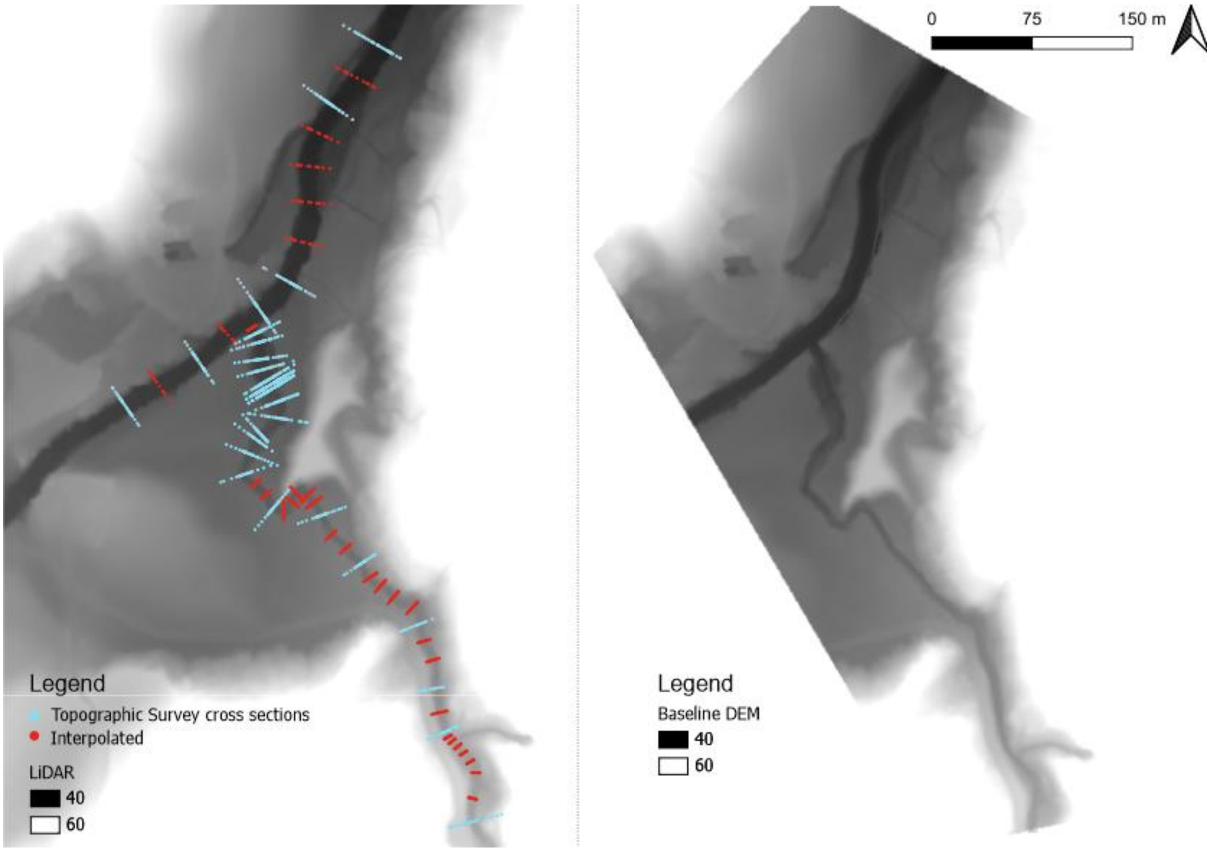


Figure 6.5 Integration of Topographic Survey data into LiDAR and final Baseline DEM.

Hydrological assessment of inflows for the model

Typical and flood flow estimates for Morlas Brook and flood flow estimates Afon Ceiriog were as outlined in Section 2.2.2. Figure 6.6 shows the hydrograph associated with the flood events that were modelled. In addition typical flow estimates for the Afon Ceiriog were derived through catchment apportioning from the gauge (Ceiriog at Brynkinalt Weir) close upstream of the study area. These are shown in Table 6.1 below.

Table 6.1 Summary of flow hydrology for Morlas Ford Weir model.

Watercourse	NGR	Catchment area (km^2)	Flow Statistics (m^3/s)		
			Q_{95}	Q_{50}	Q_{10}
Afon Ceiriog	SJ 31050 38250	115.1	0.47	2.13	6.93

1 in 2 year event **1 in 100 year + CC event**

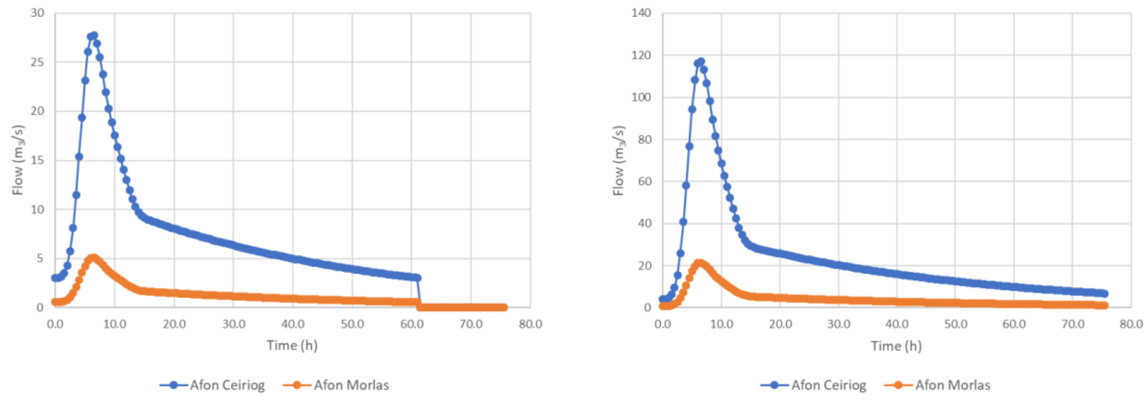


Figure 6.6 Flood events hydrographs for Morlas Brook Weir model.

6.3.4 Design Scenario

Overview

A design to improve connectivity at Morlas weir was developed. The preliminary preferred option under consideration by NRW was removal of the ford and restoration of the river channel with access to the opposite bank continuing through replacement with a clear span bridge. Optioneering was undertaken to assess the suitability of this option and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Morlas weir fish passage improvement design included:

- Improved passage for upstream and downstream migrating salmonids, lamprey and eel across a range of flows
- Use of the area by kayakers and canoeists
- No increase to public safety risk or flood risk
- Maintenance

The preliminary option was considered the best and recommended option for improving ecological connectivity including for all target species and life stages

Ford removal design

As mentioned in section 2.2.2. the preferred option for Morlas was the full removal of the weir and regrading of the channel upstream and downstream. Also, it was decided to infill the left secondary channel located downstream of the weir.

To simulate the weir removal, it was indicated on the model to interpolate the baseline DEM levels upstream and downstream of the weir. To infill the secondary channel, the procedure was similar, but in this case, the levels were defined by the channel right and left hand banks. Baseline and scenario DEM where then compared and checked through XS1 to XS6 Figure 6.7 below indicates, on the right, the area where the modifications were done and the assessment XS locations; and the scenario DEM, on the left.

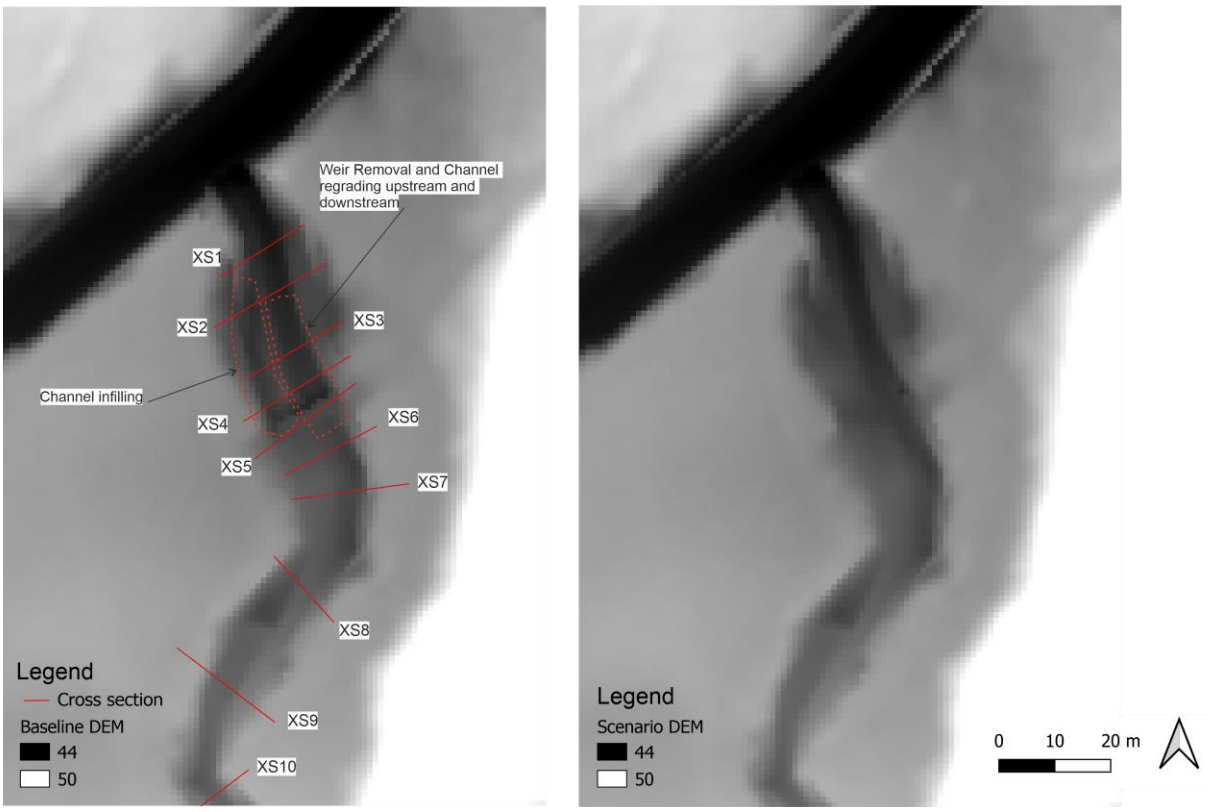


Figure 6.7 Baseline DEM modification and assessment XS location (left); and Scenario DEM (right).

Baseline and Scenario DEM was assessed on XS2 to XS6. Figure 6.8 show the result obtained. Looking XS2, XS3 and XS4, it is observed the left secondary channel has been correctly infilled avoiding then the flow through this channel. Also, the right side of XS4 shows the levels on the scourpool. It is observed that on this side, the channel was infilled due to the weir removal and regrading. Upstream, XS5 is located on Morlas Weir. The cross section shows the current weir level and the channel resulted after the removal. Finally, differences observed on XS6 are due to the channel regrading.

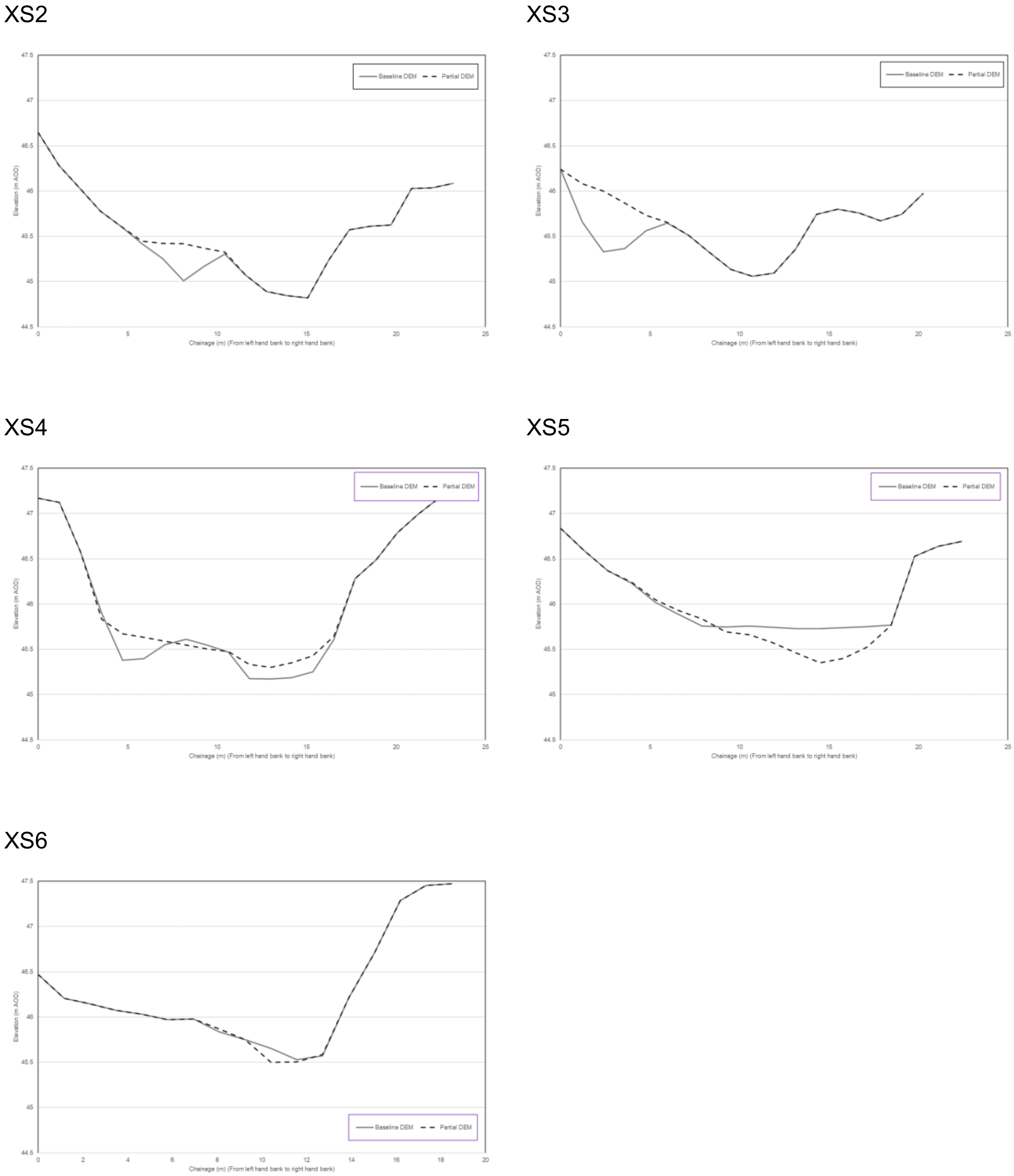


Figure 6.8 Baseline and Scenario DEM on XS2 to XS6

Long section profile along Afon Morlas is represented on Figure 6.9. The section shows the the reduction in elevation at the weir removal scenario along with the profile regrading.

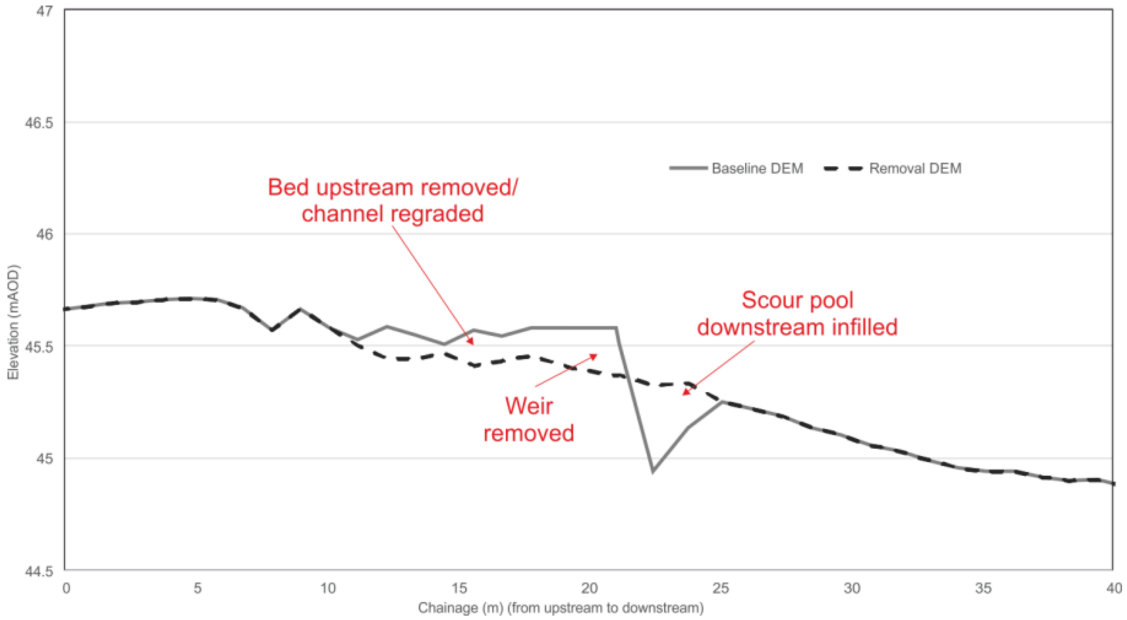


Figure 6.9 Long Profile around the weir under the baseline and removal scenarios

6.4 Results

Plots of velocities and depths for various flows under the baseline and partial removal scenarios are presented in Figure 6.10 and Figure 6.11, respectively. A long profile is also provided in Figure 6.12. Average water depth and velocity at Q_{95} - Q_{10} (Table 6.2) in the area of weir removal were sufficient for downstream migrating juvenile, and upstream migrating adult, salmonids as well as coarse fish species undertaking local migrations[10]. Weir removal should also enable passage for eel and lamprey.

[10] Armstrong, G.S., Aprahamian, M.W., Fewings, G.A., Gough, P.J., Reader, N.A. and Varallo, P.V., 2010. *Environment Agency Fish Pass Manual: Guidance notes on the legislation, selection and approval of fish passes in England and Wales.* Environment Agency, Rio House, Bristol http://publications. environment-agency. gov.uk.

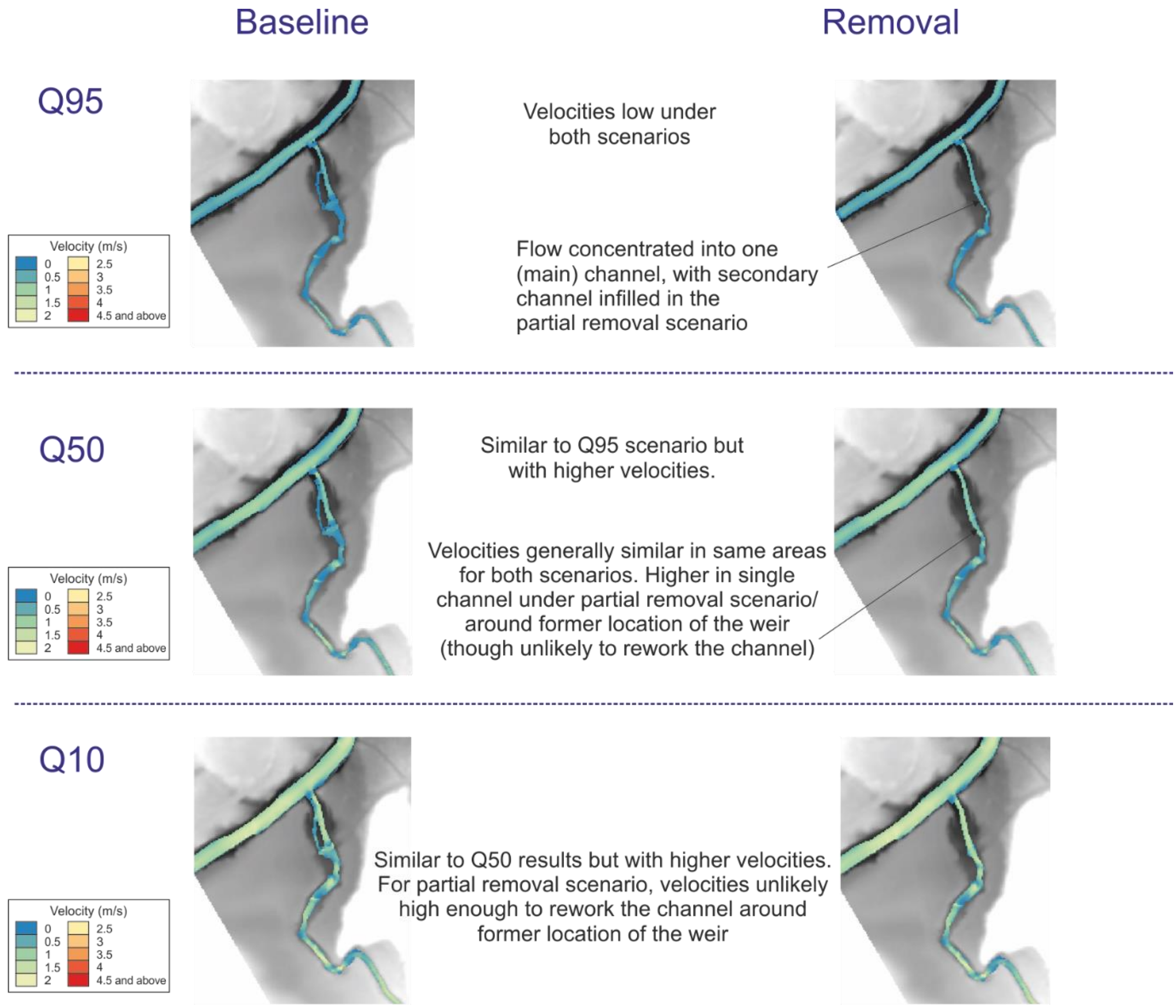


Figure 6.10 Velocity results for various flows under the baseline and weir removal scenarios

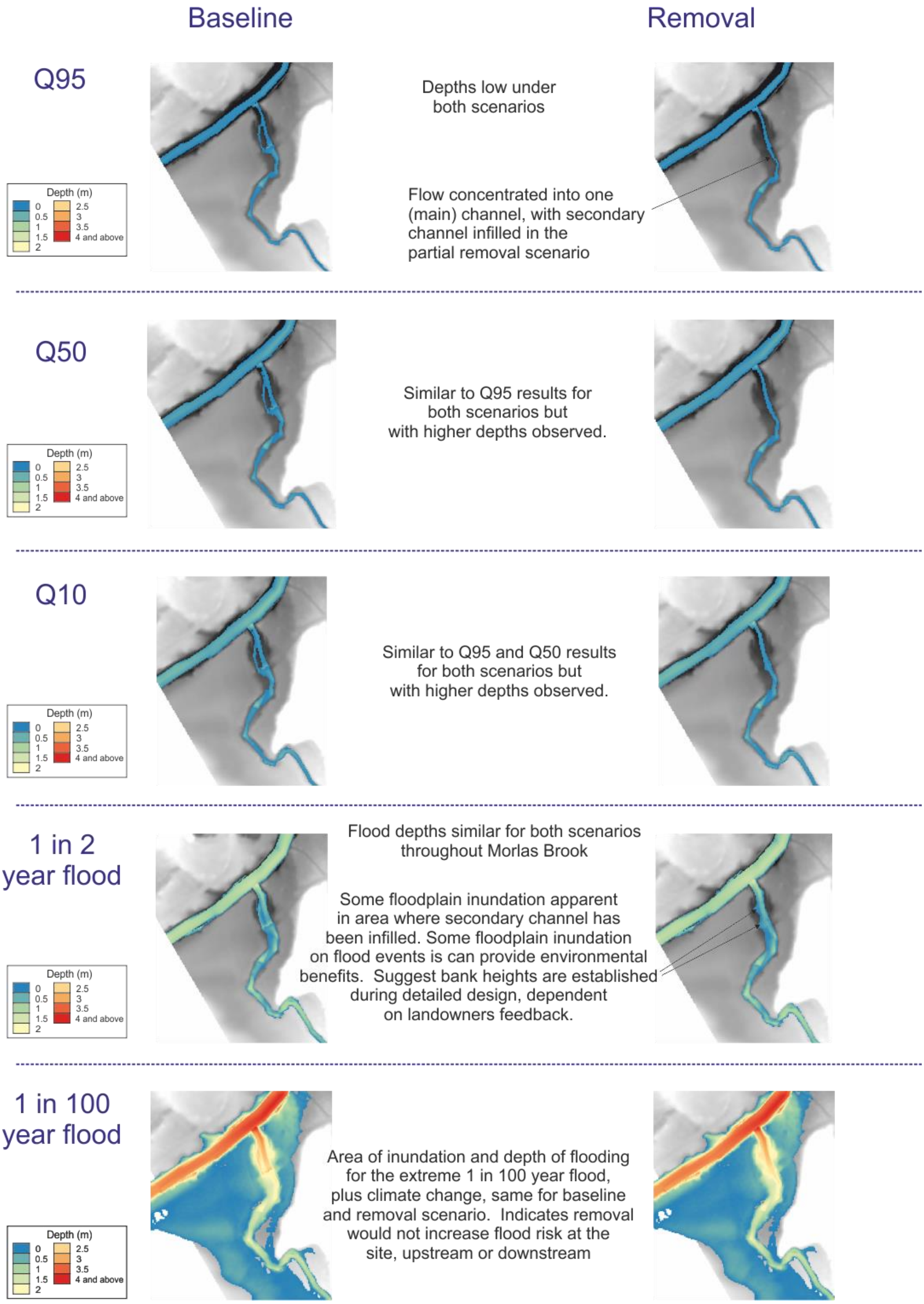


Figure 6.11 Depth results for various flows under the baseline and weir removal scenarios

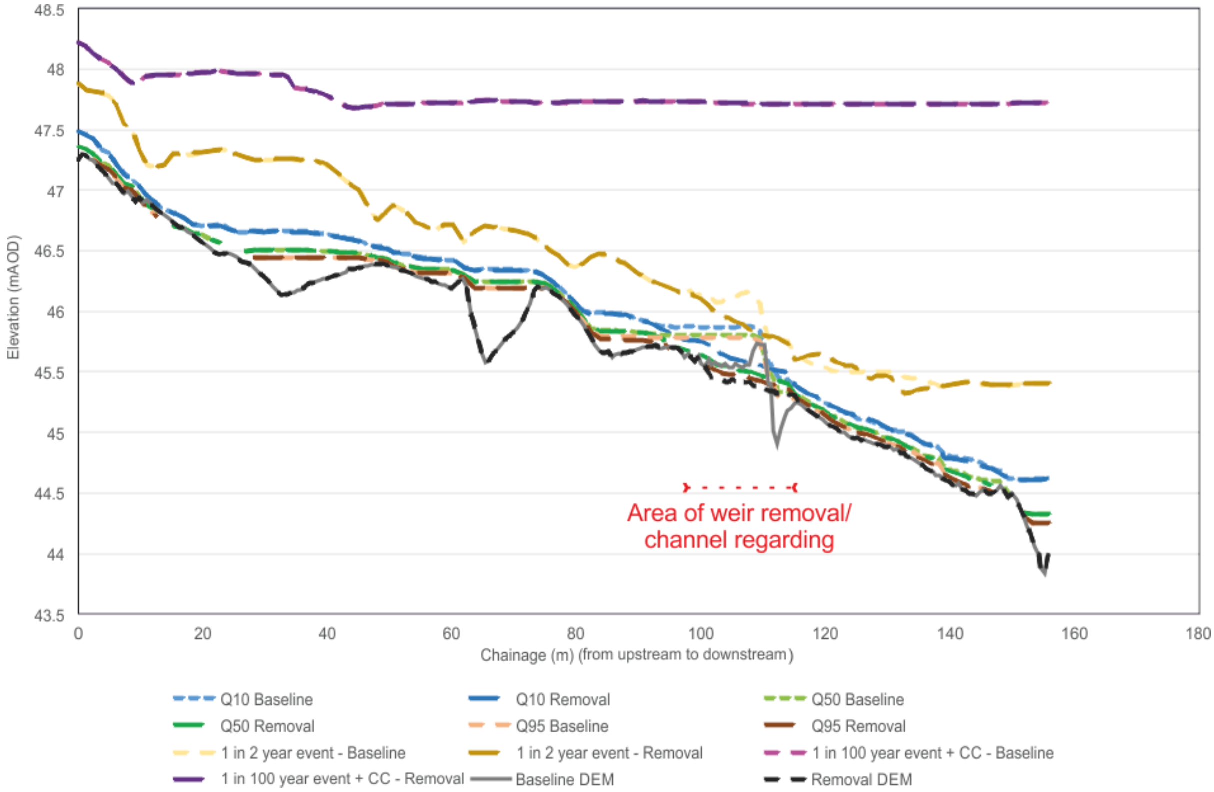


Figure 6.12 Central long profile with water levels for various flows under the baseline and weir removal scenarios

Table 6.2 Approximate maximum baseline and design (full weir removal) scenario velocities and depths at Q_{95}, Q_{50} and Q_{10} flows

Flow	Velocity (m/s)		Depth (m)	
	Baseline	Design Scenario	Baseline	Design Scenario
Q_{95}	0.5	0.5	1.0	0.5
Q_{50}	0.5	1.0	1.0	0.5
Q_{10}	0.5	1.5	1.0	1.0

Results also indicated no significant adverse hydromorphological effects as a result of weir removal. The regarding that has been included is seems sufficient to buffer from any significant channel adjustment though we suggest that this is considered further (either through detailed design or through geomorphological clerk of works during construction).

The plan of flood risk extents indicate that the scheme would not result in flood risk at the weir or upstream or downstream of it.

7. Erbistock Weir

7.1 Baseline

7.1.1 Site

Site around the weir is illustrated on Figure 7.1. Erbistock weir is associated with a historical mill that was once a listed structure. The building is functional and used for holiday lets, but is no longer used for milling. There are other buildings close by that are still be listed, located approximately 450m north-west, 375m north and 150m to the south-west of the Site.

The site is situated on the River Dee in north-east Wales, approximately 700m north of Erbistock on the east side of a road running south off the A528 from Overton Bridge. The weir is centred on NGR 335446, 342164.

A mill leat channel is located between the building and the weir. A water wheel is present in the leat channel but does not currently function. The weir is a steep stone-faced weir approximately 2.5m high and 70m wide placed at an angle to the flow as it exits a bend in the river. An existing baulk fish pass and modified sloped concrete apron is located on the right-hand bank (RHB) of the weir. There is a breach in the downstream face of the weir that reportedly occurred in 2019.

At project onset the preferred option was partial removal to bed level of approximately 30% of the weir (central section removed) and to maintain the remaining section of the weir structure on the left bank for the landowner. The removal may be extended to the failed section of the weir (leaving approximately 30% of the weir width intact) following further discussions with the landowner.

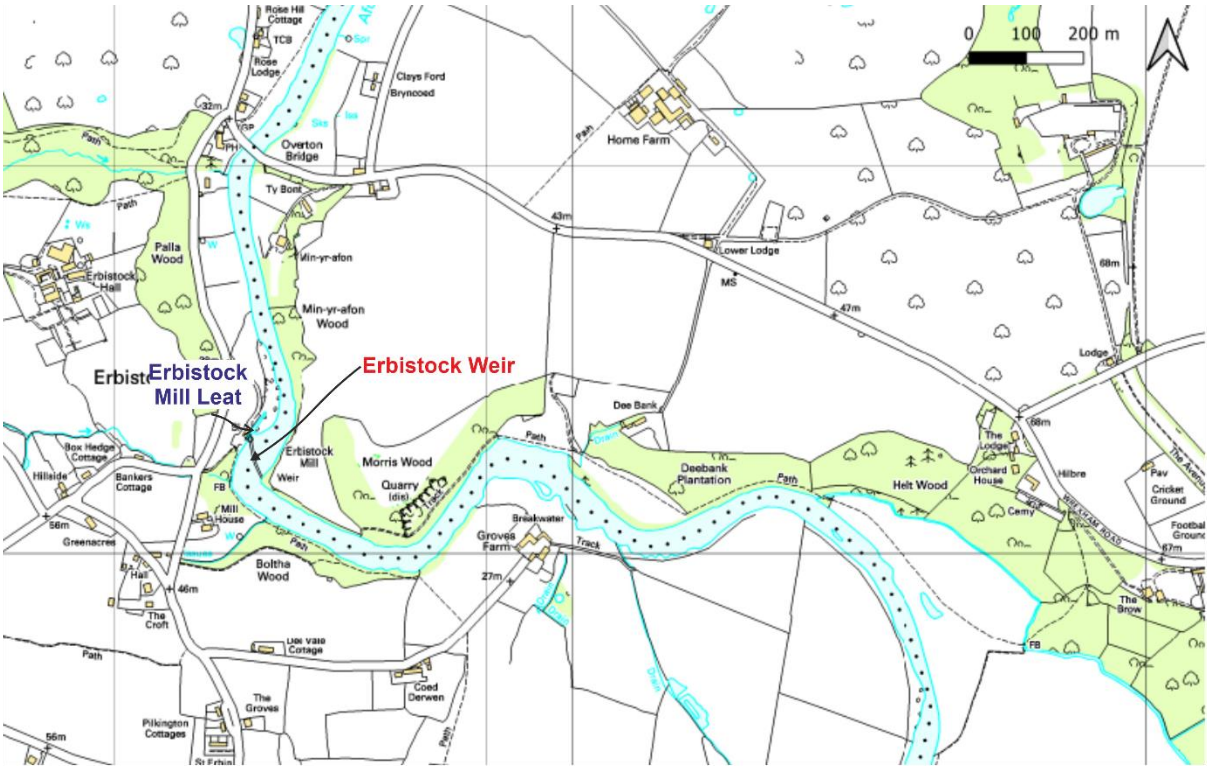


Figure 7.1 Erbistock weir study area

7.1.2 Site Visit

The site was visited on the 17 July 2020. Images from this are presented in Plates 7-1 through to 7-25.

Plate 7-1. River Dee downstream of the weir (from LHB)

Plate 7-2. River Dee downstream of the weir (from LHB) looking downstream

Plate 7-3. Residential property decking beside the river. Leat channel wheel also apparent

Plate 7-4. River Dee downstream of the weir (looking downstream/ from LHB sediment deposit)

Plate 7-5. Erbistock weir from LHB

Plate 7-6. Broken up section toward the LHB side of the weir

Plate 7-7. Erbistock weir from concrete apron on RHB side

Plate 7-8. River Dee upstream of the weir (from LHB)

Plate 7-9. River Dee upstream of the weir (from LHB)

Plate 7-10. Gravel deposit LHB/ approximately 600m upstream of the weir

Plate 7-11. Gravel deposit LHB/ approximately 600m upstream of the weir

During the visit NRW advised us of, or we could observe, the following:

- The weir is in a poor condition with parts of it notably disintegrated.
- An estates rental building (former mill) is located on the left-hand bank and neighbouring the weir. We were advised that this area flooded in February / March 2020 with flows going over the decking and back into the river downstream.

- In the same area there is a leat to the mill with a redundant water wheel. The estate have requested that this is sealed up as part of the design. To date, the estate have already undertaken some work to achieve this but some flow still gets through.
- On the right-hand bank there is an existing baulk fish pass which isn't achieving fish passage. There is a sloped concrete apron between the existing fish pass and the right-hand bank.
- Tenant famers operate the farms on the right-hand bank side (owned by the estate).
- Partial removal of the weir is proposed, leaving a small section beside the property. The estate want to maintain some of the weir structure and propose that removal is completed along the section from the most extensively broken up area through to the right-hand bank concrete apron. The concrete apron would be retained.
- NRW confirmed to the estate owners that they would take responsibility for the site through the works, but management of the structure would return to the owners on completion of construction.

7.1.3 Topographic and Bathymetric Survey

A topographic survey of the site and surrounding area was undertaken. This included a survey at and around the weir, cross sections in the River Dee upstream and downstream extending approximately 600m and 400m from the weir, respectively. The survey was undertaken to refine representation of the system within the hydraulic modelling (see Section 7.2.3).

Surveyed points around the weir itself are indicated in Figure 7.2 below, which shows a reasonable coverage with some gaps in the central weir, which could not be surveyed safely. All surveyed river points are shown in Figure 7.3 below along with an indication of the difference between the measured levels and that in LiDAR data. This shows quite stark differences, particularly upstream of the weir showing the survey depths to be 2m or more deeper than represented in the LiDAR (ultimately showing the benefit of undertaking topographic surveys).

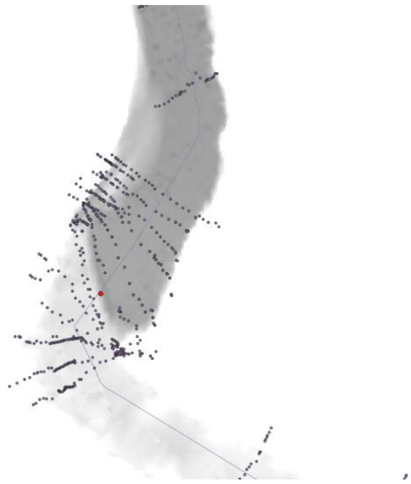


Figure 7.2 Topographic data xyz locations for Erbistock Weir (true locations)

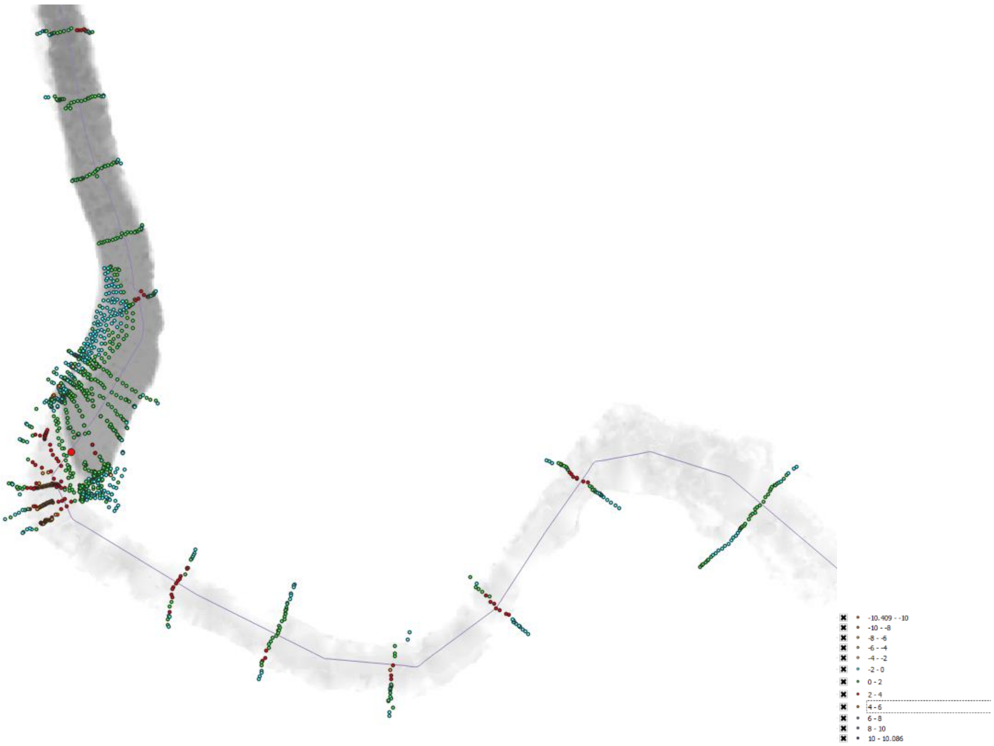


Figure 7.3. Differences between LIDAR and survey xyz data (LiDAR minus survey)

The collected topography layer was integrated into the LiDAR data of the surrounding area to improve representation of the Dee around the weir in the DEM that was to be used in the hydraulic modelling. A summary of the final DEM is provided in Figure 7.4 below. This shows levels in the area around 16 to 24m AOD.

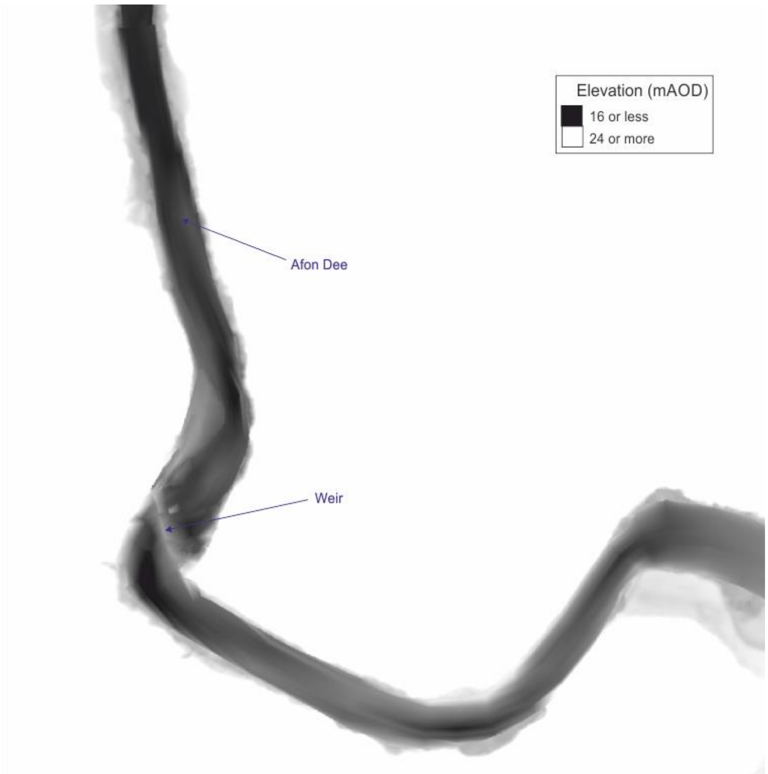


Figure 7.4 Topographic survey at and surrounding Erbistock weir

7.2 Model Build and Baseline Results

7.2.1 Modelling overview

A 2D hydraulic TUFLOW model was constructed covering the area of interest at Llangollen upstream weir. Due to the proximity to Llangollen downstream weir, the model was constructed encompassing the upstream and downstream weir. The model was constructed using available hydrological data, freely available LiDAR, topographic data and information gathered during the site visit. For the baseline and Preferred option scenario, the model was run for the events described in section 2.2.2.

The baseline surface was constructed integrating the topographic data from the survey carried out at the study area, on free available 1 m resolution LiDAR data. Then, the baseline DEM was modified simulating the partial removal of the weir.

7.2.2 Model area

The area to be modelled encompasses the area where the topographic survey was undertaken. It was extended further up and downstream of the topographic survey due to modelling purposes. The boundaries of the model are defined then by grid reference SJ 36450 42050 upstream and grid reference SJ 35576 42915 downstream. These limits are illustrated on Figure 7.5 below.

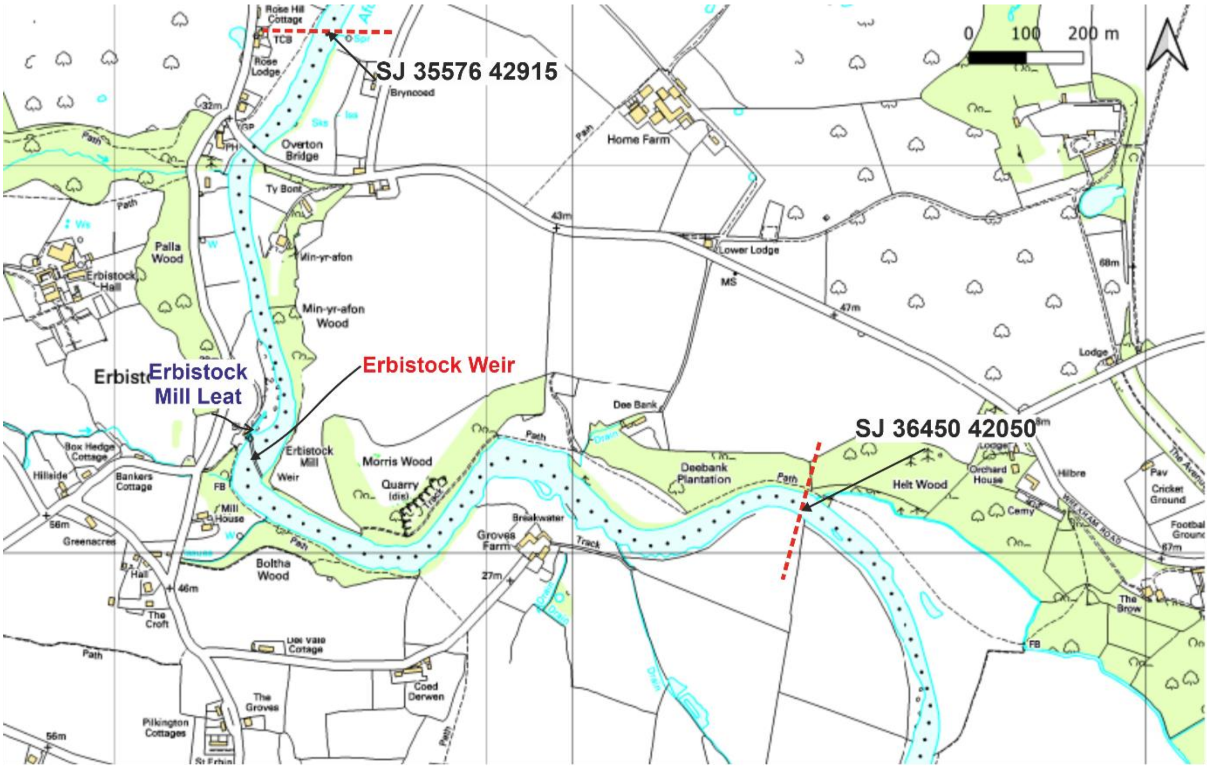


Figure 7.5 Erbistock Weir model area

7.2.3 Baseline Model

Construction of the DEM

Initial data to build the modelling surface included free available 1 m resolution LiDAR data and the topographic survey. The topographic survey included the following data:

- Survey around the weir;
- Survey on Erbistock Mill Leat; and
- Cross section along River Dee

A DEM was constructed by integrating the topographic survey data on the available LiDAR data. To integrate the topographic survey data into the DEM, the topographic survey data was interpolated using GIS software to create a TIN surface. An initial review of the TIN created with the data available indicated that, due to the looped morphology of the watercourses, iterations between cross sections were needed to ensure the surface matched the morphology (bends and cross section variations) along River Dee. Iterations were added until the TIN created matched the watercourse morphology (bends and curves were well represented). Then, the TIN was restricted into the bed channel and merged with the LiDAR by using TuFlow software.

As mentioned above, the model was extended further upstream and downstream of the topographic survey area, to do so the bed level, from the beginning of the model to the most upstream cross section of the topographic survey, was reduced 1.2m from LiDAR levels. From the most downstream cross section to the end of the model, the LiDAR bed level was reduced 1.5m. Those values where calculated by comparing the topographic cross section data and the LiDAR, upstream and downstream of the model.

Figure 7.6 shows, on the left, the LiDAR and the location of the topographic data and the iterations included to construct the surface; and on the right, the final baseline DEM.

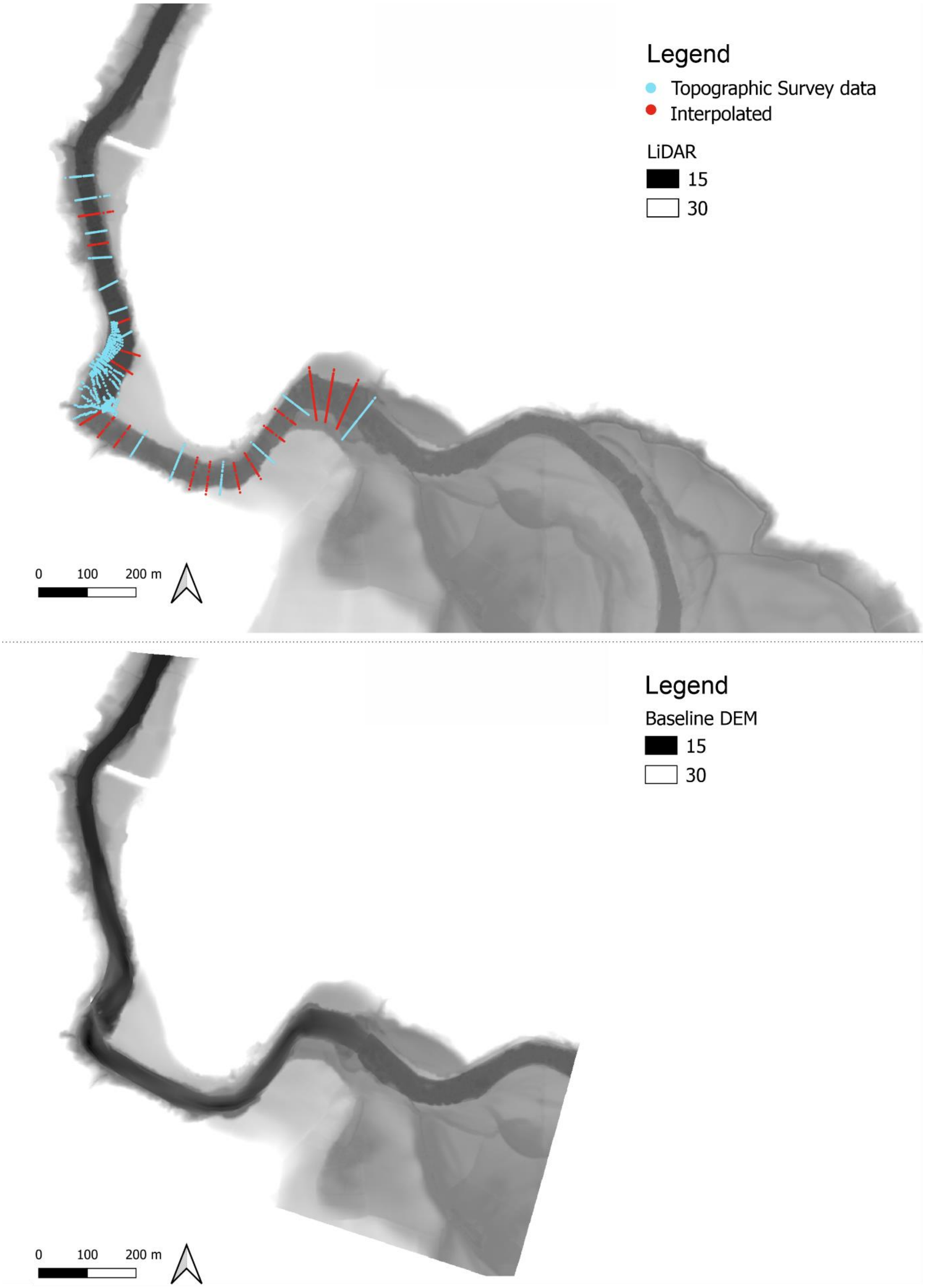


Figure 7.6 Integration of Topographic Survey data into LiDAR and final Baseline DEM.

Hydrological assessment of inflows for the model

Flow estimates for the site were as outlined in Section 2.2.2.

Figure 7.7 shows the hydrograph associated with the flood events that were modelled.

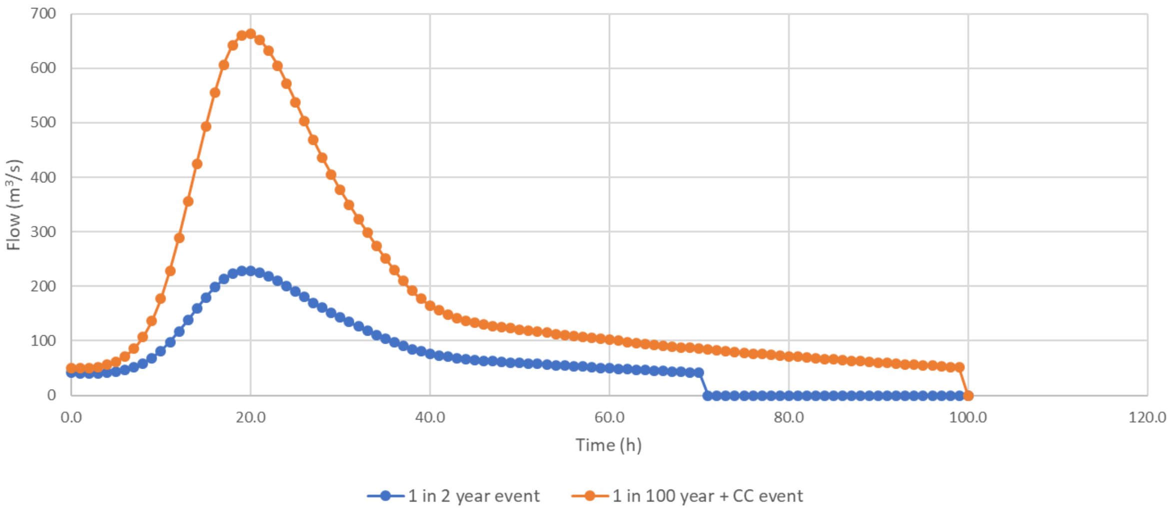


Figure 7.7 Flood events hydrographs for Erbistock Weir model

7.2.4 Design Scenario

Overview

consideration by NRW was partial removal down to bed level of 50% of the width of the weir structure, or to the failed section of the weir, following further discussion with landowner. Optioneering was undertaken to assess the suitability of this option and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Erbistock weir fish passage improvement design included:

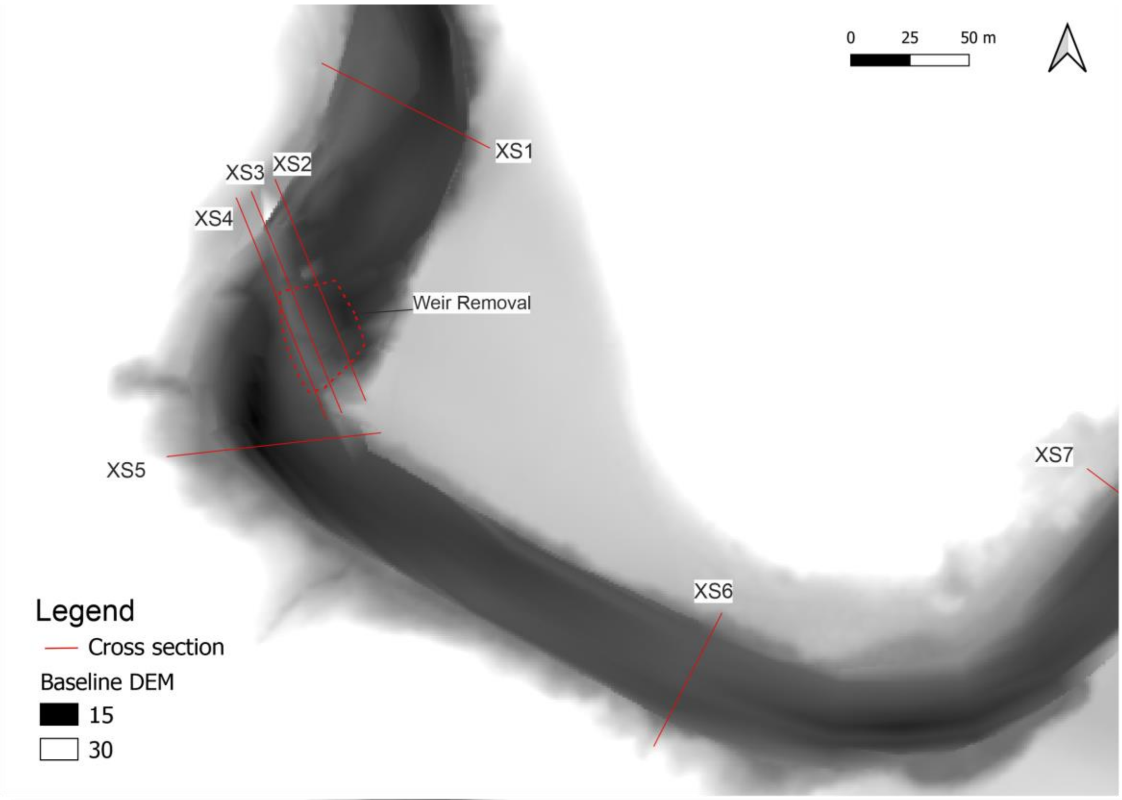
- Improved passage for upstream and downstream migrating salmonids, lamprey and eel across a range of flows
- Access is along approximately 0.7km of agricultural field, which may limit the timing of access. A footpath runs within this access route, adjacent to the river
- Trees adjacent to the site, and upstream within the drawdown reach, may require coppicing
- Road bridge downstream and bridge at Bangor-on-Dee – known blockage risks, impact of trees within drawdown reach falling in and impacting – will need to be considered, including future management
- Structural stability is uncertain
- The selected option must not increase erosion in the vicinity; the bed may need regrading to prevent excess erosion upstream
- A historical footpath exists across the top of the weir, although it appears to no longer be a route that is used
- No increase to public safety risk or flood risk
- Maintenance

The preliminary option was considered the best and recommended option for improving ecological connectivity including for all target species and life stages.

Partial removal design

As mentioned in section 2.2.2. the preferred option for Erbistock was the partial removal to bed level of ≥ 50% of the weir's width.

To simulate the partial removal, it was indicated to the model to interpolate the baseline DEM levels around the area of the weir removal. Baseline and scenario DEM where then compared and checked through XS2 to XS4. Top section of Figure 7.8 indicates the area where the modifications were done and the assessment XS locations; and the scenario DEM, at the bottom.



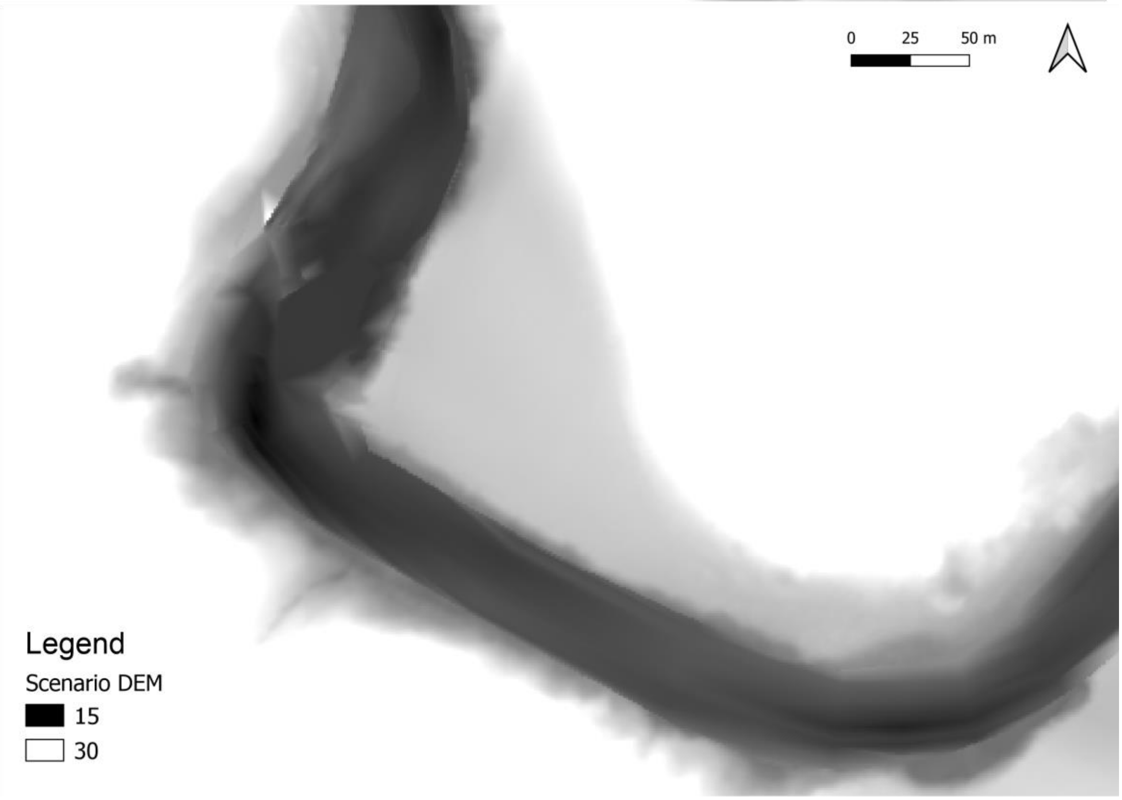


Figure 7.8 Baseline DEM modification and assessment XS location (top); and Scenario DEM (bottom).

Baseline and Scenario DEM was assessed on XS2 to XS4. Figure 7.9 show the results obtained. The three cross sections show the bed levels at the weir before and after the removal.

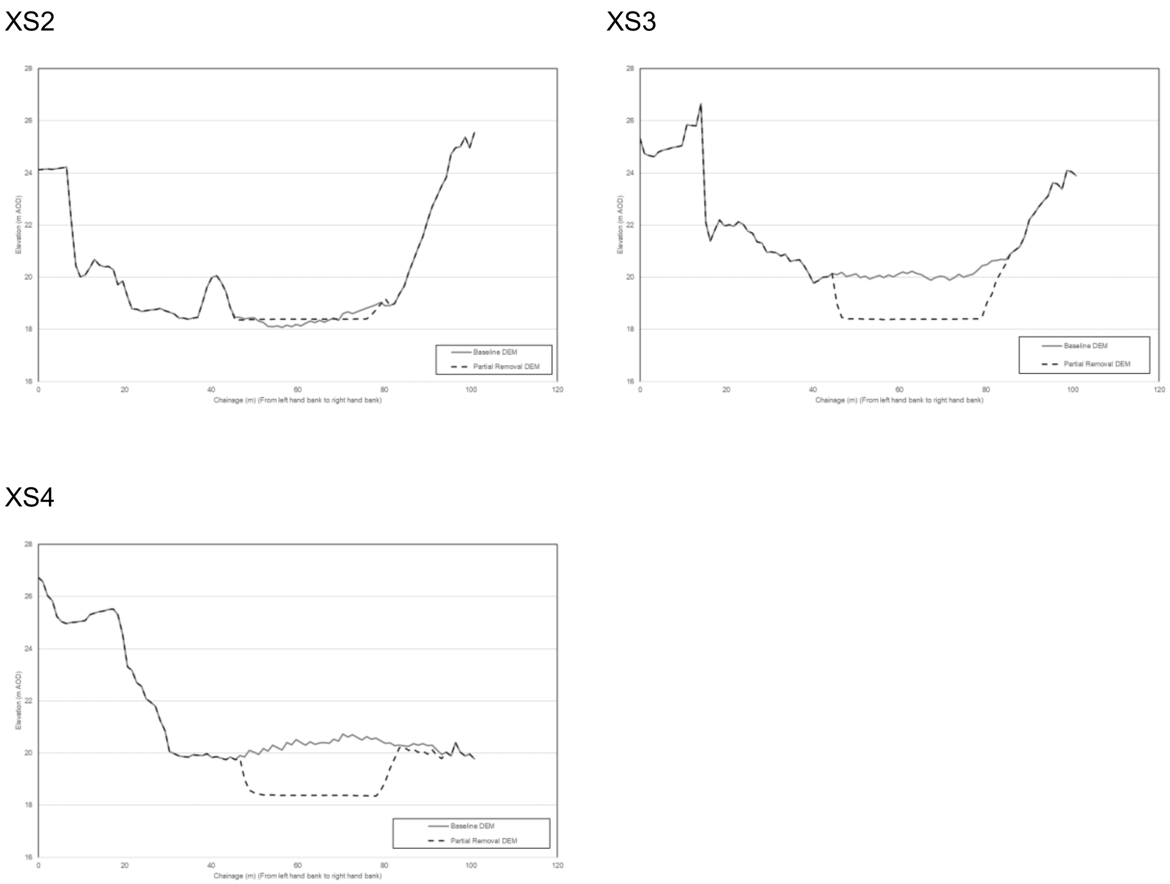


Figure 7.9 Baseline and Scenario DEM on XS2 to XS4

Long section profile along Erbistock Weir is represented on Figure 7.10. The section shows the reduction in elevation at the weir removal scenario.

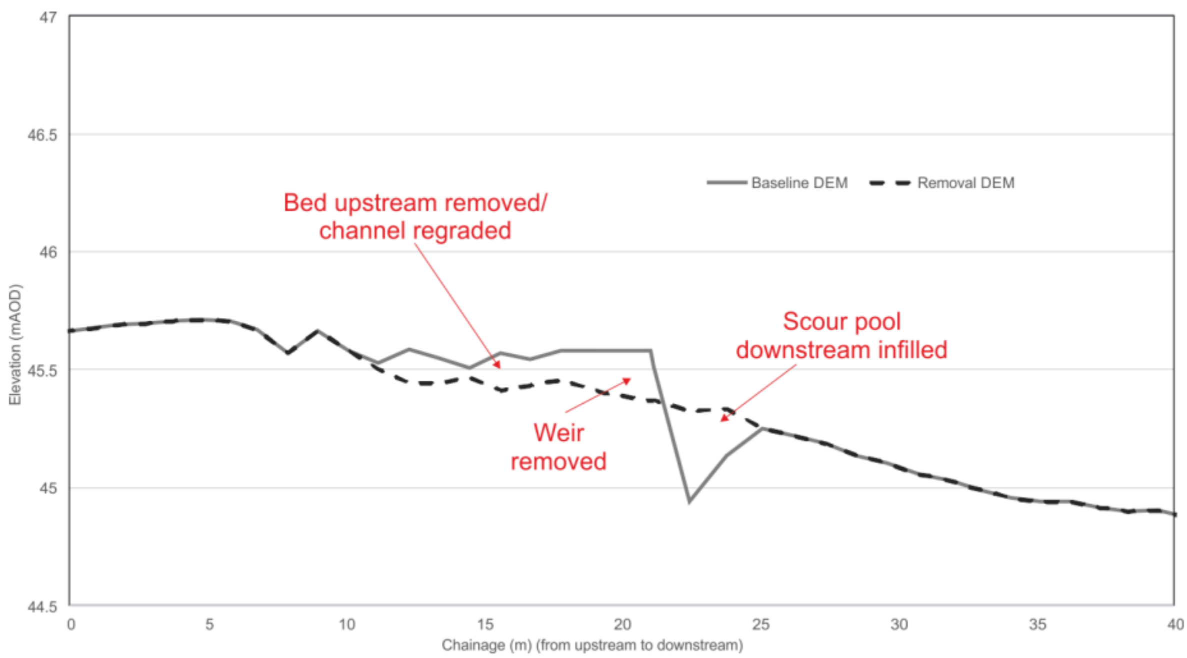


Figure 7.10 Long Profile around the weir under the baseline and partial removal scenarios

7.3 Results

Plots of velocities and depths for various flows under the baseline and partial removal scenarios are presented in Figure 7.11 and Figure 7.12, respectively. A long profile is also provided in Figure 7.13. Average water depth and velocity at Q_{95} - Q_{10} (Table 7.1) in the area of partial weir removal were sufficient for downstream migrating juvenile, and upstream migrating adult, salmonids as well as coarse fish species undertaking local migrations[11]. Partial weir removal should also enable passage for eel and lamprey.

[11] Armstrong, G.S., Aprahamian, M.W., Fewings, G.A., Gough, P.J., Reader, N.A. and Varallo, P.V., 2010. *Environment Agency Fish Pass Manual: Guidance notes on the legislation, selection and approval of fish passes in England and Wales*. Environment Agency, Rio House, Bristol http://publications. environment-agency. gov.uk.

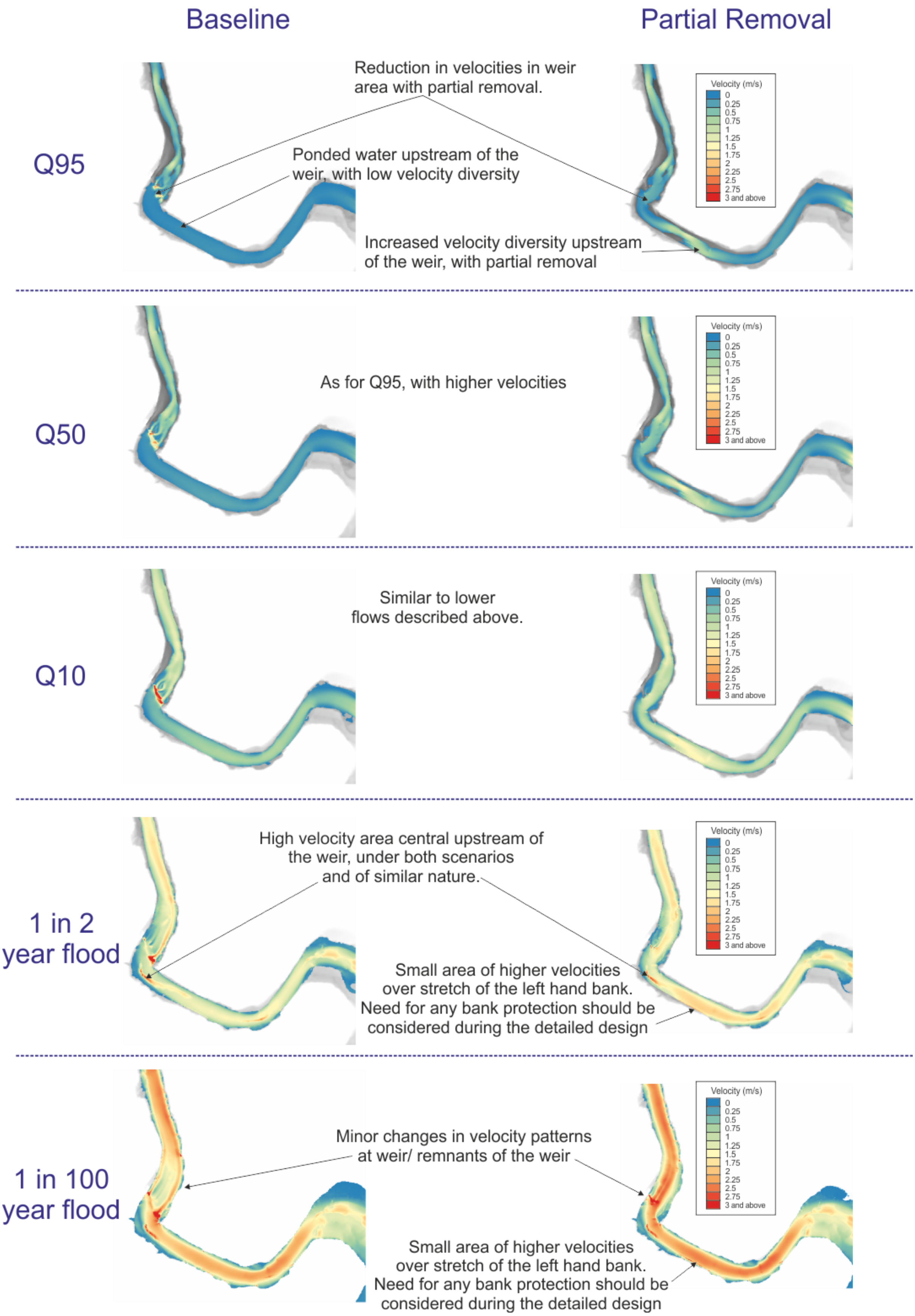


Figure 7.11 Velocity results for various flows under the baseline and partial removal scenarios

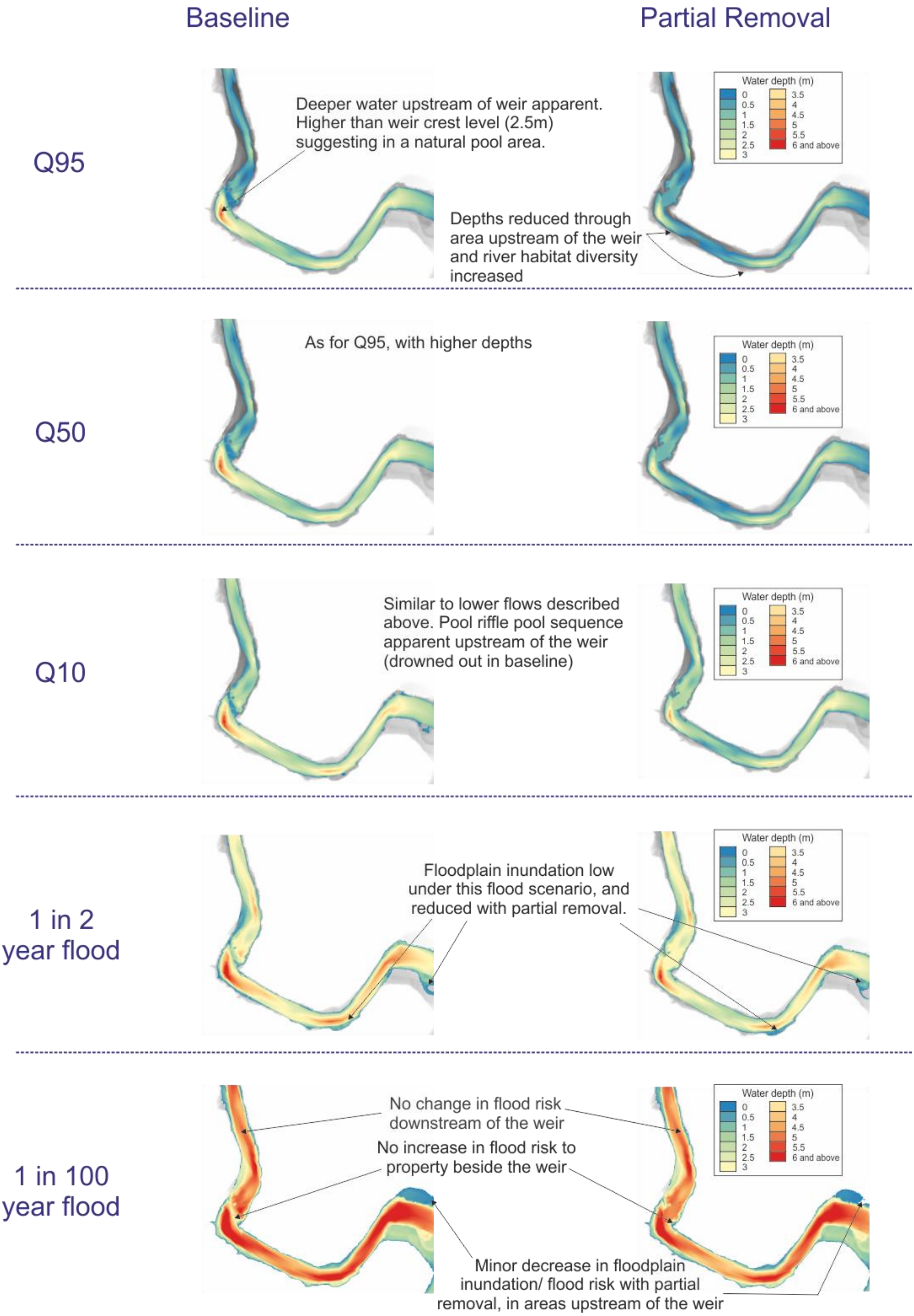


Figure 7.12 Depth results for various flows under the baseline and partial removal scenarios

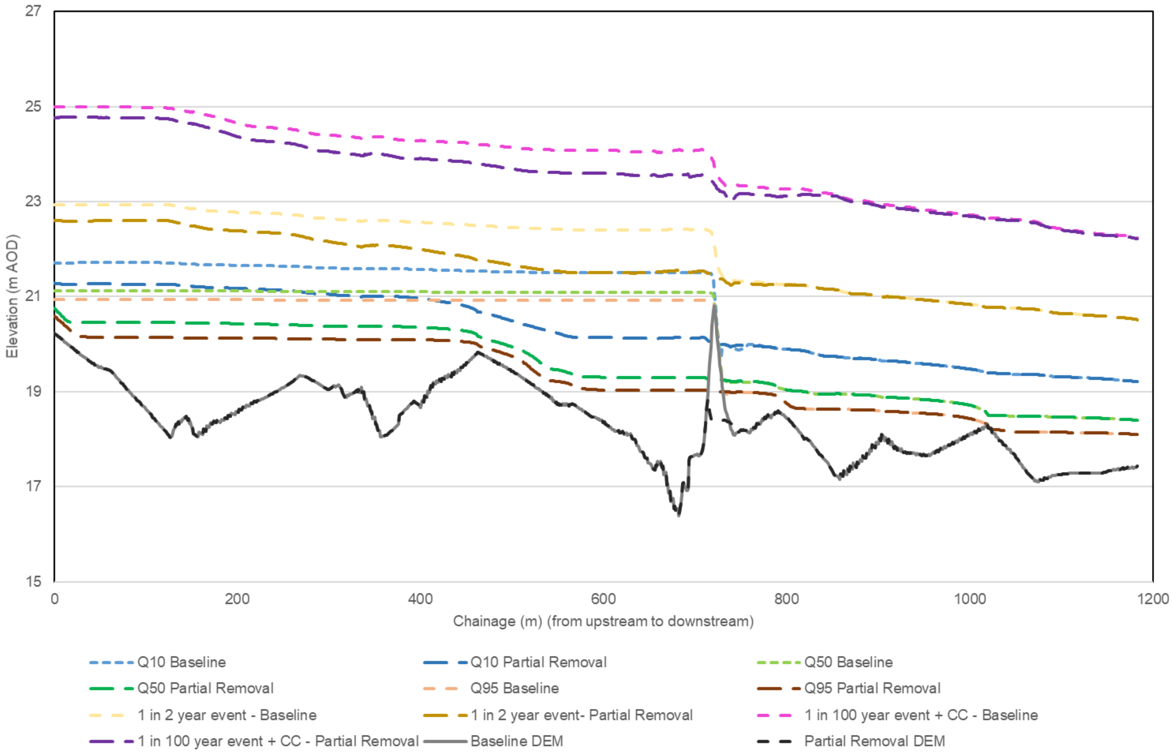


Figure 7.13 Central long profile with water levels for various flows under the baseline and partial removal scenarios

Table 7.1 Approximate maximum baseline and design (partial weir removal) scenario velocities and depths at Q_{95}, Q_{50} and Q_{10} flows

Flow	Velocity (m/s)		Depth (m)	
	Baseline	Design Scenario	Baseline	Design Scenario
Q_{95}	0.50	0.75	0.5	1.0
Q_{50}	1.75	1.00	0.5	1.5
Q_{10}	≥3.00	1.75	1.0	2.0

It seems that the natural pool upstream of the weir prevents there being a significant increase in velocities in the area of the partial removal and would likely delimit any significant re-adjustment of the river in that area, such as knickpoint erosion. The retention of the weir ends should also help the river remain in its current course. It is expected that any re-adjustment would be gradual and the existing pool riffle sequence upstream of the weir, currently drowned out by the weir, would be uncovered. Flood risk in the channel upstream of the weir is slightly reduced as a result of partial removal. There is no increase in flood risk to Erbistock Mill and the associated decked area, with data indicating that a small decrease would be expected.

Results also show a small area of the LHB upstream of the weir may erode. This would provide a benefit to the river, in terms of sediment transfer, but may not be acceptable to the landowners. It is recommended that this is examined further during detailed design, when any need for bank protection should be determined.

Depth of sediment upstream of the weir is not confirmed. Observations indicate it to not be deep, compared to other weir systems, and survey also suggests this to be the case. It is recommended that depths of sediment upstream of the weir are confirmed in support of a detailed design. This would confirm if partial removal would release large volumes of trapped substrate or not.

8. Chester Weir

8.1 Baseline

8.1.1 Site

The site around the weir is illustrated on Figure 8.1. The weir, Grade I listed, is located in the centre of Chester and is at the tidal limit of the Dee (noting it can be overtopped by high tides). It is approximately 150m in length, at an angle across the river, and about 3m high.

An existing pool and traverse fish pass is present at the site, situated on the LHB side. A fish pass (fish trap) for monitoring is present and has been in operation since 1991 (a monitoring building is located above this channel). It is believed that salmon, trout, coarse fish and lamprey use the existing pass (NRW, *pers. comm.*). Main passage requirements at the site are considered to be downstream smolt migration, particularly at low flows when they congregate and are most vulnerable to predation.

On the LHB there is a small leat channel with dual invert levels. One of them is set while the other can be lowered to encourage flow down what is effectively (and hereon referred to as) a bypass, and this is automatically done if the fish trap channel is closed off (e.g. for monitoring).

The downstream end of the pass is reported to be in need of repair including to improve the fish pass entrance. The weir is reported to be just wetted at the minimum regulation release of 4.2m^3/s, with the boat gate closed. There is an existing bypass channel on the RHB that has been used historically used to keep the intake clear of debris.

The site is bounded to the north by a towpath that runs adjacent to the historic City Walls and a mix of residential and commercial properties are located beyond.

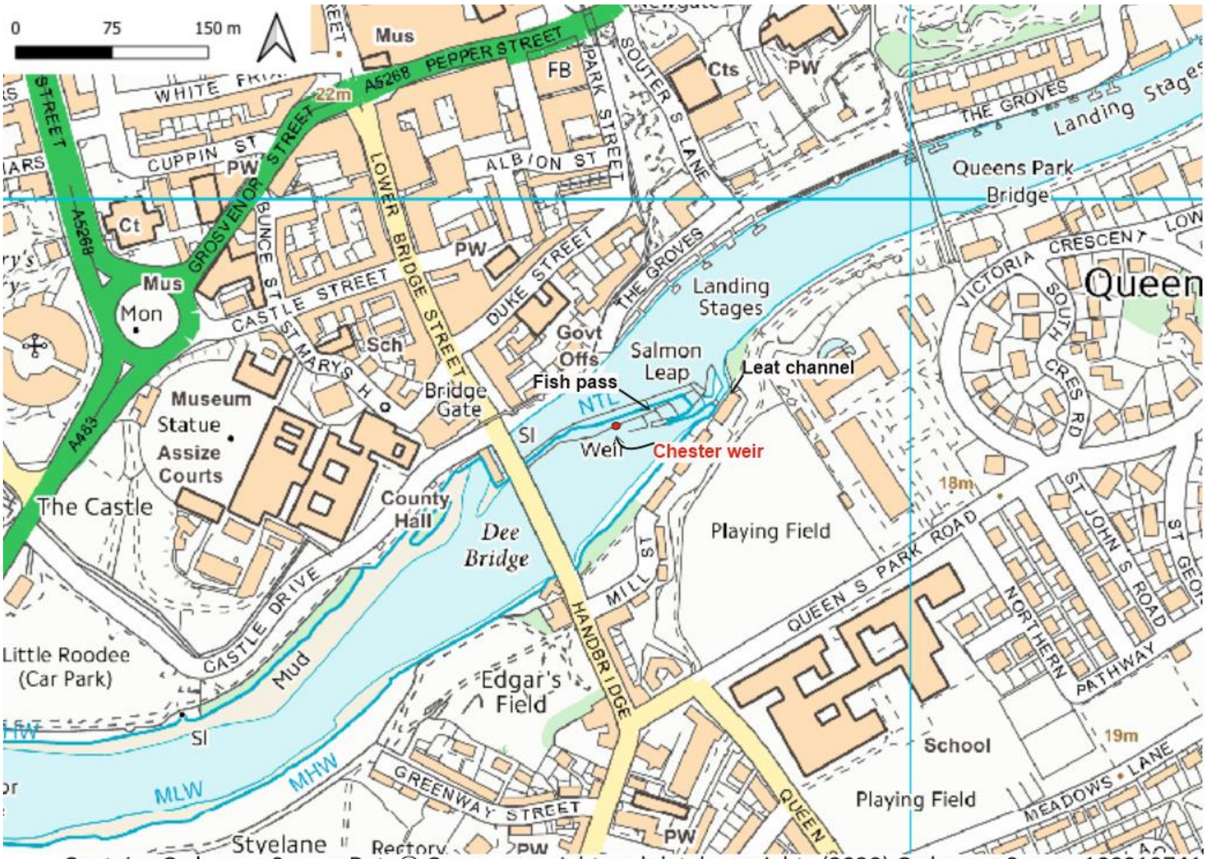


Figure 8.1 Chester weir study area

Upstream of the site and to the north, the river is bounded to the north by a towpath, an access road and a mix of commercial and residential properties, and to the south by a footpath, areas of tree cover, residential properties and a Canoe Club. Sites of historic interest lie adjacent to the site on both sides of the river while several sites of historic interest lie adjacent to the north, such as the Chester Bandstand and the Roman Gardens. To the south, the site is bounded to the south by a footpath, residential and commercial properties, public open spaces and a university campus. Sites of historic interest also lie adjacent to the south, such as the Roman Shrine to Minerva.

The River Dee continues to the west of the site, with areas of tree cover on its banks. The old City Walls, a university campus, road and car park lie adjacent on the north bank of the river, and a footpath, residential properties and a cemetery lie adjacent to the south bank.

At project onset the preferred options being considered where as follows:

- Improvements to the existing fish pass wall and notch in crest for smolt passage downstream
- Notch the weir crest to provide downstream smolt passage
- Bypass channel on the left bank through the existing channel.

Historic mapping indicates that a Causeway at the site and some form of Salmon Leap was present at the site no later than 1875. By 1899 the snuff factory, for which the leat channel was historically made for (to turn a wheel) had been constructed. By 1938 the snuff factory had been demolished.

8.1.2 Site Visit

The site was visited on the 16 July 2020. Images from this, complimented by imagery from the topographic survey (see Section 8.1.3), are presented in Plates 8-1 through to 8-9. A schematic of the various key components of the site is provided in Figure 8.2. The oversails at the top of fish pass are present to direct fish migrating upstream through the fish trap channel.

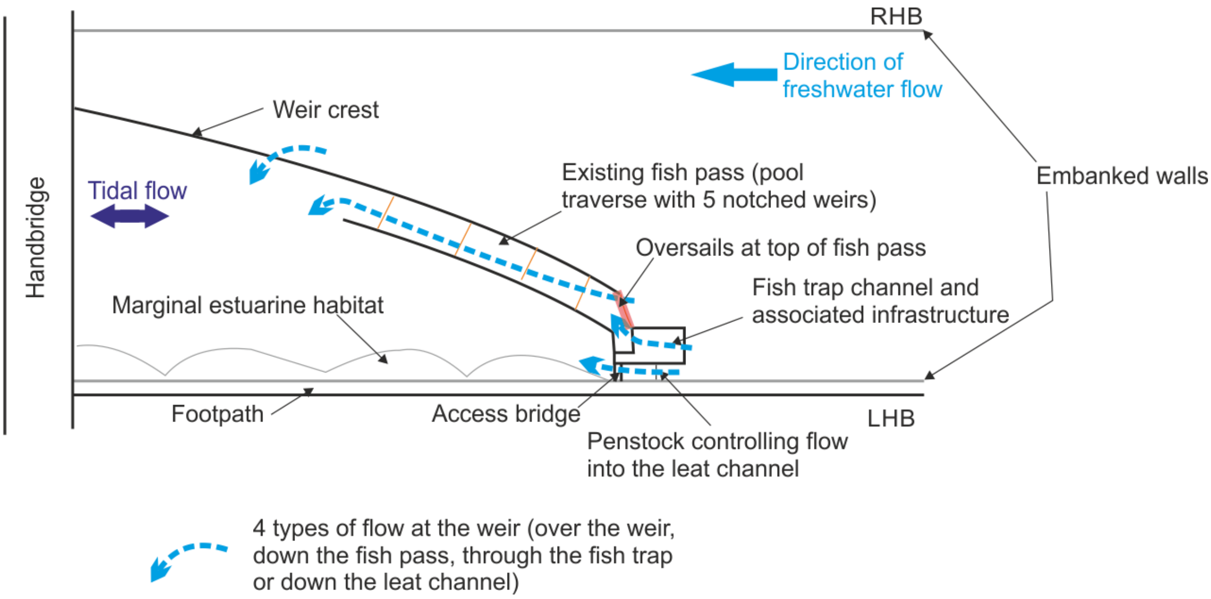


Figure 8.2 Schematic of the Key Features of Chester weir

Plate 8-1. View across channel/ weir/ fish pass (from RHB)- dilapidated fish pass walls evident at downstream end

Plate 8-2. Top of fish pass (oversails apparent)

Plate 8-3. Upper end of the fish pass channel

Plate 8-4. Downstream end of fish trap channel and building above the channel

Plate 8-5. Leat channel (looking upstream from fish trap access bridge)

Plate 8-6. Flow structures at top of the leat channel (penstock lowered)

Plate 8-7. Leat channel downstream of the fish trap access bridge (looking upstream)

Plate 8-8. Leat channel looking downstream (penstock lowered)

Plate 8-9. Leat channel looking downstream (penstock raised allowing more flow down it)

8.1.3 Topographic and Bathymetric Survey

A topographic survey of the site and surrounding area was undertaken. This included a survey at and around the weir, cross sections in the River Dee upstream and downstream extending approximately 500m and 350m from the weir, respectively. The survey was undertaken to refine representation of the system within the hydraulic modelling (see Section 8.2.3).

All surveyed points are indicated in Figure 8.3 below while those around the weir itself are indicated in Figure 8.4, which shows a reasonable coverage with some gaps where surveying could not be undertaken safely.

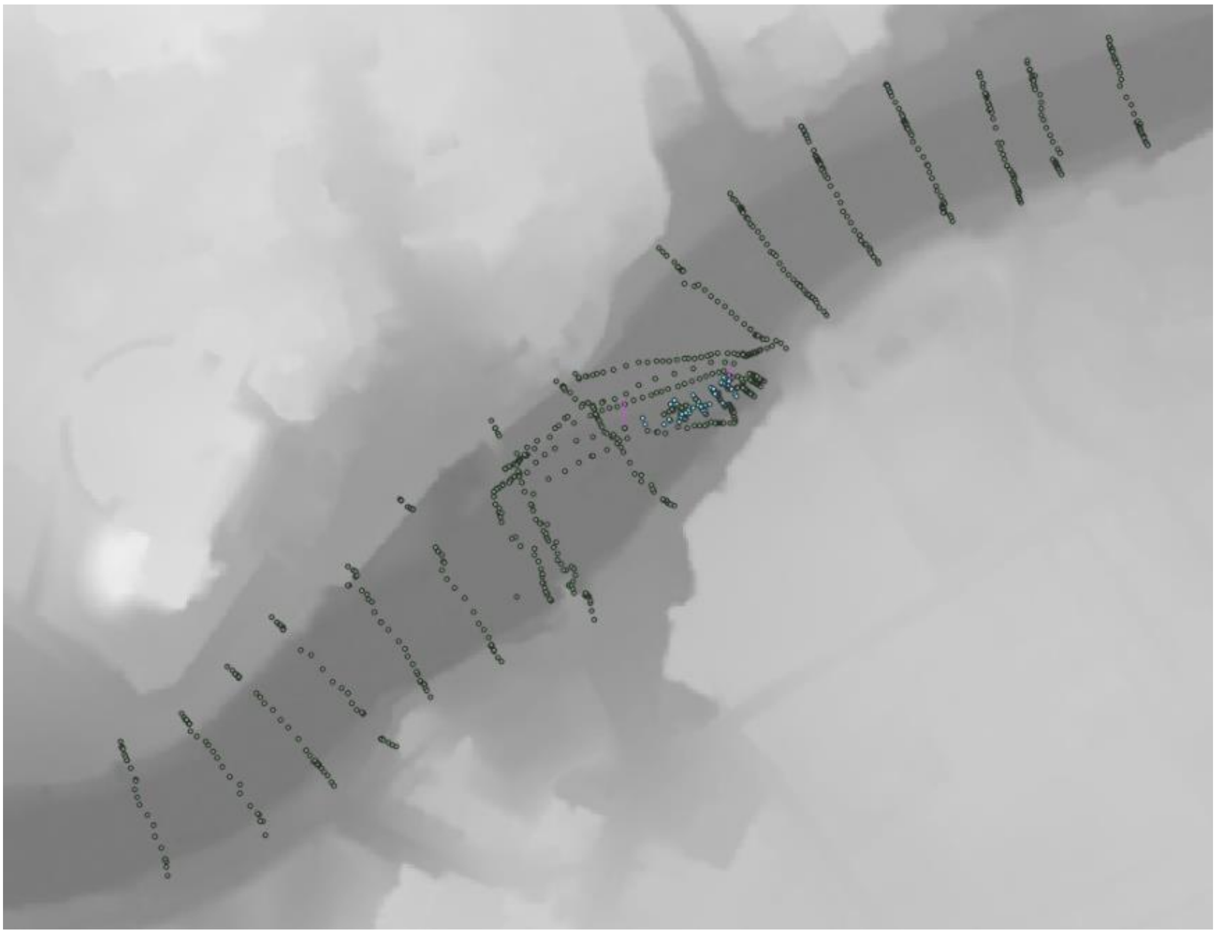


Figure 8.3 Chester weir xyz topographical data received

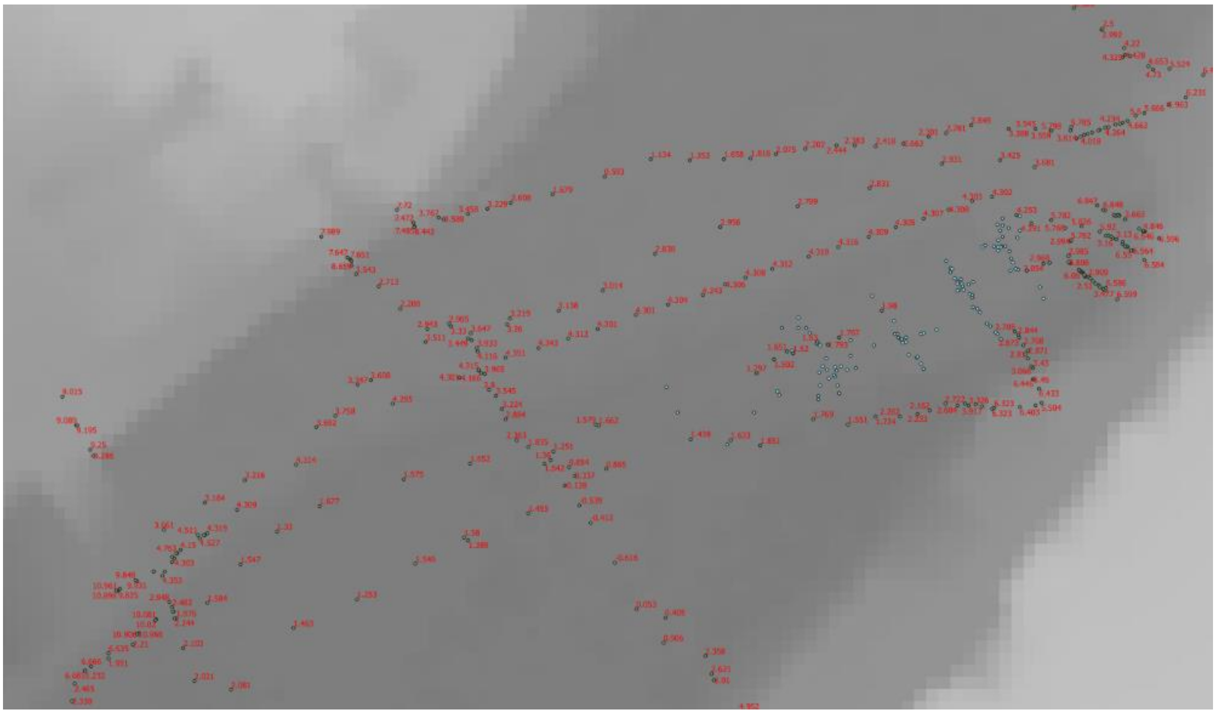


Figure 8.4 Chester weir xyz topographical data received (zoomed into area around the weir)

The difference between all surveyed river points and LiDAR data is shown in Figure 8.5. This suggests that differences were generally within 1m, though upstream of the weir a deeper central channel was picked up by the survey but not the LiDAR (which is as expected, with water refracted LiDAR rays), again showing the value of undertaking the survey.

Figure 8.5 Differences between LiDAR and survey xyz data (LiDAR minus survey)

8.2 Model Build and Baseline Results

8.2.1 Modelling overview

A 2D hydraulic TUFLOW model was constructed covering the area of interest at Chester Weir. The model was constructed using available hydrological data, freely available LiDAR, topographic data and information gathered during the site visit. For the baseline and preferred option scenario, the model was run for the events described in section 2.2.2.

The baseline surface was constructed by integrating the topographic data from the survey carried out at the study area, on freely available 1 m resolution LiDAR data. Then, the baseline surface was adjusted to represent the preferred option.

To evaluate if the hydraulic design of the preferred option allows passage for the fish species and to examine if there were any potentially issues associated with the proposed options, water depth, velocity and shear stress results were assessed. These results are generated for the model as raster files, which were interrogated to review the performance of the preferred option.

8.2.2 Model area

The area to be modelled encompasses the area where the topographic survey was undertaken. Figure 8.6 below show the limits of the model

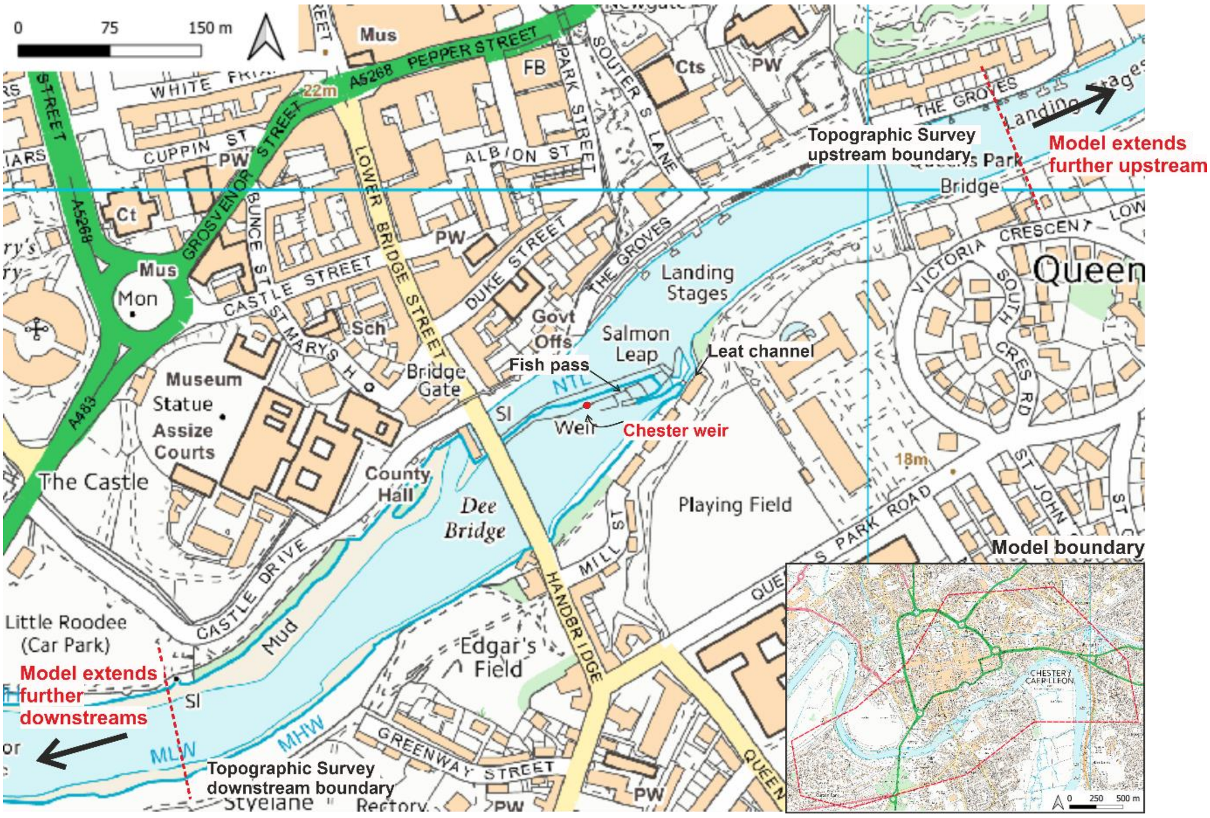


Figure 8.6 Chester Weir model area

8.2.3 Baseline Model

Construction of the DEM

Initial data to build the modelling surface included freely available 1 m resolution LiDAR data and the topographic survey. The topographic survey included the following data:

- Survey around the weir;
- Survey at the leat;
- Survey on the fish pass; and
- Cross sections along River Dee

A DEM was constructed by integrating the topographic survey data on the available LiDAR data. To integrate the topographic survey data into the DEM, the topographic survey data was interpolated using GIS software to create a TIN surface. No iterations between cross sections were needed as the river morphology and planform was well represented by the TIN surface.

The TIN was restricted into the wetted areas/ river and merged with the above water LiDAR using TuFlow software. Figure 8.7. shows, on the left, the LiDAR and the location of the topographic survey data included to construct the surface; and on the right, the final baseline DEM. Parts of the model surface were raised to represent the bridge piers of the Old Dee Bridge.

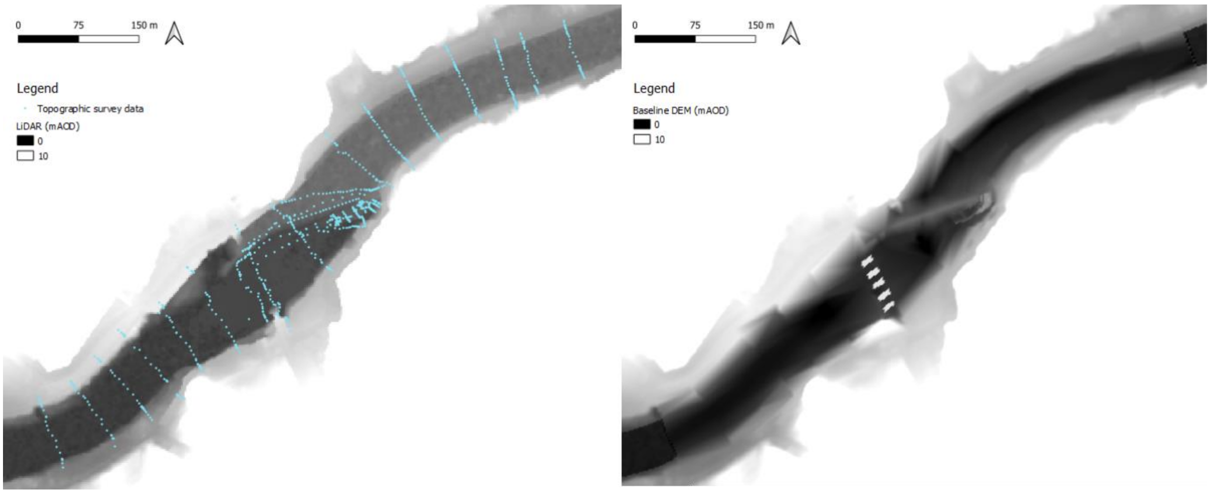


Figure 8.7 Integration of Topographic Survey data into LiDAR and final Baseline DEM.

Hydrological assessment of inflows for the model

Flow estimates for the site were as outlined in Section 2.2.2.

Figure 8.8 shows the hydrograph associated with the flood events that were modelled.

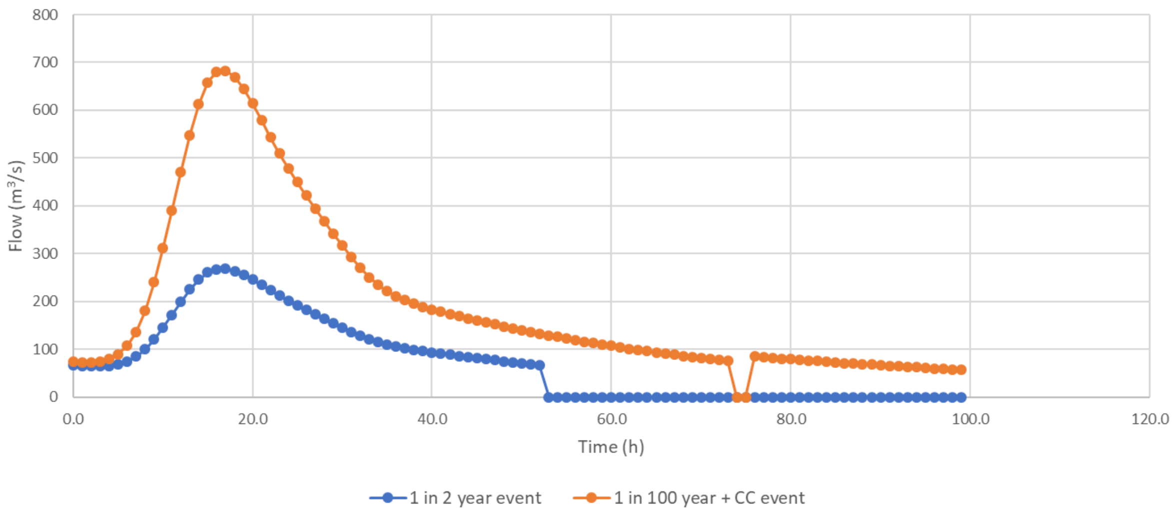


Figure 8.8 Flood events hydrographs for Chester Weir model.

8.2.4 Design Scenario

Overview

A design to improve fish passage at Chester weir was developed. The preliminary preferred options under consideration by NRW were: 1) Improvements to the existing fish pass wall and a notch in the crest for smolt passage downstream; 2) Notch the weir crest to provide downstream smolt passage, and; 3) Bypass channel on the LHB, through the existing channel. Optioneering was undertaken to assess the suitability of these options and others for achieving passage for the target species and life stages whilst balancing the requirements of key considerations.

Important considerations for Chester weir fish passage improvement design included:

- Improved passage primarily for downstream migrating salmonids, lamprey and eel across a range of flows
- Weir must not run dry at the minimum regulation release of 4.2 m^3/s (assuming boat gate is closed and not leaking)
- Area of historical interest

- Limited visual impact including avoidance of weir drying at low flows
- No increase to public safety risk or flood risk
- Maintenance
- Continuation of the fish trap monitoring programme

The preliminary preferred options would improve connectivity for downstream salmonid smolts, and cause no increase to public safety risk or maintenance. Notching the weir was considered preferable to other fish passage options e.g. technical and other non-technical options including modifying the existing channel on the LHB to create a bypass, particularly for the provision of downstream passage for smolts. The location of the existing LHB channel and associated bathymetry does not appear to be conducive to high salmonid adult or smolt attraction efficiencies and therefore passage even with the existing leat weirs removed. Maintenance and risk of debris/ sediment build up in the leat channel is also of concern (NRW, *pers. comm.*). Further, the existing fish pass is considered effective for attracting and passing upstream migrating adult salmonids with only minor modifications, including increasing the notch size at the top of the pass and potentially repairing the existing wall structure (NRW, *pers. comm.*).

Initial bathymetric assessment of the channel upstream of Chester weir identified that while flow is predominately diverted down the existing fish pass, the natural thalweg may lie more towards the RHB. This concept, combined with locating a notch towards to downstream end of the weir which is positioned diagonally across the river, lead to the recommendation for two notches in Chester weir: one enlarging the existing notch at the top of the fish pass (Notch 1), and another in line with the natural thalweg towards the downstream end of the weir (Notch 2). Subsequent optioneering discussions with NRW indicate that salmonid smolts are observed to congregate upstream of the existing fish pass in spring, however it should be noted that a second notch towards the downstream end may form an alternate route of passage once constructed. A separate pump-fed eel and lamprey pass in the existing side channel on the LHB is also proposed.

Scenario

A model comprising two notches was constructed. The historical value of Chester weir is noted, and the model is considered to present conservative results that can be applied to a single, larger notch at the top of the existing fish pass should a second notch towards the downstream end of the weir not be acceptable. Representation of the notches in the model DEM is shown in Figure 8.9 below. The as built drawing of the existing notch at the top of the existing fish pass is shown in Plate 8-10. The topographic survey did not capture a structure as pronounced as the as built drawing indicates (potentially due to the presence of the oversails). The baseline modelling relied on the survey data, hence providing a conservative indication of flow through the notch. Irrespective of this, the proposals would be to increase the existing notch (Notch 1) size to 2.0m wide and 0.3m deep. Notch 2 would comprise the same dimensions as Notch 1 and include a 2.0m wide and 0.3m deep channel along the weir face towards the riverbed, avoiding glacis drying, sudden drops and rough surfaces[12]. The channel was oriented in line with the predominant localised direction of flow. The proposed eel pass is too small to include in the modelling and changes in depths and velocities associated with it would be minimal.

The results shown for the baseline and design scenario are of simulations with the fish trap open and the invert to the leat channel raised. Sensitivity tests were carried out of the baseline scenario with the fish trap closed and invert to the leat lowered.

[12] Environment Agency, 2005. *Screening for intakes and outfalls: a best practice guide*. Science Report CS030231. Environment Agency, Bristol, 153 pp.

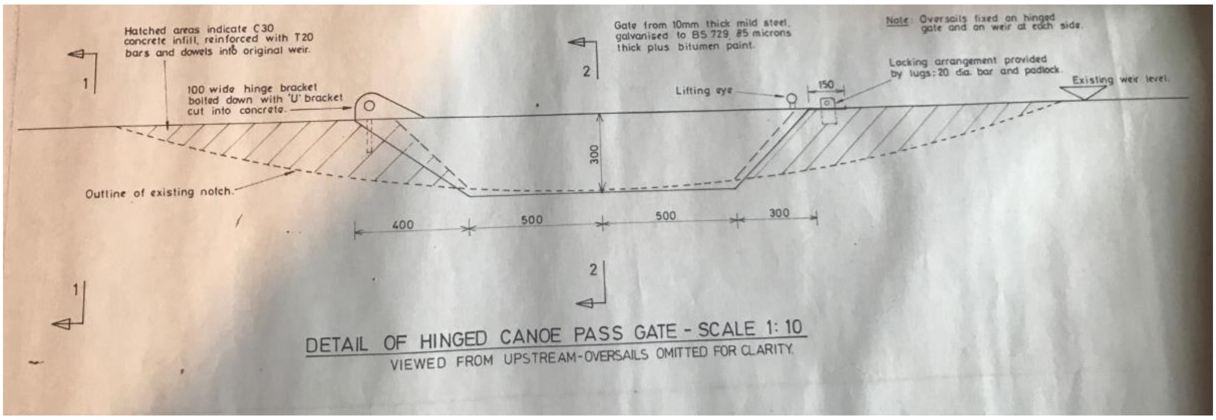


Plate 8-10 As built drawing of the notch at the top of the fish pass channel

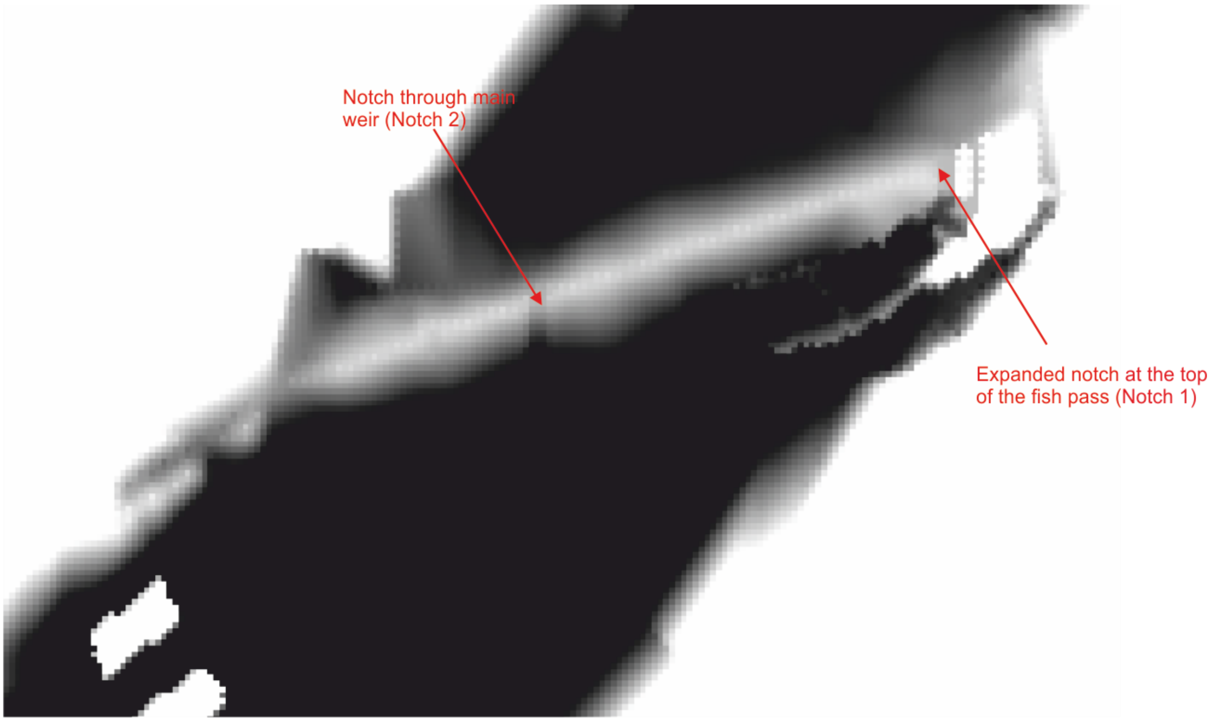


Figure 8.9 DEM for the design scenario at Chester weir scenario

8.3 Results

Plots of velocities and depths for various flows under the baseline (fish trap open) and the notched weir scenario are presented in Figure 8.10 and Figure 8.11, respectively. Average water depth and approximate maximum velocities at Q_{95} - Q_{10} in the notches were sufficient for downstream migrants (Table 8.1), providing that detailed design ensures gradual and smooth flow acceleration into a bell mouthed notch entrance[13]. A pool immediately downstream of the notch and excavated channel may

[13] Environment Agency, 2005. *Screening for intakes and outfalls: a best practice guide*. Science Report CS030231. Environment Agency, Bristol, 153 pp.

also be required. A minimum depth of 0.9m for head differences of <3.6m is suggested[14], however refinement based on local constraints may be possible. Upstream migrating adult salmonids are anticipated to be able to ascend through Notch 1 at Q_{95} - Q_{10} flows[15]. Notch 1 could be closed during periods when the fish trap requires operation. Similarly, options for lifting the oversails at Notch 1 when the fish trap is not in operation should be considered during detailed design.

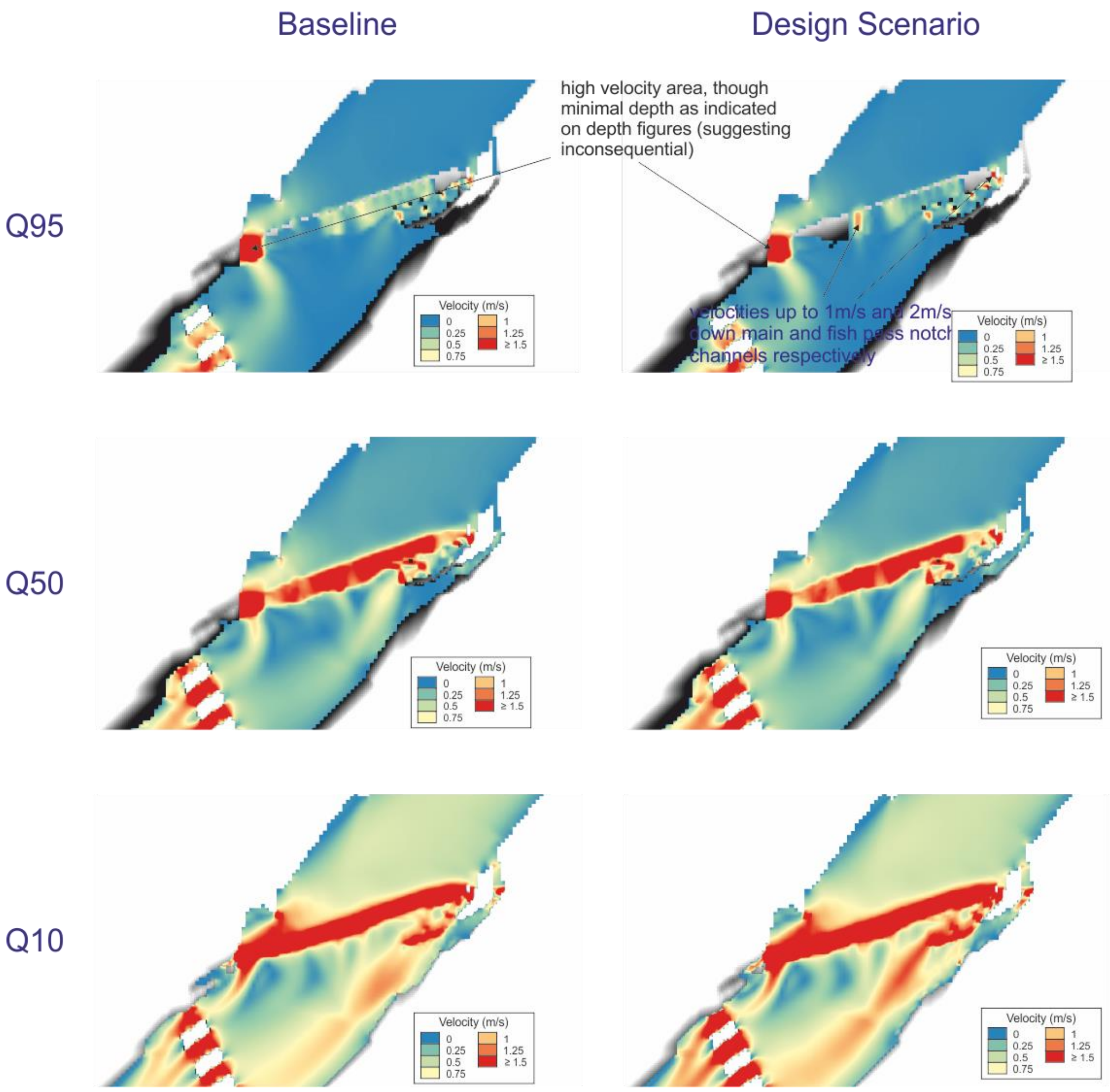


Figure 8.10 Velocity results for various flows under the baseline and design scenarios at Chester weir

[14] Odeh, M and Orvis, C., 1998. Downstream fish passage considerations and developments at hydroelectric projects in North-east USA. In: *Fish Migration and Fish Bypasses* (Ed. Jungwirth, M., Schmutz, S. and Weiss, S.). Fishing News Books, Oxford, Blackwell: 267-280.

[15] Armstrong, G.S., Aprahamian, M.W., Fewings, G.A., Gough, P.J., Reader, N.A. and Varallo, P.V., 2010. *Environment Agency Fish Pass Manual: Guidance notes on the legislation, selection and approval of fish passes in England and Wales.* Environment Agency, Rio House, Bristol http://publications. environment-agency. gov.uk.

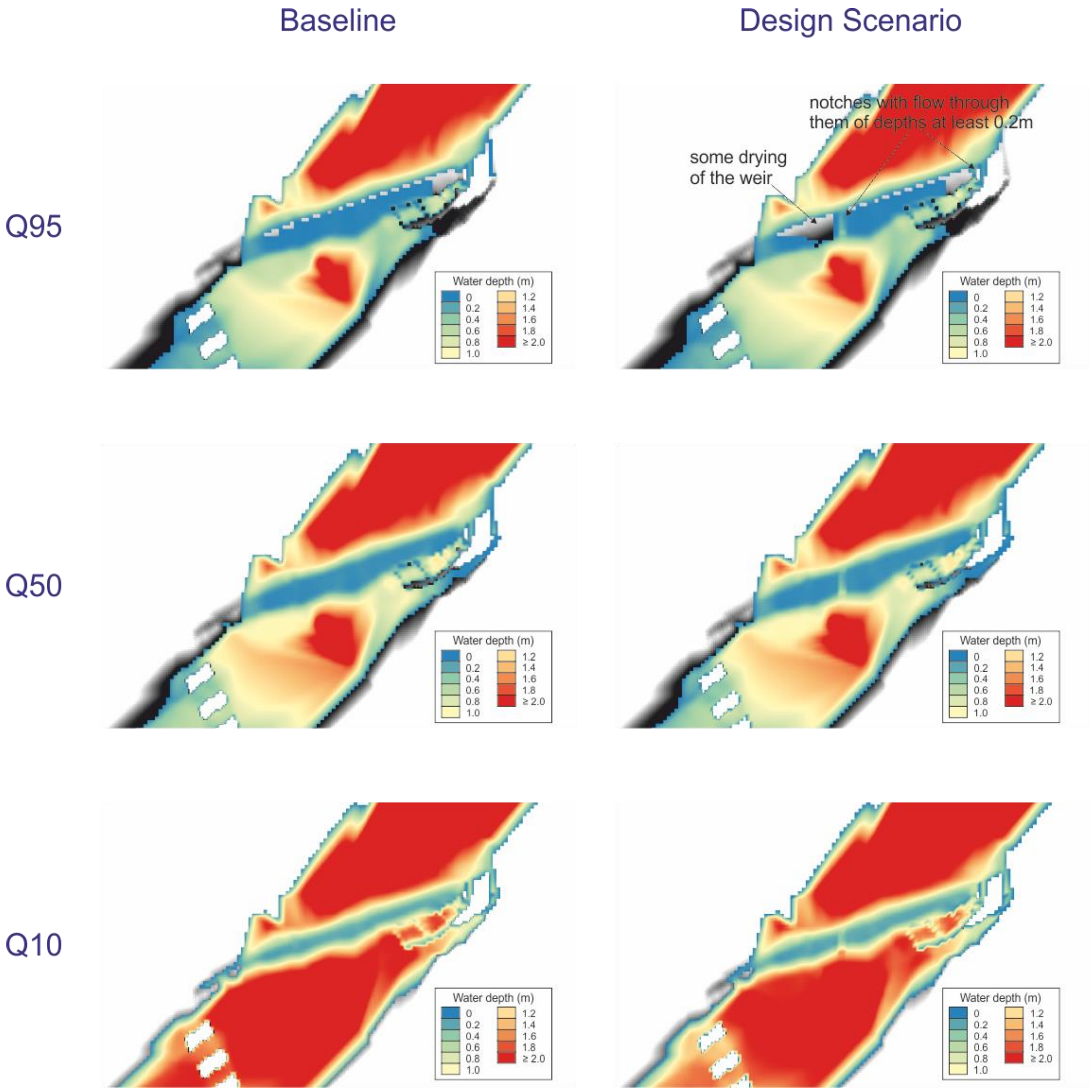


Figure 8.11. Depth results for various flows under the baseline and design scenarios at Chester weir

Table 8.1 Approximate maximum baseline and design (two notches: Notch 1 at existing fish pass exist and Notch 2 towards downstream end of Chester weir) scenario velocities and depths at Q_{95}, Q_{50} and Q_{10} flows. Both notches would be 0.3m deep by 0.2 m wide.

Flow	Velocity (m/s)				Depth (m)			
	Notch 1		Notch 2		Notch 1		Notch 2	
	Baseline	Design Scenario	Baseline	Design Scenario	Baseline	Design Scenario	Baseline	Design Scenario
Q_{95}	≥1.5	≥1.5	1.0	1.0	0.0	0.2	0.2	0.4
Q_{50}	≥1.5	≥1.5	≥1.5	≥1.5	0.2	0.4	0.2	0.6
Q_{10}	≥1.5	≥1.5	≥1.5	≥1.5	0.4	0.6	0.4	0.8

Maps of the 100 year plus climate change flood extents for the baseline and design scenario are shown in Figure 8.12. No differences were apparent indicating that the design would not change fluvial flood risk in and around the study area. No changes due to tidal flood risk were anticipated either given the relatively small changes predicted (which could even be beneficial due to increasing channel capacity).

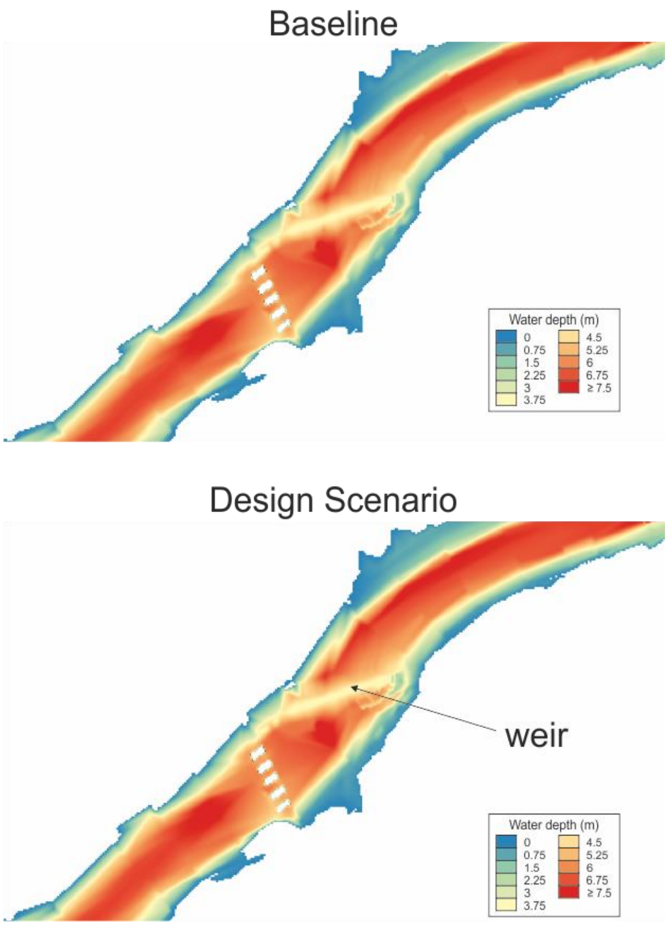


Figure 8.12 Map of 100 year plus climate change extents and depths around Chester Weir for the baseline and design scenarios

Annex 1 River Dee LIFE - Dee at Horseshoe Falls, Llangollen and Erbistock: Flood Estimation Calculation Record

River Dee LIFE - Dee at Horseshoe Falls, Llangollen and Erbistock

Flood Estimation Calculation Record

60627686

December 2020

1 Flood estimation calculation record

Introduction

This document provides a record of the calculations and decisions made during flood estimation for the Dee at three FEPs; Horseshoe Falls, Llangollen and Erbistock. The information given here should enable the work to be reproduced in the future.

Contents

Approval

	Signature	Name and qualifications
Calculations prepared by:		Andrew Heath-Brown BSc (Hons), MSc, MCIWEM
Calculations checked by:		Helen Harfoot BSc (Hons), MSc, MCIWEM, CSci, CEnv
Calculations approved by:		

Revision History

Revision	Revision Date	Details	Authorised	Name	Position

Abbreviations

AEP	Annual exceedance probability
AM	Annual maximum
AREA	Catchment area (km^2)
BFI	Base flow index
BFIHOST	Base flow index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
DPLBAR	Mean drainage path length (km)
DTM	Digital Terrain Model
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FPEXT	Floodplain extent
FSR	Flood Studies Report
HOST	Hydrology of soil types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks over threshold
QMED	Median annual flood (with return period ~2 years)
ReFH1	Revitalised Flood Hydrograph 1 method (2005)
ReFH2	Revitalised Flood Hydrograph 2 method (2013)
SAAR	Standard average annual rainfall (mm)
SPR	Standard percentage run-off
SPRHOST	Standard percentage run-off derived using the HOST soil classification
Tp (0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent
WINFAP	Windows Frequency Analysis Package – used for FEH statistical method

2 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections. The aim of the table is to enable quick and easy identification of the type of assessment undertaken. This should assist in identifying an appropriate reviewer and the ability to compare different studies more easily.

Catchment location	Horseshoe Falls (River Dee), nr Llantysilio Hall, Denbighshire
Purpose of study and scope	Routine calculation of peak flood flow estimates, for use as part of a fish passage solution assessment.
Key catchment features	No unusual features, but catchment is large
Flooding mechanisms	n/a
Gauged / ungauged	Gauged (not at site)
Final choice of method	FEH Statistical
Key limitations / uncertainties in results	Minor uncertainties (associated with data transfer from gauged site on same river).

2.1 Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval. Results tables in this document contain both return period and AEP titles; both rows can be retained, or the relevant row can be retained and the other removed, depending on the requirement of the study.

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

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

3 Method statement

3.1 Overview of requirements for flood estimates and hydraulic modelling

Item	Comments
Give an overview which includes: • purpose of study including a short discussion if there is existing hydrology reports and estimates, when they were done and why we are updating the hydrology (e.g. new data or superseded methods) • approximate number and type of flood estimates required • peak flows and/or hydrographs? • range of design event AEPs (%) • climate change allowances (ref. relevant guidance)	The study is an investigation into the possible implementation of fish passage at a number of weirs within the Dee catchment. The scope requires assessment of two flood flow events (likely 2-year and 100-year), hence the need for peak flow estimates. There are no known existing peak flow estimates. A range of estimates will be produced between 2 and 200-year return periods. However, the 1,000-year estimate will not be included. Peak flows and hydrographs are required, at a single location only for each weir site. In this calculation record, the sites on the River Dee at Horseshoe Falls, Llangollen and Erbistock are considered concurrently due to their relative proximity and similar catchment characteristics.

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report. Include general catchment map and specific map of hydraulic model extents and inflow locations.	Peak flows and hydrographs have been estimated at the Horseshoe Falls weir (SJ 19500 43350), Llangollen weir (SJ 21350 42100) and Erbistock weir (SJ 35450 42150). The main report provides a description of the catchment and the modelled reaches, including details of where the flow estimates will be entered in the associated hydraulic model.
Previous Hydrology studies	None known

3.3 Source of flood peak data

Item	Comments
Was the NRFA Peak Flows dataset used? If so, which version? If not, why not? Record any changes made.	NRFA peak flows dataset, Version 9, released October 2020. This contains data up to the end of water year 2018-19, and provisional data for water year 2019/2020 at stations which set new records.

3.4 Gauging stations (flow or level)

The following gauging stations include ones considered and used as part of a QMED data transfer, but are not necessarily one the study reach/ within the study catchment.

Watercourse	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level)	Start of record and end if station closed
Dee	Manley Hall	067015	67015	1013	Rated	1970

3.5 Data available at each flow gauging station

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling?	Data quality check needed?	Other comments on station and flow data quality
Manley Hall	1970	No	Yes	Yes	No	

3.6 Other data available and how it has been obtained

Type of data	Data relevant to this study	Data available	Source of data	Details
Check flow gaugings (if planned rating review)	~~Yes~~ / No	Yes / No	n/a	n/a
Rating equations	~~Yes~~ / No	Yes / No	n/a	n/a
Historic flood data	~~Yes~~ / No	Yes / No	n/a	n/a
Flow or level data for events	~~Yes~~ / No	Yes / No	n/a	n/a
Results from previous studies	~~Yes~~ / No	Yes / No	n/a	n/a
Other information e.g. groundwater, tides etc	~~Yes~~ / No	Yes / No	n/a	n/a

3.7 Initial choice of approach

Item	Comment
Is FEH appropriate? If not, describe why and give details of the other methods to be used.	Yes. In particular, this enables the results to account for the upstream reservoir attenuation (denoted by the FARL value). The Dee regulation scheme upstream can also be accounted for via the use of a donor on the River Dee to adjust QMED t the subject site.
Initial choice of method(s) and reasons.	FEH Statistical – catchment has no unusual features, and is within limits if applicability for FEH Statistical method. FEH Statistical is preferred over ReFH2 in most cases, unless demonstrated otherwise. ReFH2 unlikely to be appropriate for estimating peaks flows on catchments of this size (concept of catchment wide storm less applicable). The sites are ungauged so ESS is not relevant.
How will hydrograph shapes be derived if needed? E.g. ReFH1, ReFH2 or average hydrograph shape from gauge data	The ReFH2 method will be used to define hydrograph shape, based on the recommended duration. This is expected to be appropriate for the nature of this study. For detailed flood risk investigation, gauged hydrograph analysis with calibration would be required.
Will the catchment be split into sub-catchments? If so, how?	The catchment will not be split into subcatchments. It is anticipated that the modelled reach will be short, and there are no significant tributaries in close proximity to the subject site with obvious differences to the overall contributing catchment.
Software to be used (with version numbers) (delete as appropriate)	FEH Web Service[1] / ~~WINFAP-FEH v3.0.003~~[2] / WINFAP 4[3] / ~~ReFH spreadsheet~~ / ReFH2.2 / ~~Flood Modeller Pro~~

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

4 Locations where flood estimates are required

4.1 Summary of subject sites

The table below lists the locations of subject sites.

Site Code	Type of Estimate (L – lumped catchment; S- Sub-catchment)	Watercourse	Site	Grid Reference	Area on FEH Web Service (km^2)	Revised area if altered
Dee_HF	L	Dee	Horseshoe Falls Weir	SJ 19500 43350	753	n/a
Dee_Llan	L	Dee	Llangollen Weir	SJ 21350 42100	785	n/a
Dee_Erb	L	Dee	Erbistock Weir	SJ 35450 42150	1033	n/a

Reasons for choosing above locations	Location of proposed fish pass

4.2 Important catchment descriptors at each subject site (original values from FEH Web Service)

Site Code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT2000	FPEXT
Dee_HF	0.915	0.58	0.386	42.68	152.7	1476	44.21	0.0013	0.0487
Dee_Llan	0.917	0.58	0.388	44.62	156.2	1457	43.88	0.0014	0.0475
Dee_Erb	0.936	0.55	0.403	59.44	152.3	1352	42.59	0.0043	0.0471

[2] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

[3] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.3 Checking catchment descriptors

Item	Comment
Record how catchment boundary was checked • Describe any changes • Refer to maps if required	The catchment boundary has been checked against open source OS Maps, and is considered to be sufficiently accurate.
Record how other catchment descriptors were checked, especially soils • Describe any changes • Include a before and after table if required	HOST based catchment descriptors checked against BGS online soil mapping; found to be realistic.
Source of URBEXT / URBAN	FEH Web Service
Method for updating URBEXT / URBAN • Refer to WINFAP v4 Urban Adjustment procedures / guidance • CPRE formula from FEH Volume 4 / CPRE formula from 2006 CEH report on URBEXT2000[4]	As per WINFAF v4 methods, using URBEXT2000

[4] http://sciencesearch.defra.gov.uk/Document.aspx?Document=FD1919_5228_TRP.pdf#page=35

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	Comment
Summarise reasons specific to study, for example lumped estimates at key locations for purpose of checking modelled peak flows.	Lumped catchment estimate at site of interest for localised flood modelling of design event flows.

5.2 Overview of QMED method

What method of QMED estimation was used?	Comments
State method/s used to estimate QMED in study and why, for example gauged data, donor transfer, multiple donor transfer, flow variability, bankfull width or user defined.	Single gauge donor transfer from same watercourse, located downstream (upstream for Erbistock). This donor gauge is the closest site suitable for QMED, and has similar catchment descriptors in all respects. It will also reflect the attenuation of peak flows by the Dee Regulation Scheme. The catchment centroid based distance moderation factor was excluded from the data transfer process, as it produced QMED values upstream that were greater than the observed value at the donor site located downstream, which is counterintuitive.

Summary of QMED estimates at each site:

Site code	QMED rural (from CDs) (m^3s^{-1})	Final method	Final estimate of QMED (m^3s^{-1})
Dee_HF	287.4	DT (no distance moderation)	190.9
Dee_Llan	291.6	DT (no distance moderation)	193.7
Dee_Erb	343.1	DT (no distance moderation)	227.9

Note: Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

5.3 Search for donor sites for QMED

Comment on potential donor sites based on the above sections	Comments
• Number of potential donor sites available • Distances from subject site • Similarities in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data	There is only one donor suitable to the subject sites that is both geographically close and comparable in all catchment descriptors, including catchment area. This site is also appropriate for estimating QMED via data transfer.

5.4 Uncertainty in QMED

The reduction in uncertainty as a result of applying data transfer is modest, with the 68% confidence interval for QMED on a rural catchment narrowing from:

- 0.69-1.45 times the estimate with no donor

To:

- 0.70-1.42 times with one donor

and

- 0.71-1.40 times with six donors.

These figures are taken from Technical Guidance 12_17. Despite this relatively small effect on the degree of confidence, the estimate of QMED can change markedly as the result of some data transfers.

This study has adjusted the 'as rural' catchment descriptor QMED estimate, using one donor. The uncertainty, based on the 68% and 95% confidence intervals, can be described using the Factorial Standard Errors associated with one donor, for the 2-year (QMED) event, as given in Technical Guidance 12_17.

Site code	QMED (m^3s^{-1})	Bound	No donor		One donor		Six donors	
			FSE (68% CI)	QMED	FSE (68% CI)	QMED	FSE (68% CI)	QMED
Dee_HF	190.9	Upper			0.70	133.6		
		Lower			1.42	271.1		
Dee_Llan	193.7	Upper			0.70	135.6		
		Lower			1.42	275.1		
Dee_Erb	227.9	Upper			0.70	159.5		
		Lower			1.42	323.6		

Site code	QMED (m^3s^{-1})	Bound	No donor		One donor		Six donors	
			FSE (95% CI)	QMED	FSE (95% CI)	QMED	FSE (95% CI)	QMED
Dee_HF	190.9	Upper			0.50	95.5		
		Lower			2.02	385.6		
Dee_Llan	193.7	Upper			0.50	96.9		
		Lower			2.02	391.3		
Dee_Erb	227.9	Upper			0.50	114.0		
		Lower			2.02	460.4		

5.5 Derivation of pooling groups

Pooling groups were created within WINFAP v4 for each of the subject sites. An URBEXT2000 threshold of 0.3 was used to create the pooling groups in order to make maximum use of local data. The Heterogeneity statistic (H2) for the pooling groups were assessed; this provides an indication of whether a review of the pooling group is required (not required, optional, desirable or essential). The similarity of the subject site against stations within the pooling group is assessed by the Similarity Distance Measure (SDM) and is a function of Area, SAAR, FARL and FPEXT. However, it is good practice to review the pooling group to check other parameters e.g. BFIHOST and the history of the gauge, gauge record and rating quality on the NRFA website (https://nrfa.ceh.ac.uk/data/search).

As per the Environment Agency guidelines, modifications to the pooling group tend to have a relatively minor effect on the final design flow (compared with, for example, the selection of donor sites for QMED). Science Report SC050050[5] indicates that apart from the first four or five stations within a pooling group (i.e. lowest SDM), the record length at a station will only have a modest effect on its weight within the pooling group (unless the record is very short). The review of the pooling group has therefore focused on the first five stations within each pooling group, extending further where required to include stations that have moved up position following removal of others, gauges with a short record, and catchments which have extreme catchment descriptor values in comparison to the subject sites.

The table below summarises the pooling groups used in this study and provided in Annex A. Annex A also notes the reasons for removing catchments from the initial pooling group and which stations were added in to the pooling group to ensure that sufficient years of data (>500) were included in the final group.

One pooling group was produced for Horseshoe Falls and Llangollen subject sites, since the catchment descriptors upon which pooling groups are formed are very similar at these two sites. A separate pooling group was produced for Erbistock, due to greater variance from the other two sites in terms of those descriptors.

Name of Group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons. Include any sites that were investigated but retained in the group	Weighted average L-moments (L-CV and L-skew before urban adjustment)
Dee_HF_PG_Rev1_URB	Dee_HF	No	- Removed site 16001 (site from same river already in PG – removal avoids duplication of AMAX data) - Removed site 62001 (site is an outlier in terms of growth curve and L-moment plots compared to remainder of sites in PG – most critically the Dee at Manley Hall) - Added site 25001 (maintain record length; site was appropriate)	L-CV = 0.166 L-Skew = 0.176
Dee_Erb_PG_Rev1_URB	Dee_Erb	No	- Removed sites 62001 and 60001 (site is an outlier in terms of growth curve and L-moment plots compared to remainder of sites in PG – most critically the Dee at Manley Hall) -	L-CV = 0.174 L-Skew = 0.203

The table below details the H2 score and requirement for pooling group review for in the initial and final pooling groups for each site

Catchment	Initial Pooling Group H2 value	Recommendation for Pooling Group Review	Final Pooling Group H2 value	Recommendation for Final Pooling Group Review
Dee (Horseshoe Falls)	0.98	Not required	-0.38	Not required
Dee (Erbistock)	0.05	Not required	-0.67	Not required

5.6 Derivation of flood growth curves at subject sites

Site Code	Method (SS, P, ESS, FH)	If P, ESS or FH, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustment	Growth Curve Factor for 100 year return period
Dee_HF Dee_Llan	P	Dee_HF_PG_Rev1_URB	GEV (acceptable fit, and GL was not acceptable)	Adjusted for urbanisation using WINFAPv4	Location = 0.908 Scale = 0.251 Shape = -0.009	2.087
Dee_Erb	P	Dee_Erb_PG_IRev1_URB	GEV (acceptable fit, and GL was not acceptable)	Adjusted for urbanisation using WINFAPv4	Location = 0.906 Scale = 0.255 Shape = -0.051	2.229

Growth Curve Factors for the following return periods - GEV distribution for Dee_HF Pooling Group
Applicable to Horseshoe Falls and Llangollen subject sites

Distribution	2	5	10	20	30	50	75	100	200
GEV	1.000	1.287	1.479	1.664	1.771	1.905	2.011	2.087	2.270

Growth Curve Factors for the following return periods - GEV distribution for Dee_Erb Pooling Group
Applicable to Erbistock subject sites

Distribution	2	5	10	20	30	50	75	100	200
GEV	1.000	1.304	1.514	1.724	1.849	2.008	2.136	2.229	2.458

Growth Curve Factors for the following return periods - GEV distribution from Manley Hall (single-site analysis) – for comparison

Distribution	*2*	*5*	*10*	*20*	*30*	*50*	*75*	*100*	*200*
GEV	*1.000*	*1.318*	*1.537*	*1.754*	*1.882*	*2.046*	*2.177*	*2.272*	*2.504*

5.7 Flood estimates from the statistical method

QMED estimated using donor transfer method and adjusted using UAF at site location.

Flood Peak (m^3s^{-1}) for the following return periods

Site Code	2	5	10	20	30	50	75	100	200
Dee_HF	191.1	246.0	282.7	318.0	338.5	364.1	384.3	398.9	433.8
Dee_Llan	193.9	249.6	286.8	322.7	343.4	369.4	389.9	404.7	440.2
Dee_Erb	228.8	298.4	346.5	394.5	423.1	459.5	488.8	510.1	562.5

6 Revitalised flood hydrograph (ReFH2) method

6.1 Application of ReFH2 model

What is the purpose of applying this method?	Comment
Summarise reasons specific to study, for example: lumped estimates at key locations for the purpose of checking modelled peak flow estimates, distributed approach to apply inflows to a hydraulic model, deriving hydrograph shapes only, extending the flood frequency curve out to extreme events (long return periods).	Lumped estimate at site, to compare against FEH Statistical estimate and to provide an inflow hydrograph to any local hydraulic modelling.

6.2 Parameters for ReFH2 model

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

Site Code	Details of Method OPT: Optimisation BR: base flow recession fitting CD: catchment descriptors DT: Data Transfer	Tp_{rural} (hours) Time to peak	Tp_{urban} (hours) Time to peak	C_{max} (mm) Maximum storage capacity	Pr_{imp} (% runoff for impermeable surfaces)	BL (hours) Base flow lag	BR Base flow recharge
Dee_HF	CD	6.20	n/a	271.43	n/a	50.72	1.23 (2-year)
Dee_Llan	CD	6.31	n/a	272.84	n/a	51.38	1.25 (2-year)
Dee_Erb	CD	7.96	n/a	286.88	n/a	57.33	1.39 (2-year)
Brief description of any flood event analysis undertaken: Provide further details here or in a project report	n/a						

6.3 Design events for ReFH2 method: Lumped catchments

Site Code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Source of design rainfall statistic (FEH13 or FEH99)
Dee_HF	Rural	Winter	15	FEH13
Dee_Llan	Rural	Winter	15	FEH13
Dee_Erb	Rural	Winter	19	FEH13

6.4 Flood estimates from the ReFH2 method

- Please indicate whether you have used urban or rural results
- We recommend that urban results are used regardless of the extent of urbanisation at the subject sites to avoid discontinuity when URBEXT reaches a given threshold.

As per the Technical Guidance Document: ReFH 2.2, the urban results are reported in the table below. These results take account of the urban extent within the catchment based on URBEXT2000 and are considered representative of existing conditions.

Flood Peak (m^3s^{-1}) for the following return periods

Site Code	2	5	10	20	30	50	75	100	200
Dee_HF	336.7	430.1	502.8	583.3	635.4	705.9	765.7	810.1	924.9
Dee_Llan	339.6	434.2	508.0	589.5	642.0	713.9	774.1	818.6	934.9
Dee_Erb	357.3	455.5	531.3	615.6	669.3	741.5	802.8	848.2	963.6

7 Discussion and summary of results

7.1 Comparison of results from different methods

This table compares peak flows from the ReFH2 method with those from the FEH Statistical method (donor adjusted inclusive of urbanisation) at each site for two key return periods. This illustrates that flow estimates from the FEH statistical method are approximately 10% less than those derived using ReFH2.

	Return period 2 years (50% AEP)			Return Period 100 years (1% AEP)		
Site Code	Statistical	ReFH2	Ratio (ReFH2/Statistical)	Statistical	ReFH2	Ratio (ReFH2/Statistical)
Dee_HF	191.1	336.7	1.76	398.9	810.1	2.03
Dee_Llan	193.9	339.6	1.75	404.7	818.6	2.02
Dee_Erb	228.8	357.3	1.56	510.1	848.2	1.66

7.2 Final choice of method

Choice of method and reason Include reference to type of study, nature of catchment and type of data available	FEH Statistical – preferred approach of NRW, and the catchment and associated sites are applicable with the approaches. For the subject sites considered in this calculation, the method selected uses gauged data on the Dee as part of the flow estimation process, from a site that is geographically close and hydrologically similar. This also allows consideration of some impact of the Dee regulation scheme. (This may explain some of the significant difference between ReFH2 and Statistical).
How will the flows be applied to a hydraulic model?	Direct inflow (single boundary)

7.3 Assumptions, limitations, and uncertainty

List the main assumptions made specific to the study	Assumed that NRFA classification of Manley Hall subject site as suitable for QMED is appropriate, and that the observed data from this gauge is accurate.
Discuss any particular limitations For example applying methods outside the range of catchment types or return periods for which they were developed	Site is ungauged, but watercourse is not.
Give what information you can on uncertainty in the design peak flows or in the methodology For example using the methods detailed in 'Making better use of local data in flood frequency estimation' - Science Report SC130009/R	Uncertainty has been quantified in respect of the QMED estimate, based on standard factorial standard errors associated with transferring data from a single donor site.
Comment on the suitability of the results for future studies For example at nearby locations or for different purposes	Suitable for studies requiring comparison of impacts, but not for flood defence schemes or mapping (more work required to investigate hydrology estimates).
Give any other comments on the study For example suggestions for additional work	Consider sensitivity testing flow in respect of QMED uncertainty. More importantly, more detailed flood related studies should investigate flood hydrograph shape and critical durations/ volumes from observed data, rather than relying on a scaled ReFH2 boundary.

7.4 Checks

Are the results consistent, for example at confluences?	n/a
What do the results imply regarding the return periods / frequency of floods during the period of record?	n/a
What is the 100-year (1% AEP) growth curve factor? Is this realistic? (The guidance suggests a typical range of 2.1 – 4.0)	2.011 to 2.136 – although this is a low growth curve factor, it is likely to be realistic since the pooling groups are strongly homogeneous, the GEV was the preferred (and only acceptable fitting) distribution, and the catchment includes relatively high reservoir/ lake attenuation.
If 1000 year (0.1% AEP) flows have been derived, what is the range of ratios for the 1000-year (0.1% AEP) flow over the 100-year (1% AEP) flow?	n/a
What is the range of specific run-offs (l/s/ha) do the results equate to? Are there any inconsistencies?	1% AEP specific run-off is: - 5.30 l/s/ha at Horseshoe Falls - 5.16 l/s/ha at Llangollen - 4.94 l/s/ha at Erbistock. Values are sensible as specific runoff usually decreases with increasing catchment area, but no significant variation.
How do the results compare with those of other studies? Explain the difference and conclude which results should be preferred	n/a
Are the results compatible with the longer-term flood history?	n/a
Describe any other checks on the results	n/a

7.5 Final results

The final peak flow results for use in the hydraulic model are provided in the table below. This includes the appropriate allowances for climate change.

	Flood peak ($m^3 s^{-1}$) for required return periods (in years)						
Site Code	2	10	100	100 +30%	100 +40%	100 + 85%	1000
Dee_HF	191.1	282.7	398.9	518.5	n/a	n/a	n/a
Dee_Llan	193.9	286.8	404.7	526.1	n/a	n/a	n/a
Dee_Erb	228.8	346.5	510.1	663.1	n/a	n/a	n/a

7.6 Uncertainty bounds

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

The uncertainty bounds given below are based on the 68% confidence interval surrounding the QMED estimate only.

Site Code	Flood peak ($m^3 s^{-1}$) for required return periods (in years)							
	2		10		100		1000	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Dee_HF	133.8	271.4	197.9	404.3	275.2	582.4	n/a	n/a
Dee_Llan	135.7	275.3	200.8	410.1	279.2	590.9	n/a	n/a
Dee_Erb	160.2	324.9	242.6	495.5	352.0	744.8	n/a	n/a

8 Annex A–WINFAP v4 Pooling Groups

8.1 Initial pooling group composition - Dee_HF_PG_Initial_URB

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
16004 (Earn @ Forteviot Bridge)	0.178	35	249.200	0.137	0.095	0.424	
16001 (Earn @ Kinkell Bridge)	0.388	58	203.984	0.110	0.019	1.522	
54028 (Vyrnwy @ Llanymynech)	0.394	45	264.248	0.158	0.229	0.702	
67015 (Dee @ Manley Hall)	0.452	49	226.000	0.182	0.198	0.748	
27002 (Wharfe @ Wetherby Flint Mill)	0.465	83	235.996	0.165	0.211	0.446	
79002 (Nith @ Friars Carse)	0.488	49	447.640	0.138	0.191	0.798	
84004 (Clyde @ Sills of Clyde)	0.499	64	210.636	0.170	0.240	0.176	
56001 (Usk @ Chainbridge)	0.516	59	369.700	0.174	0.221	1.091	

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
27089 (Wharfe @ Tadcaster)	0.540	28	210.337	0.214	0.167	2.050	
27098 (Calder @ Dewsbury)	0.557	24	259.357	0.177	0.086	1.329	
62001 (Teifi @ Glanteifi)	0.577	60	208.900	0.208	0.368	1.716	
Total		554					
Weighted means				0.165	0.184		

8.2 Final pooling group composition - Dee_HF_PG_Rev1_URB

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
16004 (Earn @ Forteviot Bridge)	0.178	35	249.200	0.137	0.095	1.304	
54028 (Vyrnwy @ Llanymynech)	0.394	45	264.248	0.158	0.229	1.082	
67015 (Dee @ Manley Hall)	0.452	49	226.000	0.182	0.198	1.116	
27002 (Wharfe @ Wetherby Flint Mill)	0.465	83	235.996	0.165	0.211	0.368	
79002 (Nith @ Friars Carse)	0.488	49	447.640	0.138	0.191	0.878	
84004 (Clyde @ Sills of Clyde)	0.499	64	210.636	0.170	0.240	0.499	
56001 (Usk @ Chainbridge)	0.516	59	369.700	0.174	0.221	0.930	
27089 (Wharfe @ Tadcaster)	0.540	28	210.337	0.214	0.167	1.798	
27098 (Calder @ Dewsbury)	0.557	24	259.357	0.177	0.086	1.317	
25001 (Tees @ Darlington Broken Scar)	0.583	63	396.100	0.165	0.107	0.708	
Total		499					
Weighted means				0.166	0.176		

8.3 Initial pooling group composition - Dee_Erb_PG_Initial_URB

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
67015 (Dee @ Manley Hall)	0.045	49	226.000	0.182	0.198	0.823	
56001 (Usk @ Chainbridge)	0.330	59	369.700	0.174	0.221	1.303	
8005 (Spey @ Boat of Garten)	0.348	68	170.150	0.229	0.210	1.772	
55007 (Wye @ Erwood)	0.348	79	517.502	0.187	0.230	0.104	
84018 (Clyde @ Tulliford Mill)	0.359	38	247.738	0.170	0.222	1.054	
71009 (Ribble @ New Jumbles Rock)	0.365	50	534.000	0.174	0.264	0.143	
84003 (Clyde @ Hazelbank)	0.414	51	274.929	0.144	0.250	1.101	
62001 (Teifi @ Glanteifi)	0.424	60	208.900	0.208	0.368	0.910	
60001 (Tywi @ ty Castell)	0.424	47	387.800	0.240	0.410	1.790	
Total		501					
Weighted means				0.189	0.253		

8.4 Final pooling group composition - Dee_Erb_PG_Rev1_URB

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
67015 (Dee @ Manley Hall)	0.045	49	226.000	0.182	0.198	0.557	
56001 (Usk @ Chainbridge)	0.330	59	369.700	0.174	0.221	1.421	
8005 (Spey @ Boat of Garten)	0.348	68	170.150	0.229	0.210	1.809	
55007 (Wye @ Erwood)	0.348	79	517.502	0.187	0.230	0.156	
84018 (Clyde @ Tulliford Mill)	0.359	38	247.738	0.170	0.222	1.018	
71009 (Ribble @ New Jumbles Rock)	0.365	50	534.000	0.174	0.264	0.523	
84003 (Clyde @ Hazelbank)	0.414	51	274.929	0.144	0.250	1.161	
16004 (Earn @ Forteviot Bridge)	0.440	35	249.200	0.137	0.095	1.545	
71001 (Ribble @ Samlesbury)	0.442	59	629.757	0.162	0.117	0.810	
Total		488					
Weighted means				0.175	0.202		

Annex 2 River Dee LIFE - Morlas Brook: Flood Estimation Calculation Record

River Dee LIFE - Morlas Brook

Flood Estimation Calculation Record

60627686

December 2020

1 Flood estimation calculation record

Introduction

This document provides a record of the calculations and decisions made during flood estimation for the Morlas Brook. The information given here should enable the work to be reproduced in the future.

Contents

Approval

	Signature	Name and qualifications
Calculations prepared by:		Andrew Heath-Brown BSc (Hons), MSc, MCIWEM
Calculations checked by:		Helen Harfoot BSc (Hons), MSc, MCIWEM, CSci, CEnv
Calculations approved by:		

Revision History

Revision	Revision Date	Details	Authorised	Name	Position

Abbreviations

AEP	Annual exceedance probability
AM	Annual maximum
AREA	Catchment area (km^2)
BFI	Base flow index
BFIHOST	Base flow index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
DPLBAR	Mean drainage path length (km)
DTM	Digital Terrain Model
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FPEXT	Floodplain extent
FSR	Flood Studies Report
HOST	Hydrology of soil types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks over threshold
QMED	Median annual flood (with return period ~2 years)
ReFH1	Revitalised Flood Hydrograph 1 method (2005)
ReFH2	Revitalised Flood Hydrograph 2 method (2013)
SAAR	Standard average annual rainfall (mm)
SPR	Standard percentage run-off
SPRHOST	Standard percentage run-off derived using the HOST soil classification
Tp (0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent
WINFAP	Windows Frequency Analysis Package – used for FEH statistical method

2 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections. The aim of the table is to enable quick and easy identification of the type of assessment undertaken. This should assist in identifying an appropriate reviewer and the ability to compare different studies more easily.

Catchment location	Morlas Brook, Llandudno
Purpose of study and scope	Routine calculation of peak flood flow estimates, for use as part of a fish passage solution assessment.
Key catchment features	No unusual features
Flooding mechanisms	n/a
Gauged / ungauged	Ungauged
Final choice of method	FEH Statistical
Key limitations / uncertainties in results	Lack of flow data to enhance or validate FEH estimates; reliance on FEH models.

2.1 Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval. Results tables in this document contain both return period and AEP titles; both rows can be retained, or the relevant row can be retained and the other removed, depending on the requirement of the study.

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

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

3 Method statement

3.1 Overview of requirements for flood estimates and hydraulic modelling

Item	Comments
Give an overview which includes: • purpose of study including a short discussion if there is existing hydrology reports and estimates, when they were done and why we are updating the hydrology (e.g. new data or superseded methods) • approximate number and type of flood estimates required • peak flows and/or hydrographs? • range of design event AEPs (%) • climate change allowances (ref. relevant guidance)	The study is an investigation into the possible implementation of fish passage at a number of weirs within the Dee catchment. The scope requires assessment for two flood flow events (likely 2-year and 100-year), hence the need for peak flow estimates. There are no known existing peak flow estimates. A range of peak flow estimates will be produced between 2 and 200-year return periods. However, the 1,000-year estimate will not be included. Peak flows and hydrographs are required, at a single location only.

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report. Include general catchment map and specific map of hydraulic model extents and inflow locations.	Peak flows and hydrographs have been estimated at the Morlas Brook weir (SJ 31200 38300). The main report provides a description of the catchment and the modelled reach, including details of where the flow estimate will be entered in the hydraulic model.
Previous Hydrology studies	None known

3.3 Source of flood peak data

Item	Comments
Was the NRFA Peak Flows dataset used? If so, which version? If not, why not? Record any changes made.	NRFA peak flows dataset, Version 9, released October 2020. This contains data up to the end of water year 2018-19, and provisional data for water year 2019/2020 at stations which set new records.

3.4 Gauging stations (flow or level)

The following gauging stations include ones considered and used as part of a QMED data transfer, but are not necessarily one the study reach/ within the study catchment.

Watercourse	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level)	Start of record and end if station closed
Ceiriog	Brynkinalt Weir	067005	67005	113.7	Rated	1952

3.5 Data available at each flow gauging station

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling?	Data quality check needed?	Other comments on station and flow data quality
Brynkinalt Weir	1952-1953	No	Yes	No	No	

3.6 Other data available and how it has been obtained

Type of data	Data relevant to this study	Data available	Source of data	Details
Check flow gaugings (if planned rating review)	~~Yes~~ / No	Yes / No	n/a	n/a
Rating equations	~~Yes~~ / No	Yes / No	n/a	n/a
Historic flood data	~~Yes~~ / No	Yes / No	n/a	n/a
Flow or level data for events	~~Yes~~ / No	Yes / No	n/a	n/a
Results from previous studies	~~Yes~~ / No	Yes / No	n/a	n/a
Other information e.g. groundwater, tides etc	~~Yes~~ / No	Yes / No	n/a	n/a

3.7 Initial choice of approach

Item	Comment
Is FEH appropriate? If not, describe why and give details of the other methods to be used.	Yes.
Initial choice of method(s) and reasons.	FEH Statistical – catchment has no unusual features, and is within limits if applicability for FEH Statistical method. FEH Statistical is preferred over ReFH2 (as per NRW GN008) in most cases, unless demonstrated otherwise.
How will hydrograph shapes be derived if needed? E.g. ReFH1, ReFH2 or average hydrograph shape from gauge data	The ReFH2 method will be used to define hydrograph shape, based on the recommended duration. If the FEH Statistical method produces significantly different flows, parameters of the ReFH2 model may need to be adjusted to ensure sensible hydrograph volumes are maintained.
Will the catchment be split into sub-catchments? If so, how?	The catchment will not be split into subcatchments. It is anticipated that the modelled reach will be short, and there are no significant tributaries in close proximity to the subject site with obvious differences to the overall contributing catchment.
Software to be used (with version numbers) (**delete as appropriate**)	FEH Web Service[1] / ~~WINFAP-FEH v3.0.003~~[2] / WINFAP 4[3] / ~~ReFH spreadsheet~~ / ReFH2.2 / ~~Flood Modeller Pro~~

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

[2] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

4 Locations where flood estimates are required

4.1 Summary of subject sites

The table below lists the locations of subject sites.

Site Code	Type of Estimate (L – lumped catchment; S- Sub-catchment)	Watercourse	Site	Grid Reference	Area on FEH Web Service (km^2)	Revised area if altered
MB	L	Morlas Brook	Weir	SJ 31200 38300	21.1	n/a

Reasons for choosing above locations: Location of proposed fish pass

4.2 Important catchment descriptors at each subject site (original values from FEH Web Service)

Site Code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT2000	FPEXT
MB	0.996	0.51	0.502	6.52	84	914	37.43	0.0135	0.045

[3] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.3 Checking catchment descriptors

Item	Comment
Record how catchment boundary was checked • Describe any changes • Refer to maps if required	The catchment boundary has been checked against open source OS Maps, and is considered to be sufficiently accurate.
Record how other catchment descriptors were checked, especially soils • Describe any changes • Include a before and after table if required	HOST based catchment descriptors checked against BGS online soil mapping; found to be realistic.
Source of URBEXT / URBAN	FEH Web Service
Method for updating URBEXT / URBAN • Refer to WINFAP v4 Urban Adjustment procedures / guidance • CPRE formula from FEH Volume 4 / CPRE formula from 2006 CEH report on URBEXT2000[4]	Standard WINFAP v2 UAF application

[4] http://sciencesearch.defra.gov.uk/Document.aspx?Document=FD1919_5228_TRP.pdf#page=35

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	Comment
Summarise reasons specific to study, for example lumped estimates at key locations for purpose of checking modelled peak flows.	Lumped catchment estimate at site of interest for localised flood modelling of design event flows.

5.2 Overview of QMED method

What method of QMED estimation was used?	Comments
State method/s used to estimate QMED in study and why, for example gauged data, donor transfer, multiple donor transfer, flow variability, bankfull width or user defined.	Single gauge donor transfer from adjacent watercourse catchment. This donor gauge (67005) is the closest site suitable for QMED, and has similar catchment descriptors in all respects. (Note that although the gauged site has an upstream catchment that is 5-6 times larger than that of the subject site, this is within acceptable bounds, in terms of donor transfer).

Summary of QMED estimates at each site:

Site code	QMED rural (from CDs) (m^3s^{-1})	Final method	Final estimate of QMED (m^3s^{-1})
MB	6.149	DT	5.091

Note: Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

5.3 Search for donor sites for QMED

Comment on potential donor sites based on the above sections	Comments
• Number of potential donor sites available • Distances from subject site • Similarities in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data	3No. sites within 20km. Of the 3No. sites (67005, 54020 and 54038), all were relatively similar in terms of CDs, although 54020 was less similar in terms of BFIHOST, and 67005 was much more comparable in terms of AREA. 67005 however had a shorter record length (22 years) compared to the others (approx. 50 years). All are classed as suitable for QMED on NRFA.

5.4 (Multiple) donor transfers and QMED adjustment

The donor adjustment method embedded within WINFAPv4 has been utilised any adjustment for urbanisation[5] has also been applied using the functionality within WINFAPv4.

The weighting of each donor catchment to provide the adjusted QMED is not provided within WINFAPv4 but is described within Kjeldsen et al 2014[5].

See WINFAP screenshots below.

[5] Wallingford HydroSolutions (2016), WINFAP 4 Urban adjustment procedures, Wallingford HydroSolutions Ltd 2016.

QMED Rural Estimation

Method | Donor Adjustment | Flow Variability

Target Info — QMED Catchment Descriptors: 6.149

QMED Donor Adjusted: 5.009

Donor Adjusted F.S.E.: 1.394

No. Donors: 1

Show

○ All Sites ◉ Only sites suitable for QMED

URBEXT 2000 < 0.0300 Apply

	Station	Distance	URBEXT	Use QMED Obs Deurbanised	QMED Obs	QMED Deurbanised	QMED CDs	Centroid X	Centroid Y	Area	
1	999200 (Morlasbrook @ Weir)		0.013					327925	335328	21.110	914
2	67005 (Ceiriog @ Brynkinalt Weir)	10.45	0.001	☐	28.930	28.889	49.865	317500	336107	111.720	1196
3	54020 (Perry @ Yeaton)	11.45	0.014	☐	10.590	10.347	12.927	337383	328872	188.080	739
4	54038 (Tanat @ Llanyblodwel)	17.24	0.001	☐	79.126	79.047	99.498	312720	327196	241.130	1274
5	67009 (Alyn @ Rhydymwyn)	24.16	0.002	☐	8.780	8.759	15.308	319020	357784	81.600	968
6	67008 (Alyn @ Pont-y-capel)	24.24	0.029	☐	21.635	20.781	35.712	323013	359066	225.650	917
7	66005 (Clwyd @ Ruthin Weir)	24.49	0.005	☐	17.030	16.922	24.673	309817	351811	96.390	958
8	54016 (Roden @ Rodington)	24.67	0.013	☐	10.858	10.645	18.568	351700	328761	261.940	693
9	67015 (Dee @ Manley Hall)	25.27	0.004	☐	226.000	225.004	338.746	303097	340024	1008.740	1367
10	54028 (Vyrnwy @ Llanymynech)	26.17	0.001	☐	264.248	264.000	293.570	307743	318661	779.140	1339
11	54001 (Severn @ Bewdley)	26.94	0.020	☐	334.052	325.845	489.319	336745	309876	4329.900	912

Site of Interest ■ Selected Donor ■

QMED Rural Estimation

Method | Donor Adjustment | Flow Variability

Target Info

QMED Catchment Descriptors: 6.149

QMED Donor Adjusted: 5.009

Donor Adjusted F.S.E.: 1.394

No. Donors: 1

Show

All Sites | Only sites suitable for QMED

URBEXT 2000 < 0.0300 Apply

	Station	Centroid X	Centroid Y	Area	SAAR	BFIHOST	FARL	Years of data	QMED Suitability	Pooling Suitability	Weight
1	999200 (Morlasbrook @ Weir)	327925	335328	21.110	914	0.520	0.996				
2	67005 (Ceiriog @ Brynkinalt Weir)	317500	336107	111.720	1198	0.462	1.000	22	Yes	No	0.377
3	54020 (Perry @ Yeaton)	337383	328872	188.080	739	0.654	0.954	57	Yes	Yes	0.368
4	54038 (Tanat @ Llanyblodwel)	312720	327196	241.130	1274	0.477	0.996	48	Yes	Yes	0.326
5	67009 (Alyn @ Rhydymwyn)	319020	357784	81.600	968	0.615	0.990	63	Yes	Yes	0.284
6	67008 (Alyn @ Pont-y-capel)	323013	359066	225.650	917	0.591	0.990	54	Yes	Yes	0.283
7	66005 (Clwyd @ Ruthin Weir)	309817	351811	96.390	958	0.518	0.995	43	Yes	Yes	0.282
8	54016 (Roden @ Rodington)	351700	328761	261.940	693	0.615	0.981	51	Yes	Yes	0.281
9	67015 (Dee @ Manley Hall)	303097	340024	1008.740	1367	0.431	0.934	49	Yes	Yes	0.277
10	54028 (Vyrnwy @ Llanymynech)	307743	318661	779.140	1339	0.439	0.969	45	Yes	Yes	0.272
11	54001 (Severn @ Bewdley)	336745	309876	4329.900	912	0.541	0.973	96	Yes	Yes	0.268

Site of Interest | Selected Donor

5.5 Uncertainty in QMED

The reduction in uncertainty as a result of applying data transfer is modest, with the 68% confidence interval for QMED on a rural catchment narrowing from:

- 0.69-1.45 times the estimate with no donor

to

- 0.70-1.42 times with one donor

and

- 0.71-1.40 times with six donors.

These figures are taken from Technical Guidance 12_17. Despite this relatively small effect on the degree of confidence, the estimate of QMED can change markedly as the result of some data transfers.

This study has adjusted the 'as rural' catchment descriptor QMED estimate, using one donor. The uncertainty, based on the 68% and 95% confidence intervals, can be described using the Factorial Standard Errors associated with **one donor**, for the 2-year (QMED) event, as given in Technical Guidance 12_17.

Site code	QMED rural (from CDs) (m^3s^{-1})	Bound	No donor		One donor		Six donors	
			FSE (68% CI)	QMED	FSE (68% CI)	QMED	FSE (68% CI)	QMED
	5.009	Upper			0.70	3.506		
		Lower			1.42	7.113		

Site code	QMED rural (from CDs) (m^3s^{-1})	Bound	No donor		One donor		Six donors	
			FSE (95% CI)	QMED	FSE (95% CI)	QMED	FSE (95% CI)	QMED
	5.009	Upper			0.50	2.505		
		Lower			2.02	10.118		

5.6 Derivation of pooling groups

Pooling groups were created within WINFAP v4 for each of the subject sites. An URBEXT2000 threshold of 0.3 was used to create the pooling groups in order to make maximum use of local data. The Heterogeneity statistic (H2) for the pooling groups were assessed; this provides an indication of whether a review of the pooling group is required (not required, optional, desirable or essential). The similarity of the subject site against stations within the pooling group is assessed by the Similarity Distance Measure (SDM) and is a function of Area, SAAR, FARL and FPEXT. However, it is good practice to review the pooling group to check other parameters e.g. BFIHOST and the history of the gauge, gauge record and rating quality on the NRFA website (https://nrfa.ceh.ac.uk/data/search).

As per the Environment Agency guidelines, modifications to the pooling group tend to have a relatively minor effect on the final design flow (compared with, for example, the selection of donor sites for QMED). Science Report SC050050[5] indicates that apart from the first four or five stations within a pooling group (i.e. lowest SDM), the record length at a station will only have a modest effect on its weight within the pooling group (unless the record is very short). The review of the pooling group has therefore focused on the first five stations within each pooling group, extending further where required to include stations that have moved up position following removal of others, gauges with a short record, and catchments which have extreme catchment descriptor values in comparison to the subject sites.

The table below summarises the pooling groups used in this study and provided in Annex A. Annex A also notes the reasons for removing catchments from the initial pooling group and which stations were added in to the pooling group to ensure that sufficient years of data (>500) were included in the final group.

Name of Group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons. Include any sites that were investigated but retained in the group	Weighted average L-moments (L-CV and L-skew before urban adjustment)
MB_PG_Rev2_URB	MB	No	- Removed site 7011 (record length less than 8 years) - Removed site 28058 (record length covers 1973-1986; records after this have been rejected – period of record considered unrepresentative of current hydrological conditions) - Considered removing sites 44008, 26016, 44013 and 26014 due to BHIHOST being in excess of 0.8. However, their respective growth curves were not clustered, or any different to others, and their L-moments were not 'outliers'. There were no other reasons to reject their validity. Sites therefore retained.	L-CV = 0.267 L-Skew = 0.238
			-	

The table below details the H2 score and requirement for pooling group review for in the initial and final pooling groups for each site

Catchment	Initial Pooling Group H2 value	Recommendation for Pooling Group Review	Final Pooling Group H2 value	Recommendation for Final Pooling Group Review
MB	1.64	Optional	1.08	Optional

5.7 Derivation of flood growth curves at subject sites

Site Code	Method (SS, P, ESS, FH)	If P, ESS or FH, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustment	Growth Curve Factor for 100 year return period
MB	P	MorlasBrook_PG_Rev2_URB	GL (acceptable fit, and preferred fit for UK catchments)	Adjusted for urbanisation using WINFAPv4	Location = 1.000 Scale = 0.267 Shape = -0.241	3.246

Growth Curve Factors for the following return periods for GL distribution for MB Pooling Group

Distribution	2	5	10	20	30	50	75	100	200
GL	1.000	1.440	1.774	2.145	2.387	2.723	3.019	3.246	3.861

5.8 Flood estimates from the statistical method

QMED estimated using donor transfer method and adjusted using UAF at site location.

	Flood Peak (m^3s^{-1}) for the following return periods								
Site Code	2	5	10	20	30	50	75	100	200
MB	5.1	7.3	9.0	10.9	12.2	13.9	15.4	16.5	19.7

6 Revitalised flood hydrograph (ReFH2) method

6.1 Application of ReFH2 model

What is the purpose of applying this method?	Comment
Summarise reasons specific to study, for example: lumped estimates at key locations for the purpose of checking modelled peak flow estimates, distributed approach to apply inflows to a hydraulic model, deriving hydrograph shapes only, extending the flood frequency curve out to extreme events (long return periods).	Lumped estimate at site, to compare against FEH Statistical estimate and to provide an inflow hydrograph to the hydraulic model.

6.2 Parameters for ReFH2 model

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

Site Code	Details of Method OPT: Optimisation BR: base flow recession fitting CD: catchment descriptors DT: Data Transfer	Tp_{rural} (hours) Time to peak	Tp_{urban} (hours) Time to peak	C_{max} (mm) Maximum storage capacity	Pr_{imp} (% runoff for impermeable surfaces)	BL (hours) Base flow lag	BR Base flow recharge
MB	CD	2.96		376.96		41.86	2.85 (2-year)

Brief description of any flood event analysis undertaken: Provide further details here or in a project report	n/a

6.3 Design events for ReFH2 method: Lumped catchments

Site Code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Source of design rainfall statistic (FEH13 or FEH99)
MB	Rural	Winter	5.5	FEH13

6.4 Flood estimates from the ReFH2 method

- Please indicate whether you have used urban or rural results
- We recommend that urban results are used regardless of the extent of urbanisation at the subject sites to avoid discontinuity when URBEXT reaches a given threshold.

As per the Technical Guidance Document: ReFH 2.2, the urban results are reported in the table below. These results take account of the urban extent within the catchment based on URBEXT2000 and are considered representative of existing conditions.

	Flood Peak (m^3s^{-1}) for the following return periods								
Site Code	2	5	10	20	30	50	75	100	200
MB	5.812	8.036	9.827	11.945	13.363	15.316	17.020	18.307	21.613

7 Discussion and summary of results

7.1 Comparison of results from different methods

This table compares peak flows from the ReFH2 method with those from the FEH Statistical method (donor adjusted inclusive of urbanisation) at each site for two key return periods. This illustrates that flow estimates from the FEH statistical method are approximately 10% less than those derived using ReFH2.

	Return period 2 years (50% AEP)			Return Period 100 years (1% AEP)		
Site Code	Statistical	ReFH2	Ratio (ReFH2/Statistical)	Statistical	ReFH2	Ratio (ReFH2/Statistical)
MB	5.1	5.8	1.14	16.5	18.3	1.11

7.2 Final choice of method

Choice of method and reason Include reference to type of study, nature of catchment and type of data available	FEH Statistical – preferred approach of NRW, and the catchment is suitable for application.
How will the flows be applied to a hydraulic model?	Direct inflow (single boundary)

7.3 Assumptions, limitations, and uncertainty

List the main assumptions made specific to the study	Lack of flow data to enhance or validate FEH estimates; reliance on FEH models.
Discuss any particular limitations For example applying methods outside the range of catchment types or return periods for which they were developed	Site is ungauged.
Give what information you can on uncertainty in the design peak flows or in the	Uncertainty has been quantified in respect of the QMED estimate, based on standard

methodology For example using the methods detailed in 'Making better use of local data in flood frequency estimation' - Science Report SC130009/R	factorial standard errors associated with transferring data from a single donor site.
Comment on the suitability of the results for future studies For example at nearby locations or for different purposes	Suitable for studies requiring comparison of impacts, but not for flood defence schemes or mapping (more work required to investigate hydrology estimates).
Give any other comments on the study For example suggestions for additional work	Consider sensitivity testing flow in respect of QMED uncertainty.

7.4 Checks

Are the results consistent, for example at confluences?	n/a
What do the results imply regarding the return periods / frequency of floods during the period of record?	n/a
What is the 100-year (1% AEP) growth curve factor? Is this realistic? (The guidance suggests a typical range of 2.1 – 4.0)	3.246 – this is a realistic growth factor for this type of catchment.
If 1000 year (0.1% AEP) flows have been derived, what is the range of ratios for the 1000-year (0.1% AEP) flow over the 100-year (1% AEP) flow?	n/a
What is the range of specific run-offs (l/s/ha) do the results equate to? Are there any inconsistencies?	1% AEP specific run-off is 7.83 l/s/ha. No other FEPs to compare against.
How do the results compare with those of other studies? Explain the difference and conclude which results should be preferred	n/a
Are the results compatible with the longer-term flood history?	n/a
Describe any other checks on the results	n/a

7.5 Final results

The final peak flow results for use in the hydraulic model are provided in the table below. This includes the appropriate allowances for climate change.

	Flood peak ($m^3 s^{-1}$) for required return periods (in years)						
Site Code	2	10	100	100 +30%	100 +40%	100 + 85%	1000
MB	5.1	9.0	16.5	21.5	n/a	n/a	n/a

7.6 Uncertainty bounds

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

The uncertainty bounds given below are based on the 68% confidence interval surrounding the QMED estimate only.

Site Code	Flood peak ($m^3 s^{-1}$) for required return periods (in years)							
	2		10		100		1000	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
MB	3.6	7.2	6.3	12.9	11.4	24.1	n/a	n/a

8 Annex A–WINFAP v4 Pooling Groups

8.1 Initial pooling group composition

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
44008 (South Winterbourne @ Winterbourne Steepleton)	0.394	40	0.434	0.411	0.337	0.914	
27010 (Hodge Beck @ Bransdale Weir)	0.454	41	9.420	0.224	0.293	0.476	
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	0.521	9	5.777	0.271	0.151	3.896	
26016 (Gypsey Race @ Kirby Grindalythe)	0.563	22	0.100	0.321	0.266	0.242	
25019 (Leven @ Easby)	0.580	41	5.090	0.342	0.386	0.795	
84035 (Kittoch Water @ Waterside)	0.605	28	20.033	0.126	0.104	0.684	
27081 (Oulton Beck @ Oulton Farrer Lane)	0.624	33	2.395	0.245	0.249	0.098	
73015 (Keer @ High Keer Weir)	0.756	28	12.375	0.204	0.260	0.439	
44013 (Piddle @ Little Puddle)	0.767	27	0.857	0.501	0.295	2.451	
72014 (Conder @ Galgate)	0.794	51	16.646	0.231	0.160	0.071	
41020 (Bevern Stream @ Clappers Bridge)	0.806	50	13.575	0.207	0.182	0.282	
26014 (Water Forlornes @ Driffield)	0.820	21	0.424	0.306	0.147	0.438	
28041 (Hamps @ Waterhouses)	0.863	34	26.313	0.219	0.288	0.881	
28058 (Henmore Brook @ Ashbourne)	0.866	13	8.838	0.188	-0.109	3.409	
27032 (Hebden Beck @ Hebden)	0.905	53	4.052	0.204	0.237	0.357	
24006 (Rookhope Burn @ Eastgate)	0.919	20	24.620	0.152	0.117	0.567	
Total		511					
Weighted means				0.263	0.224		

8.2 Final pooling group composition

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
27010 (Hodge Beck @ Bransdale Weir)	0.454	41	9.420	0.224	0.293	0.351	
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	0.521	9	5.777	0.271	0.151	3.642	
25019 (Leven @ Easby)	0.580	41	5.090	0.342	0.386	1.327	
84035 (Kittoch Water @ Waterside)	0.605	28	20.033	0.126	0.104	0.901	
27081 (Oulton Beck @ Oulton Farrer Lane)	0.624	33	2.395	0.245	0.249	0.081	
73015 (Keer @ High Keer Weir)	0.756	28	12.375	0.204	0.260	0.321	
72014 (Conder @ Galgate)	0.794	51	16.646	0.231	0.160	0.042	
41020 (Bevern Stream @ Clappers Bridge)	0.806	50	13.575	0.207	0.182	0.168	
28041 (Hamps @ Waterhouses)	0.863	34	26.313	0.219	0.288	0.699	
28058 (Henmore Brook @ Ashbourne)	0.866	13	8.838	0.188	-0.109	2.744	
27032 (Hebden Beck @ Hebden)	0.905	53	4.052	0.204	0.237	0.263	
24006 (Rookhope Burn @ Eastgate)	0.919	20	24.620	0.152	0.117	0.608	
36010 (Bumpstead Brook @ Broad Green)	0.922	52	7.395	0.382	0.181	2.680	
48004 (Warleggan @ Trengoffe)	0.924	50	9.957	0.257	0.258	0.173	
Total		503					
Weighted means				0.234	0.210		

Annex 3 River Dee LIFE - Chester: Flood Estimation Calculation Record

River Dee LIFE - Dee at Chester

Flood Estimation Calculation Record

60627686

December 2020

1 Flood estimation calculation record

Introduction

This document provides a record of the calculations and decisions made during flood estimation for the Morlas Brook. The information given here should enable the work to be reproduced in the future.

Contents

Approval

	Signature	Name and qualifications
Calculations prepared by:		Andrew Heath-Brown BSc (Hons), MSc, MCIWEM
Calculations checked by:		Helen Harfoot BSc (Hons) MSc MCIWEM CEnv CSci
Calculations approved by:		

Revision History

Revision	Revision Date	Details	Authorised	Name	Position

Abbreviations

AEP	Annual exceedance probability
AM	Annual maximum
AREA	Catchment area (km^2)
BFI	Base flow index
BFIHOST	Base flow index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
DPLBAR	Mean drainage path length (km)
DTM	Digital Terrain Model
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FPEXT	Floodplain extent
FSR	Flood Studies Report
HOST	Hydrology of soil types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks over threshold
QMED	Median annual flood (with return period ~2 years)
ReFH1	Revitalised Flood Hydrograph 1 method (2005)
ReFH2	Revitalised Flood Hydrograph 2 method (2013)
SAAR	Standard average annual rainfall (mm)
SPR	Standard percentage run-off
SPRHOST	Standard percentage run-off derived using the HOST soil classification
Tp (0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent
WINFAP	Windows Frequency Analysis Package – used for FEH statistical method

2 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections. The aim of the table is to enable quick and easy identification of the type of assessment undertaken. This should assist in identifying an appropriate reviewer and the ability to compare different studies more easily.

Catchment location	Chester (River Dee), Cheshire
Purpose of study and scope	Routine calculation of peak flood flow estimates, for use as part of a fish passage solution assessment.
Key catchment features	Some upstream storage with regulation scheme. Tidally influenced flood levels at Chester.
Flooding mechanisms	n/a
Gauged / ungauged	Gauged (not at site)
Final choice of method	FEH Statistical
Key limitations / uncertainties in results	Minor uncertainties (associated with data transfer from gauged site on same river)

2.1 Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval. Results tables in this document contain both return period and AEP titles; both rows can be retained, or the relevant row can be retained and the other removed, depending on the requirement of the study.

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

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

3 Method statement

3.1 Overview of requirements for flood estimates and hydraulic modelling

Item	Comments
Give an overview which includes: • purpose of study including a short discussion if there is existing hydrology reports and estimates, when they were done and why we are updating the hydrology (e.g. new data or superseded methods) • approximate number and type of flood estimates required • peak flows and/or hydrographs? • range of design event AEPs (%) • climate change allowances (ref. relevant guidance)	The study is an investigation into the possible implementation of fish passage at a number of weirs within the Dee catchment. Low flows are of greater significance. The scope however requires assessment for two flood flow events (likely 2-year and 100-year), hence the need for estimates. There are no known existing peak flow estimates. A range of estimates will be produced between 2 and 200-year return periods. However, the 1,000-year estimate will not be included. Peak flows and hydrographs are required, at a single location only.

3.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report. Include general catchment map and specific map of hydraulic model extents and inflow locations.	Peak flows and hydrographs have been estimated at the Chester weir (SJ 40800 65900). The main report provides a description of the catchment and the modelled reach, including details of where the flow estimate will be entered in the hydraulic model.
Previous Hydrology studies	None known

3.3 Source of flood peak data

Item	Comments
Was the NRFA Peak Flows dataset used? If so, which version? If not, why not? Record any changes made.	NRFA peak flows dataset, Version 9, released October 2020. This contains data up to the end of water year 2018-19, and provisional data for water year 2019/2020 at stations which set new records.

3.4 Gauging stations (flow or level)

Watercourse	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level)	Start of record and end if station closed
Dee	Manley Hall	067015	67015	1013.2	Rated	1970-
Dee	Ironbridge	067027	67027	1674.1	ADVP/ Cross-path	1994-
Dee	Chester Susp. Br.	067033	67033	1816.8	Ultrasonic	1994-

3.5 Data available at each flow gauging station

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling?	Data quality check needed?	Other comments on station and flow data quality
Manley Hall	1970-	No	Yes	Yes	No	
Ironbridge	1994-	n/a	No	No	n/a	No peak flow data Site is tidally influenced
Chester Susp. Br.	1994-	n/a	No	No	n/a	No peak flow data Site is tidally influenced

3.6 Other data available and how it has been obtained

Type of data	Data relevant to this study	Data available	Source of data	Details
Check flow gaugings (if planned rating review)	~~Yes~~ / No	Yes / No	n/a	n/a
Rating equations	~~Yes~~ / No	Yes / No	n/a	n/a
Historic flood data	~~Yes~~ / No	Yes / No	n/a	n/a
Flow or level data for events	~~Yes~~ / No	Yes / No	n/a	n/a
Results from previous studies	~~Yes~~ / No	Yes / No	n/a	n/a
Other information e.g. groundwater, tides etc	~~Yes~~ / No	Yes / No	n/a	n/a

3.7 Initial choice of approach

Item	Comment
Is FEH appropriate? If not, describe why and give details of the other methods to be used.	Yes.
Initial choice of method(s) and reasons.	FEH Statistical – catchment has no unusual features, and is within limits if applicability for FEH Statistical method. FEH Statistical is preferred over ReFH2 in most cases, unless demonstrated otherwise. ReFH2 unlikely to be appropriate for estimating peaks flows on catchments of this size (concept of catchment wide storm less applicable).
How will hydrograph shapes be derived if needed? E.g. ReFH1, ReFH2 or average hydrograph shape from gauge data	The ReFH2 method will be used to define hydrograph shape, based on the recommended duration. This is expected to be appropriate for the nature of this study. For detailed flood risk investigation, gauged hydrograph analysis with calibration would be require, although as gauge at Chester Suspension Bridge is tidally influenced, it will be difficult to separate the fluvial and tidal responses.
Will the catchment be split into sub-catchments? If so, how?	There will be a single inflow so the catchment will not be split into subcatchments. It is anticipated that the modelled reach will be short, and there are no significant tributaries in close proximity to the subject site with obvious differences to the overall contributing catchment.
Software to be used (with version numbers) (**delete as appropriate**)	FEH Web Service[1] / ~~WINFAP-FEH v3.0.003~~[2] / WINFAP 4[3] / ~~ReFH spreadsheet~~ / ReFH2.2 / ~~Flood Modeller Pro~~

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

4 Locations where flood estimates are required

4.1 Summary of subject sites

The table below lists the locations of subject sites.

Site Code	Type of Estimate (L – lumped catchment; S- Sub-catchment)	Watercourse	Site	Grid Reference	Area on FEH Web Service (km^2)	Revised area if altered
Dee_Chst	L	Dee	Weir	SJ 40800 65900	1801	n/a

Reasons for choosing above locations	Location of proposed fish pass

4.2 Important catchment descriptors at each subject site (original values from FEH Web Service)

Site Code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT2000	FPEXT
Dee_Chst	0.959	0.43	0.451	83.53	112.8	1110	38.91	0.0169	0.0819

[2] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.
[3] WINFAP 4 © Wallingford HydroSolutions Limited 2016.

4.3 Checking catchment descriptors

Item	Comment
Record how catchment boundary was checked • Describe any changes • Refer to maps if required	The catchment boundary has been checked against open source OS Maps, and is considered to be sufficiently accurate.
Record how other catchment descriptors were checked, especially soils • Describe any changes • Include a before and after table if required	HOST based catchment descriptors checked against BGS online soil mapping; found to be realistic.
Source of URBEXT / URBAN	FEH Web Service
Method for updating URBEXT / URBAN • Refer to WINFAP v4 Urban Adjustment procedures / guidance • CPRE formula from FEH Volume 4 / CPRE formula from 2006 CEH report on URBEXT2000[4]	As per WINFAF v4 methods, using URBEXT2000

[4] http://sciencesearch.defra.gov.uk/Document.aspx?Document=FD1919_5228_TRP.pdf#page=35

5 Statistical method

5.1 Application of Statistical method

What is the purpose of applying this method?	Comment
Summarise reasons specific to study, for example lumped estimates at key locations for purpose of checking modelled peak flows.	Lumped catchment estimate at site of interest for localised flood modelling of design event flows.

5.2 Overview of QMED method

What method of QMED estimation was used?	Comments
State method/s used to estimate QMED in study and why, for example gauged data, donor transfer, multiple donor transfer, flow variability, bankfull width or user defined.	Single gauge donor transfer from same watercourse, located upstream. This donor gauge is not the closest to the site, but is the nearest suitable gauge for QMED donor adjustment, and has similar catchment descriptors in most respects, except Area and FPEXT (and arguably SAAR and PROPWET). The catchment centroid based distance moderation factor was excluded from the data transfer process, as per the decision on other sites on the Dee (see Flood estimation calc record for Horseshoe Falls, Llangollen and Erbistock).

Summary of QMED estimates at each site:

Site code	QMED rural (from CDs) (m^3s^{-1})	Final method	Final estimate of QMED (m^3s^{-1})
Dee_Chst	395.9	DT (no distance moderation)	268.0

Note: Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

5.3 Search for donor sites for QMED

Comment on potential donor sites based on the above sections	Comments
• Number of potential donor sites available • Distances from subject site • Similarities in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data	There is only one donor suitable to the subject site that is both geographically close and comparable in all catchment descriptors, including catchment area. This site is also appropriate for estimating QMED via data transfer.

5.4 Uncertainty in QMED

The reduction in uncertainty as a result of applying data transfer is modest, with the 68% confidence interval for QMED on a rural catchment narrowing from:

- 0.69-1.45 times the estimate with no donor

to

- 0.70-1.42 times with one donor

and

- 0.71-1.40 times with six donors.

These figures are taken from Technical Guidance 12_17. Despite this relatively small effect on the degree of confidence, the estimate of QMED can change markedly as the result of some data transfers.

This study has adjusted the 'as rural' catchment descriptor QMED estimate, using one donor. The uncertainty, based on the 68% and 95% confidence intervals, can be described using the Factorial Standard Errors associated with **one donor**, for the 2-year (QMED) event, as given in Technical Guidance 12_17.

Site code	QMED (m^3s^{-1})	Bound	No donor		One donor		Six donors	
			FSE (68% CI)	QMED	FSE (68% CI)	QMED	FSE (68% CI)	QMED
Dee_Chst	268.0	Upper			0.70	187.6		
		Lower			1.42	380.6		

Site code	QMED (m^3s^{-1})	Bound	No donor		One donor		Six donors	
			FSE (95% CI)	QMED	FSE (95% CI)	QMED	FSE (95% CI)	QMED
Dee_Chst	268.0	Upper			0.50	134.0		
		Lower			2.02	541.4		

5.5 Derivation of pooling groups

Pooling groups were created within WINFAP v4 for each of the subject sites. An URBEXT2000 threshold of 0.3 was used to create the pooling groups in order to make maximum use of local data. The Heterogeneity statistic (H2) for the pooling groups were assessed; this provides an indication of whether a review of the pooling group is required (not required, optional, desirable or essential). The similarity of the subject site against stations within the pooling group is assessed by the Similarity Distance Measure (SDM) and is a function of Area, SAAR, FARL and FPEXT. However, it is good practice to review the pooling group to check other parameters e.g. BFIHOST and the history of the gauge, gauge record and rating quality on the NRFA website (https://nrfa.ceh.ac.uk/data/search).

As per the Environment Agency guidelines, modifications to the pooling group tend to have a relatively minor effect on the final design flow (compared with, for example, the selection of donor sites for QMED). Science Report SC050050[5] indicates that apart from the first four or five stations within a pooling group (i.e. lowest SDM), the record length at a station will only have a modest effect on its weight within the pooling group (unless the record is very short). The review of the pooling group has therefore focused on the first five stations within each pooling group, extending further where required to include stations that have moved up position following removal of others, gauges with a short record, and catchments which have extreme catchment descriptor values in comparison to the subject sites.

The table below summarises the pooling groups used in this study and provided in Annex A. Annex A also notes the reasons for removing catchments from the initial pooling group and which stations were added in to the pooling group to ensure that sufficient years of data (>500) were included in the final group.

Name of Group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons. Include any sites that were investigated but retained in the group	Weighted average L-moments (L-CV and L-skew before urban adjustment)
Dee_Chst_PG_Initial_URB	Dee_Chst	No	- No changes following review	L-CV = 0.161 L-Skew = 0.144
			-	

The table below details the H2 score and requirement for pooling group review for in the initial and final pooling groups for each site

Catchment	Initial Pooling Group H2 value	Recommendation for Pooling Group Review	Final Pooling Group H2 value	Recommendation for Final Pooling Group Review
Dee (Chester)	1.99	Optional	1.99	Optional

5.6 Derivation of flood growth curves at subject sites

Site Code	Method (SS, P, ESS, FH)	If P, ESS or FH, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustment	Growth Curve Factor for 100 year return period
Dee_Chst	P	Dee_Chst_PG_Initial_URB	GEV (acceptable and best fit)	Adjusted for urbanisation using WINFAPv4	Location = 0.910 Scale = 0.248 Shape = 0.037	1.959

Growth Curve Factors for the following return periods for GL and GEV distributions for DeeHFPooling Group

Distribution	2	5	10	20	30	50	75	100	200
GEV	1.000	1.272	1.445	1.608	1.699	1.811	1.898	1.959	2.103

5.7 Flood estimates from the statistical method

QMED estimated using donor transfer method and adjusted using UAF at site location.

	Flood Peak (m^3s^{-1}) for the following return periods								
Site Code	2	5	10	20	30	50	75	100	200
Dee_Chst	268.0	340.9	387.3	430.9	455.3	485.3	508.7	525.0	563.6

6 Revitalised flood hydrograph (ReFH2) method

6.1 Application of ReFH2 model

What is the purpose of applying this method?	Comment
Summarise reasons specific to study, for example: lumped estimates at key locations for the purpose of checking modelled peak flow estimates, distributed approach to apply inflows to a hydraulic model, deriving hydrograph shapes only, extending the flood frequency curve out to extreme events (long return periods).	Lumped estimate at site, to compare against FEH Statistical estimate and to provide an inflow hydrograph to any local hydraulic modelling.

6.2 Parameters for ReFH2 model

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

Site Code	Details of Method OPT: Optimisation BR: base flow recession fitting CD: catchment descriptors DT: Data Transfer	Tp_{rural} (hours) Time to peak	Tp_{urban} (hours) Time to peak	C_{max} (mm) Maximum storage capacity	Pr_{imp} (% runoff for impermeable surfaces)	BL (hours) Base flow lag	BR Base flow recharge
Dee_Chst	CD	10.07		342.28		73.93	1.91 (2-year)

Brief description of any flood event analysis undertaken: Provide further details here or in a project report	n/a

6.3 Design events for ReFH2 method: Lumped catchments

Site Code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Source of design rainfall statistic (FEH13 or FEH99)
Dee_Chst	Rural	Winter	30	FEH13

6.4 Flood estimates from the ReFH2 method

- Please indicate whether you have used urban or rural results
- We recommend that urban results are used regardless of the extent of urbanisation at the subject sites to avoid discontinuity when URBEXT reaches a given threshold.

As per the Technical Guidance Document: ReFH 2.2, the urban results are reported in the table below. These results take account of the urban extent within the catchment based on URBEXT2000 and are considered representative of existing conditions.

	Flood Peak (m^3s^{-1}) for the following return periods								
Site Code	2	5	10	20	30	50	75	100	200
Dee_Chst	341.6	435.5	508.0	586.0	634.7	699.3	753.0	792.5	891.9

7 Discussion and summary of results

7.1 Comparison of results from different methods

This table compares peak flows from the ReFH2 method with those from the FEH Statistical method (donor adjusted inclusive of urbanisation) at each site for two key return periods. This illustrates that flow estimates from the FEH statistical method are approximately 10% less than those derived using ReFH2.

	Return period 2 years (50% AEP)			Return Period 100 years (1% AEP)		
Site Code	Statistical	ReFH2	Ratio (ReFH2/Statistical)	Statistical	ReFH2	Ratio (ReFH2/Statistical)
Dee_Chst	268.0	341.6	1.27	525.0	792.5	1.50

7.2 Final choice of method

Choice of method and reason Include reference to type of study, nature of catchment and type of data available	FEH Statistical – preferred approach of NRW, and the catchment and associated sites are applicable with the approaches. For the subject sites considered in this calculation, the method selected uses gauged data on the Dee as part of the flow estimation process, from a site that is geographically close and hydrologically similar. This also allows consideration of some impact of the Dee regulation scheme.
How will the flows be applied to a hydraulic model?	Direct inflow (single boundary)

7.3 Assumptions, limitations, and uncertainty

List the main assumptions made specific to the study	Assumed that NRFA classification of Manley Hall subject site as suitable for QMED is appropriate, and that the observed data from this gauge is accurate.
Discuss any particular limitations For example applying methods outside the range of catchment types or return periods	Although the site is gauged (Chester Suspension Bridge), there are no peak flow records at the gauge (as per NRFA). This is because the gauge is tidally influenced.

for which they were developed	The use of an upstream gauge (Manley Hall) provides some resilience against that limitation.
Give what information you can on uncertainty in the design peak flows or in the methodology For example using the methods detailed in 'Making better use of local data in flood frequency estimation' - Science Report SC130009/R	Uncertainty has been quantified in respect of the QMED estimate, based on standard factorial standard errors associated with transferring data from a single donor site.
Comment on the suitability of the results for future studies For example at nearby locations or for different purposes	Suitable for studies requiring comparison of impacts, but not for flood defence schemes or mapping (more work required to investigate hydrology estimates).
Give any other comments on the study For example suggestions for additional work	Consider sensitivity testing flow in respect of QMED uncertainty. More importantly, more detailed flood related studies should investigate flood hydrograph shape and critical durations/ volumes from observed data, rather than relying on a scaled ReFH2 boundary.

7.4 Checks

Are the results consistent, for example at confluences?	n/a
What do the results imply regarding the return periods / frequency of floods during the period of record?	n/a
What is the 100-year (1% AEP) growth curve factor? Is this realistic? (The guidance suggests a typical range of 2.1 – 4.0)	1.959 – although this is a low growth curve factor, it is likely to be realistic since the pooling group is relatively homogeneous, the GEV was the preferred distribution, and the catchment includes some reservoir/ lake attenuation.
If 1000 year (0.1% AEP) flows have been derived, what is the range of ratios for the 1000-year (0.1% AEP) flow over the 100-year (1% AEP) flow?	n/a
What is the range of specific run-offs (l/s/ha) do the results equate to? Are there any inconsistencies?	1% AEP specific run-off is 2.91 l/s/ha. Value is sensible in comparison to Erbistock (see separate flood calc record) since this catchment is much larger (almost 3.5 times greater area).
How do the results compare with those of other studies? Explain the difference and conclude which results should be preferred	n/a
Are the results compatible with the longer-term flood history?	n/a
Describe any other checks on the results	n/a

7.5 Final results

The final peak flow results for use in the hydraulic model are provided in the table below. This includes the appropriate allowances for climate change.

	Flood peak ($m^3 s^{-1}$) for required return periods (in years)						
Site Code	2	10	100	100 +30%	100 +40%	100 + 85%	1000
Dee_Chst	268.0	387.3	525.0	682.5	n/a	n/a	n/a

7.6 Uncertainty bounds

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

The uncertainty bounds given below are based on the 68% confidence interval surrounding the QMED estimate only.

Site Code	Flood peak ($m^3 s^{-1}$) for required return periods (in years)							
	2		10		100		1000	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Dee_Chst	187.6	380.6	271.1	553.8	362.3	766.5	n/a	n/a

8 Annex A–WINFAP v4 Pooling Groups

8.1 Initial pooling group composition

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
84013 (Clyde @ Daldowie)	0.197	56	424.099	0.166	0.259	1.015	
54005 (Severn @ Montford)	0.238	68	305.952	0.138	0.045	0.873	
55002 (Wye @ Belmont)	0.257	112	392.897	0.114	0.101	1.711	
8010 (Spey @ Grantown)	0.304	67	233.616	0.175	0.096	1.699	
76007 (Eden @ Sheepmount)	0.367	53	615.615	0.200	0.237	0.967	
12002 (Dee @ Park)	0.403	33	556.402	0.155	0.032	0.944	
23001 (Tyne @ Bywell)	0.471	63	837.962	0.163	0.149	0.132	
21006 (Tweed @ Boleside)	0.498	58	395.378	0.190	0.227	0.659	
Total		510					
Weighted means				0.161	0.144		

8.2 Final pooling group composition

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	*Comments*
84013 (Clyde @ Daldowie)	0.197	56	424.099	0.166	0.259	1.015	
54005 (Severn @ Montford)	0.238	68	305.952	0.138	0.045	0.873	
55002 (Wye @ Belmont)	0.257	112	392.897	0.114	0.101	1.711	
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21006 (Tweed @ Boleside)	0.498	58	395.378	0.190	0.227	0.659	
Total		510					
Weighted means				0.161	0.144		