Castleconnell FRS – Hydraulics Report

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Final Report

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Revision History

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Contract

This report describes work commissioned by Limerick City & County Council as part of the Castleconnell Flood Relief Scheme. Daniel Iordache of JBA Consulting carried out this work.

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Purpose

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Abbreviations

Abbreviation	Meaning
AEP	Annual Exceedance Probability
AFA	Area for Further Assessment
DTM	Digital Terrain Model
EPA	Environment Protection Agency
FM	Flood Modeller
FRS	Flood Relief Scheme
HEFS	High End Future Scenario
LiDAR	Light Detection and Ranging
MBE	Mass Balance Error
MRFS	Mid Range Future Scenario
mOD	Metres above Ordnance Datum
OPW	Office of Public Works
OSI	Ordnance Survey Ireland
TUFLOW	Two-Dimensional Unsteady Flow

Glossary

Terminology	Meaning
Operational conditions	Set of conditions for operating Ardnacrusha Power Station
Standard operational conditions	Ardnacrusha Power Station functions as standard, namely with the Parteen spillway and four turbines in operation, 345m³/s being the operational maximum intake flow to the turbines from Parteen Basin during flood conditions due to hydraulic constraints in the canal capacity.
Limitations in operational conditions	Two turbines or the spillway and one turbine are not in operation at the Ardnacrusha Power Station, 258m³/s being regulated to the turbine to suit the operational capacity at the station.
"504" Event	AEP Event on Shannon River when Ardnacrusha Power Station functions in standard operational conditions (345m³/s inflow to the turbines), resulting a peak flow of 504m³/s downstream of Parteen Weir during the 1% AEP event. This is the scale of flow experienced in the 2009 event.
Baseline Design Event	Event on Shannon River downstream of Parteen Weir when Ardnacrusha Power Station functions with limitations in operational conditions (258 m³/s regulated from Parteen Basin), resulting a peak flow of 591m³/s downstream of Parteen Weir during the 1% AEP event

1 Introduction

The Hydraulics Report aims to provide technical details about the construction and schematisation of the hydraulic model of Castleconnell and the surrounding area used within the development of the Castleconnell Flood Relief Scheme (FRS).

1.1 Project aim

The overall purpose of the Castleconnell FRS project is to design and build flood defences that will protect properties and critical infrastructure in future flood events, with a standard of protection up to 1% AEP. Hydraulic modelling of the River Shannon downstream of the Parteen Weir was developed to assess design flood levels and potential defence options.

1.2 Study area overview

The River Shannon is the dominant source of flood flows at Castleconnell although it is heavily influenced by Parteen Weir and Lough Derg. The Shannon River is the natural outlet of Lough Derg, with the ESB regulating the flows over Parteen Weir. Other fluvial sources influencing the area are the Kilmastulla River, Black River, Cedarwood Stream and Stradbally Stream.

Over time, as a result of the modified flow regime, the Shannon River downstream of Parteen Weir has significantly changed geomorphic characteristics with the manmade development of river features which have further developed into semi-permanent features and islands with heavy vegetation growth. The riverbed is also regularly intersected by inline rock weirs creating a stepped profile through the reach at Castleconnell.

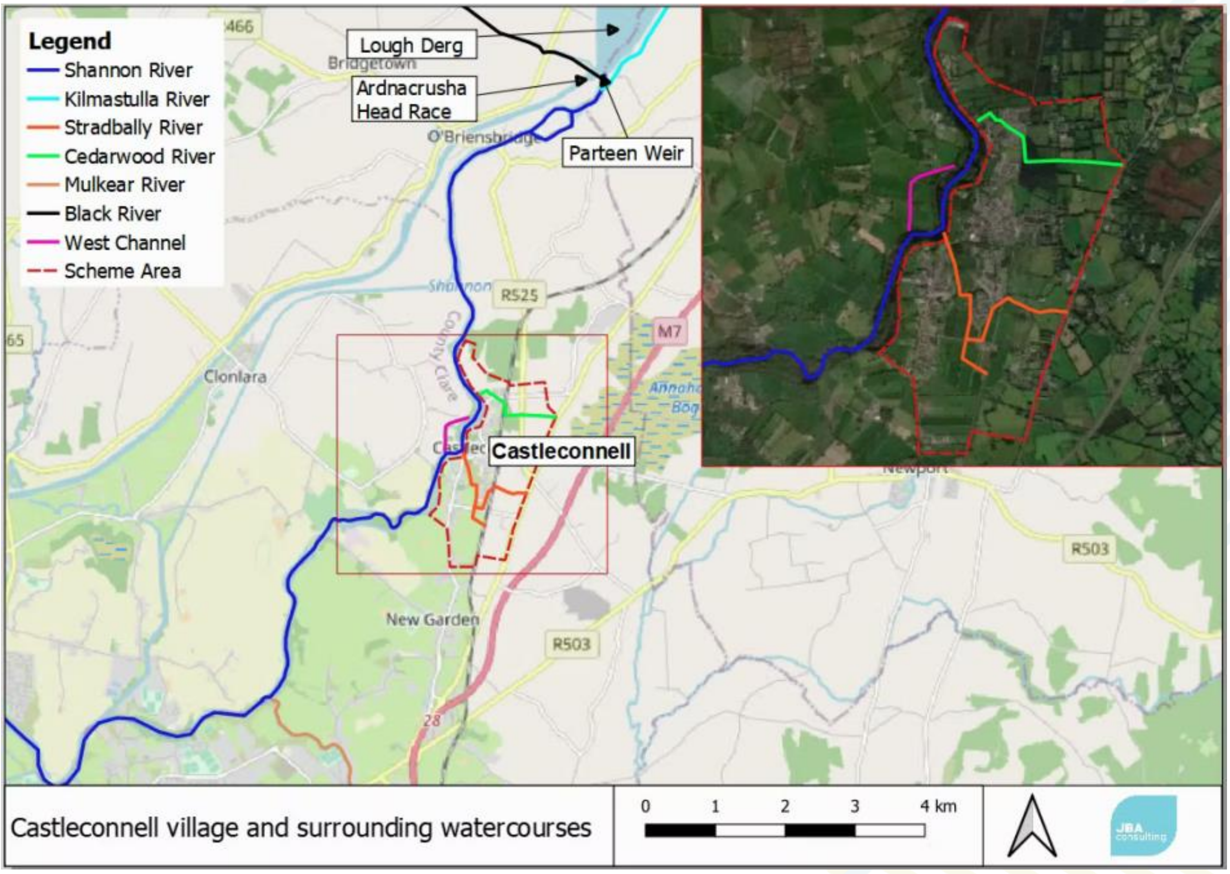


Figure 1-1: Castleconnell village and surrounding watercourses

2 Model development

2.1 Overview

The Shannon River channel through the study area is characterised by high hydraulic complexity, with many islands, pools and weirs present along the channel, influencing the hydraulic regime. A 1-dimensional (1D) model built by interpolating cross sections is not capable of capturing the hydraulic effects and spatial variation created by the in-channel features, therefore, this study commissioned additional topographical and river survey to provide sufficient detail to construct a 2-dimensional (2D) model able to directly represent the hydraulic behaviour of these features. Reaches upstream and downstream of the scheme area have retained representation of the channel within 1D to provide routing from Parteen Weir to the study area and to provide sufficient distance downstream of the study area to reduce uncertainty associated with tailwater conditions.

Two models were built for this study. The primary model represents the Shannon River from Parteen Weir to the Mulkear River confluence, whilst the second model represents the Cedarwood tributary in Castleconnell (refer to Appendix D – Cedarwood Stream).

The Cedarwood Stream at the northern end of Castleconnell is a significantly smaller watercourse than the Shannon River. CFRAM modelling has shown it is a source of flood risk to Castleconnell and there is also the potential for backwater flow from the Shannon River exceeding bank heights of the Cedarwood Stream.

Due to the significant change in scale between the Cedarwood Stream and the Shannon River, a separate 1D-2D FM-Tuflow model was built for Cedarwood Stream with a downstream water level – time boundary using the water levels recorded on the Shannon from the main model. In order to prioritize the completion of the main hydraulic model (Shannon River), the survey for Cedarwood Stream was undertaken later than for Shannon River, as such, the Cedarwood Stream discussion is included in Appendix D – Cedarwood Stream.

In the main Shannon River model, Cedarwood Stream is not included, with the flows being added directly into the 2D domain at its outlet to the Shannon River. The main Shannon model includes the Stradbally stream, modelled in 2D, with the culverts modelled in 1D as Estry elements. Table 2-1 provides a summary of general model details for the Shannon model.

Table 2-1: Hydraulic model summary – Shannon model

1D model	**Value**
Total 1D modelled length	10.29km (5.88km upstream and 4.41km downstream)
1D timestep	2 seconds (half the 2D 8m domain timestep)
Number of inflows	2 inflows (points)
Number of outflows	1 1D outflow (Normal depth boundary)
2D model	
Total model area	1.94 km^2
Model orientation	North-east to South-west
2D grid cell size	8m/4m multi-domain
2D timestep	4 seconds for the 8m domain/2 seconds for the 4m domain (half the grid cell size)
Number of inflows	4 2d inflows
Number of outflows	No 2d outflows
1D-2D model linkage	Via SX and CN points and lines

Coordinate reference system	TM65 (Irish National Grid)
Average model run time	11 hours for a 1000h simulation

2.2 Software

The model was developed using Flood Modeller and TUFLOW software packages creating a linked model with 1D and 2D components. The 1D model domain was modelled using Flood Modeller (FM) Pro v4.5, while the culverts on the Stradbally tributary have been modelled in Estry and linked to the 2D domain. The 2D domain has been modelled using TUFLOW Classic 2018-03-AE. These versions were the latest releases at the time of initial model build. The double precision versions of both software were used.

2.3 Stradbally Stream Representation

Stradbally Stream was modelled in 1D-2D in the previous CFRAM model, as a separate model. However, given the fact its floodplain is heavily dominated by Shannon River, conveyance within the Stradbally Stream is inconsequential when considered with the overall Stradbally floodplain storage volume from the Shannon River. As such, the Stradbally watercourse is included directly within the Shannon River model. The 2D inflow boundary is applied upstream of the railway line, as shown in Figure 2-3: 2D Model Schematisation.

2.4 Shannon River Model

2.4.1 1D-2D Model Extents and Schematisation

The full Castleconnell model is composed of 3 parts:

- 1D only upstream;
- 2D only in the study area (with 1D structures);
- 1D only downstream.

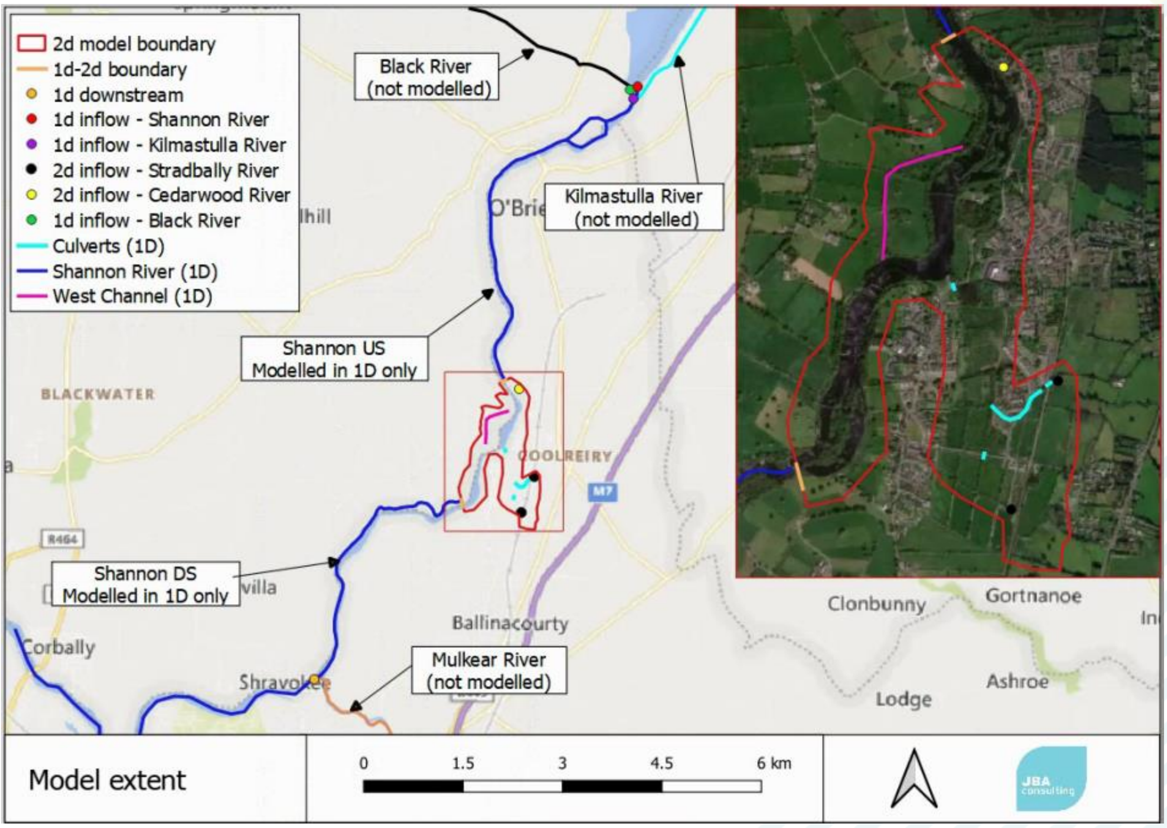


Figure 2-1: Model extent

1D only model upstream of the study area

The model represents the River Shannon and floodplains from just downstream of Parteen Weir to upstream of Castleconnell village. The modelled length of the watercourse is 5.88km. The Black River and Kilmastulla watercourses are not modelled, but the flows being added into the system at the confluence with Shannon.

1D only model downstream of the study area

The model represents the River Shannon and floodplains from downstream of Castleconnell to the confluence with Mulkear River. The modelled length of the watercourse is 4.41km. The Mulkear River does not have any effect in the area of Castleconnell, therefore its flows were not added into the system.

Both upstream and downstream of the study area, the 1D cross-sections are spaced approximatively every 100m. The extended cross-sections represent both the channel and the floodplains, with a length that varies between 300m and 1500m. There is one structure (bridge) upstream modelled as arch type unit. The Manning's Roughness applied varies from 0.035 to 0.045 for the riverbed and from 0.07 to 0.11 for the floodplains.

The 1D model schematization is presented in Figure 2-2.

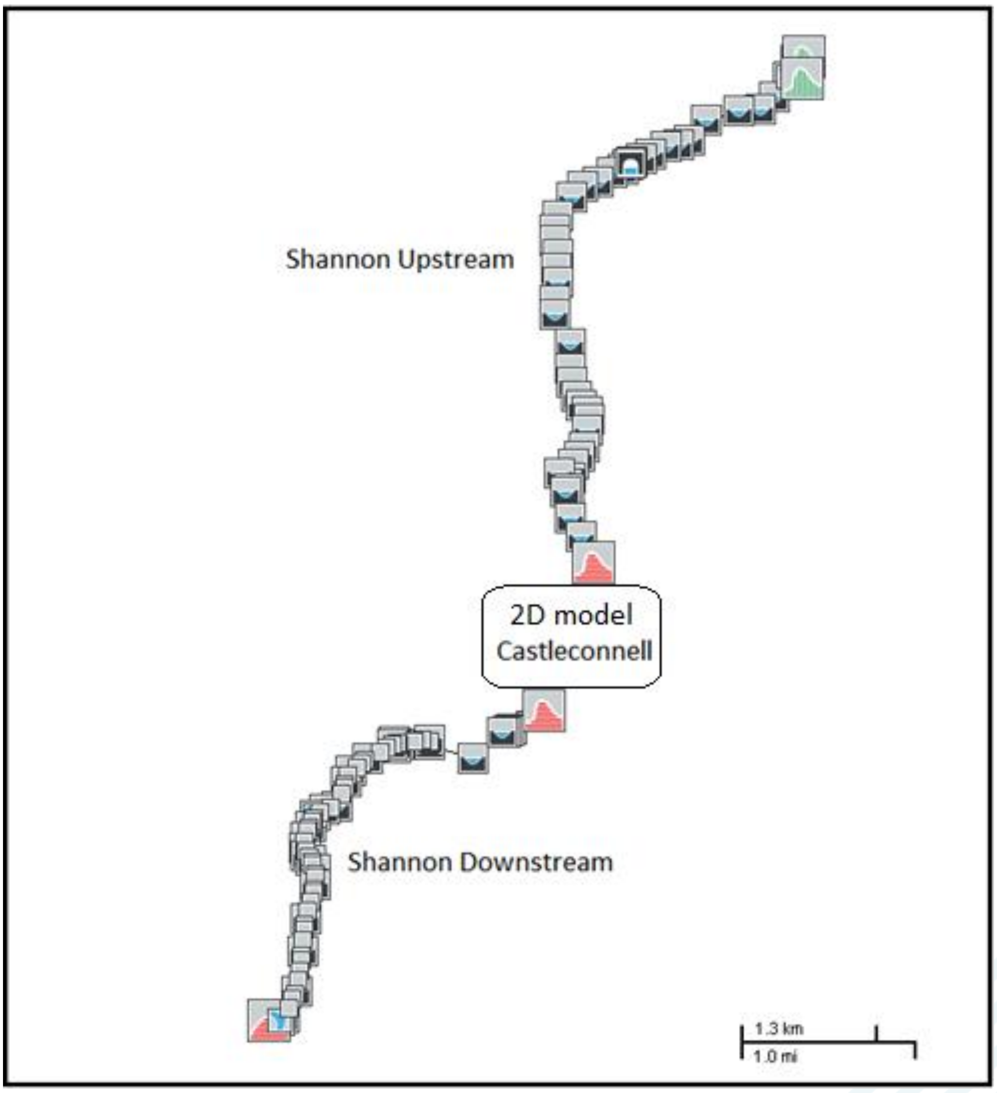


Figure 2-2: 1D Flood Modeller schematisation

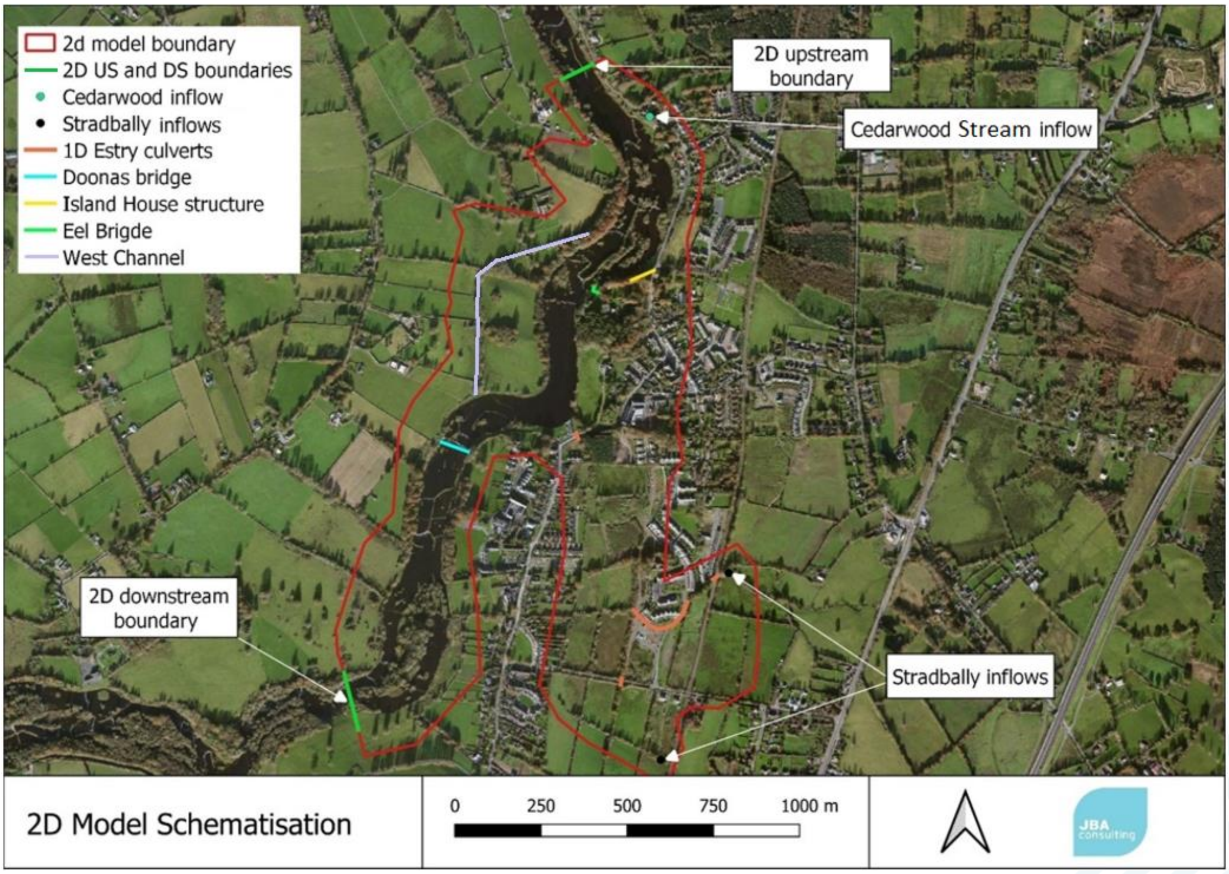


Figure 2-3: 2D Model Schematisation

Study area – 2D only

A 2D only model was built in the study area of Castleconnell village. The modelled length of Shannon is 2.24km and the total modelled area is 1.94km^2.

Stradbally is not modelled because its floodplain is heavily dominated by Shannon's backflows, while their peak flows are significantly different: the peak flow of Stradbally represents less than one percent of the peak flow of Shannon in the 1% AEP event.

Figure 2-4 presents the Stradbally channel in comparison with the floodplain and the water level resulted from Shannon backflows in the 1% AEP event.

The flows are added into the 2D domain as inflow points and the 4 culverts along the watercourse are modelled as 1D Estry elements.

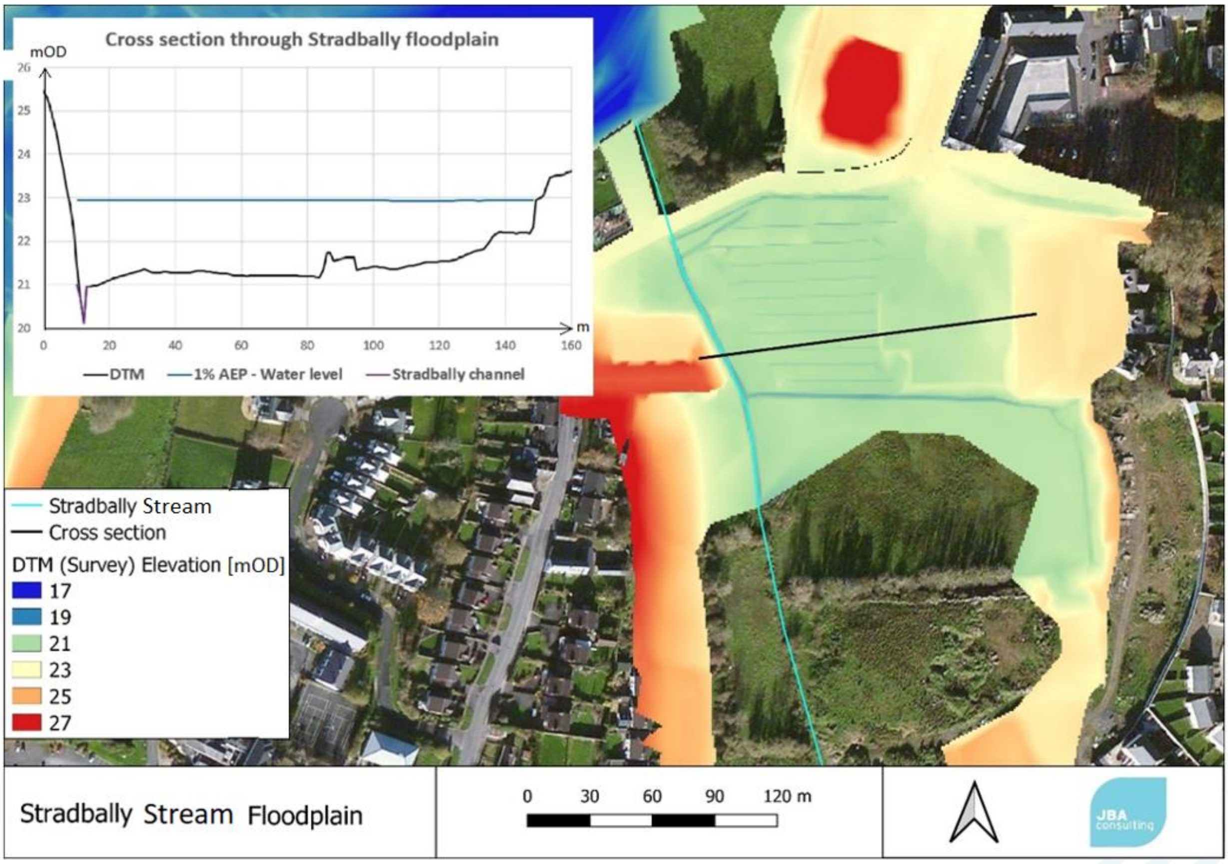


Figure 2-4: Stradbally Stream Floodplain

2.4.2 Boundaries

2.4.2.1 Inflow boundaries

The hydrographs and flows calculated for this study were applied both in the 1D and the 2D components of the model. Refer to Appendix C for all the flow values used in the model.

Shannon flows are applied at the upstream end of the model, immediately downstream of the Parteen Weir. The flows are derived from the Parteen Weir AMAX single site LN2 distribution growth curve and the hydrograph shape is derived from the 2015 hydrograph shape of Old Shannon flows.

Black River and Kilmastulla flows are applied at their confluence with Shannon, approximately 100m and 200m downstream of Parteen Weir. The Black River hydrograph was calculated using standard FSU methods, while the Kilmastulla hydrograph is derived from a routing model from the Coole Gauge to the confluence with the Shannon. It is unlikely that a Kilmastulla or Black River flood response could occur at the same time as when the Shannon is in high flow conditions. The probability of such an event would be less than the Shannon flow probability (e.g. a 1% AEP Kilmastulla or a 1% AEP Black River at the same time as a Shannon 1% AEP event would in total be less likely than the 1% AEP event). The joint probability of flow events on the Kilmastulla and Black River with the Shannon are assessed in the Hydrology Report. For the 1% AEP event, a 5% AEP Kilmastulla and Black River flow hydrograph was applied to the Shannon River flows, the peaks of Kilmastulla and Black River being applied at the peak of Shannon River.

The magnitude of the Shannon River hydrograph is significantly higher compared to the Kilmastulla and Black River hydrographs, in terms of both peak flows and flood duration, as shown in Figure 2-5. The Kilmastulla and Black River hydrographs are displayed in Figure 2-6 and the location of the inflow boundaries (flow-time) within the model are presented in Figure 2-9.

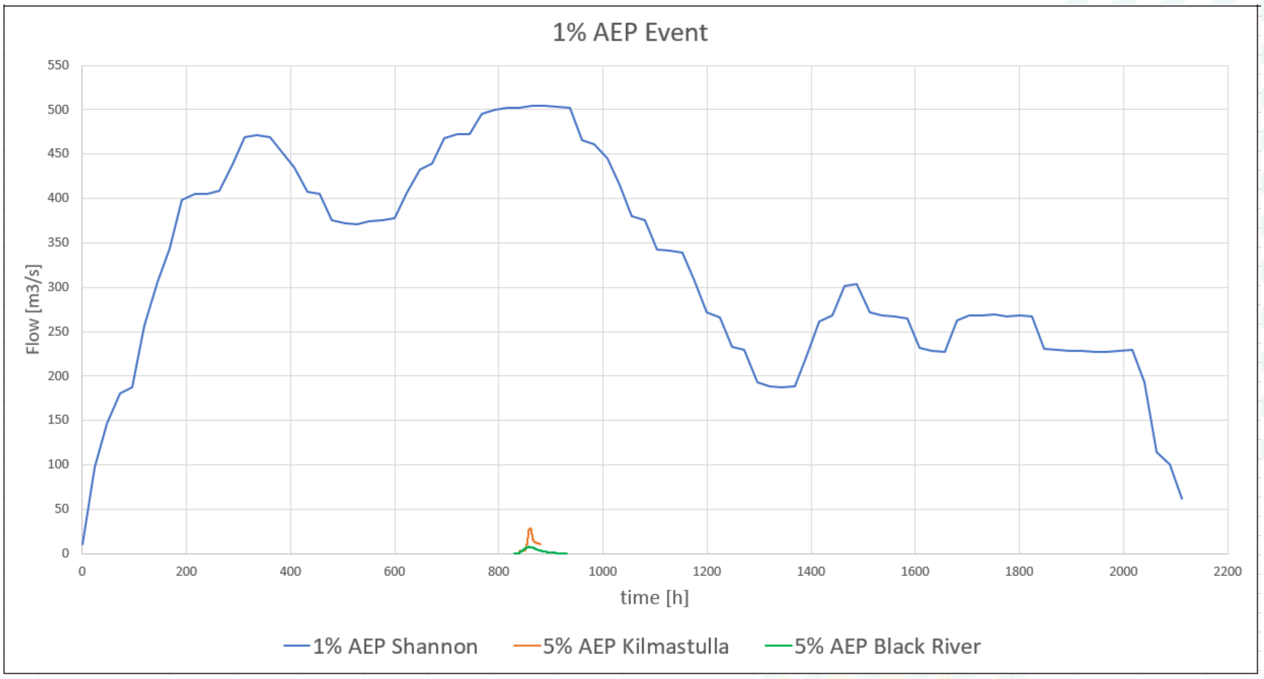


Figure 2-5: Old Shannon downstream of Parteen Weir 1% AEP (after headrace flow of 345 m³/s to Ardnacrusha – "504 event")

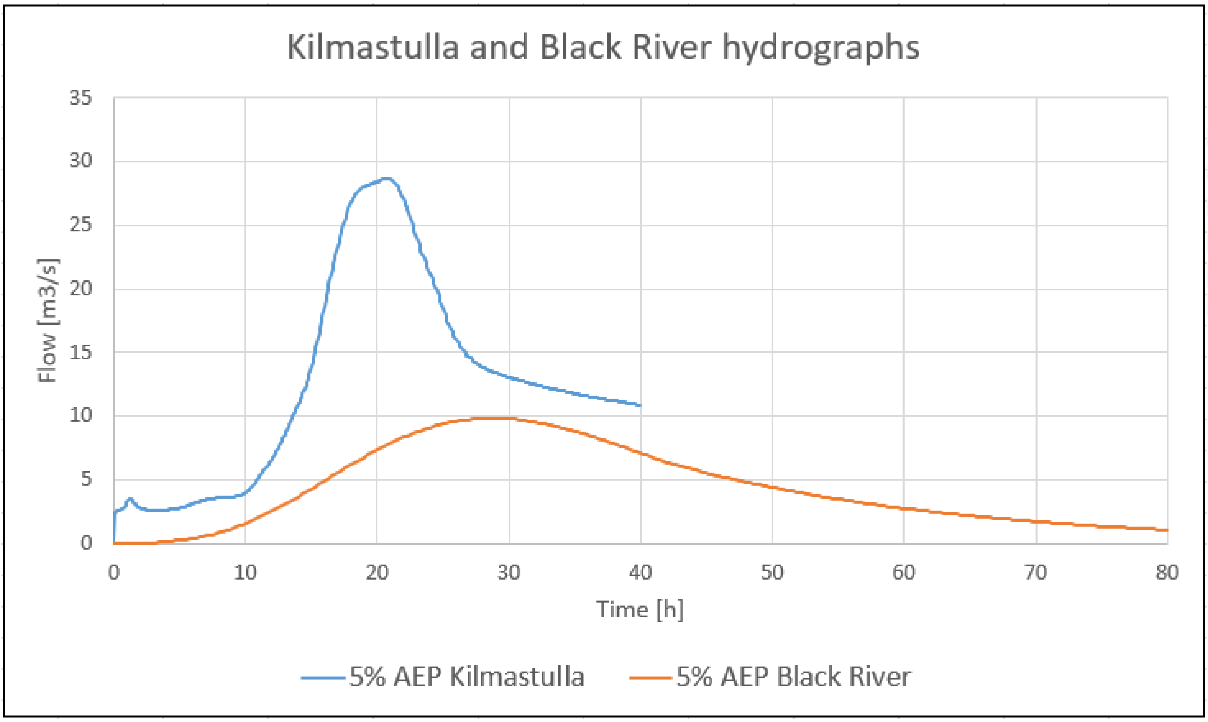


Figure 2-6: Kilmastulla and Black River 5% AEP

The Cedarwood and Stradbally streams have significantly smaller catchments than the River Shannon, therefore the peak flows and the flood durations are also considerably smaller, as presented in Table 2-2.

Flood events in the Shannon can last for prolonged periods, so it is possible that during a flood event on the Shannon when the water levels are high, a flood event could also occur on the smaller tributaries. Therefore, it is an acceptable approach to have the Cedarwood and Stradbally peaking at the same time as the Shannon River.

As presented in the Hydrology Report, the Cedarwood Streams flows were calculated at four HEP locations (25_3823_6a, 25_3823_6b, 25_3823_6c, 25_3823_6d). The combined flow was added directly into the 2D domain at its outlet to the Shannon River (HEP 25_3823_6).

Table 2-2: Hydrograph values

Watercourse	HEP reference	Peak flow [m³/s]	Duration [hours]
Old Shannon downstream of Parteen Weir (1% AEP)	25_3886_1	504.4 (with 345 m³/s headrace flow to Ardnacrusha – "504 event")	2112
Kilmastulla (5% AEP)	25_3881_9	28.71	40
Black River (5% AEP)	25_3838_4	9.82	80
Cedarwood (1% AEP)	25_3823_6	1.07	13
Stradbally East	25_3823_8d	1.31	15
Stradbally South	25_3823_8a	0.63	15

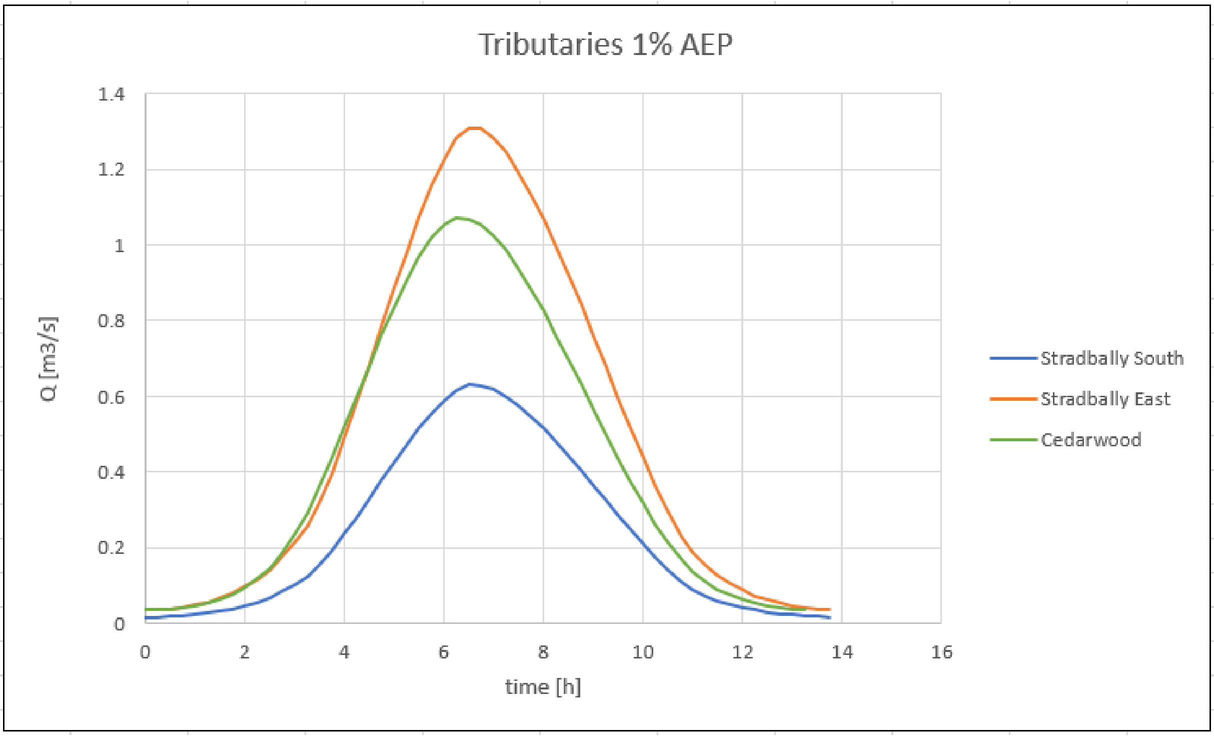


Figure 2-7: Cedarwood and Stradbally Streams 1% AEP

2.4.2.2 1D only model - Downstream boundary

The 1D model continues 4.4km downstream of the study area, sufficiently for the water levels in the 2D domain to not be influenced by the downstream boundary of the 1D model, as seen in Figure 2-8. The 1D downstream boundary is a Normal Depth (approximately 0.5% slope).

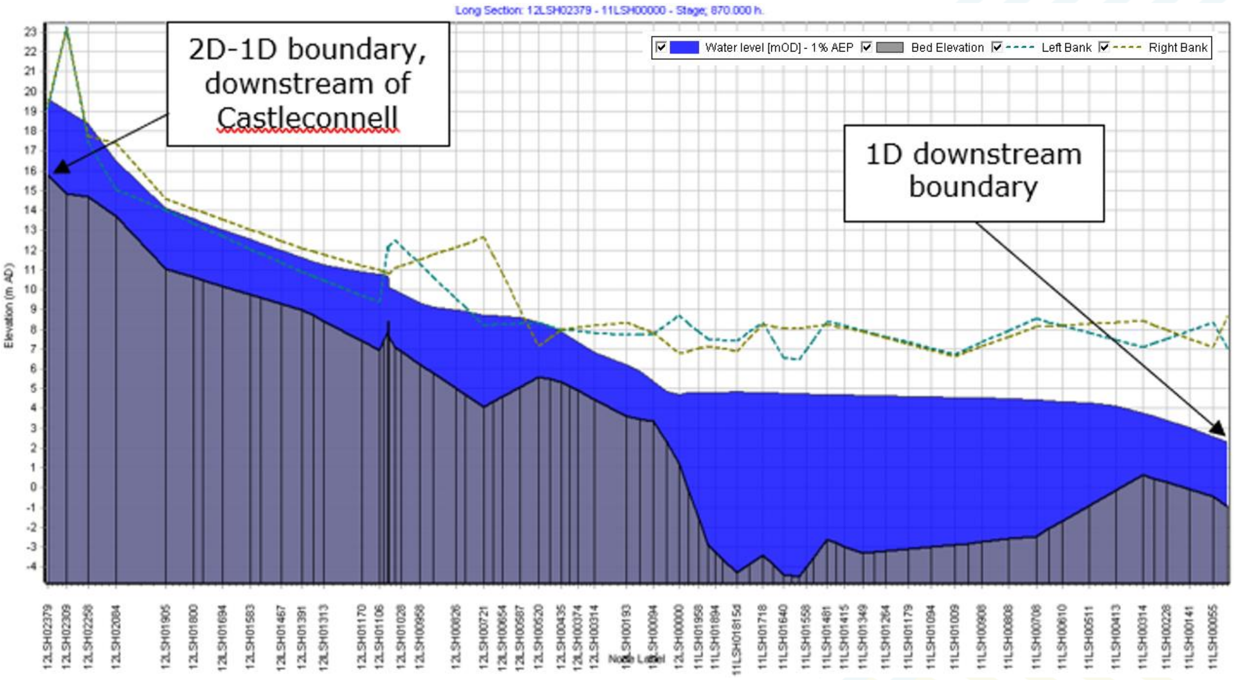


Figure 2-8: 1D model downstream of Castleconnell– Longitudinal Profile (1% AEP)

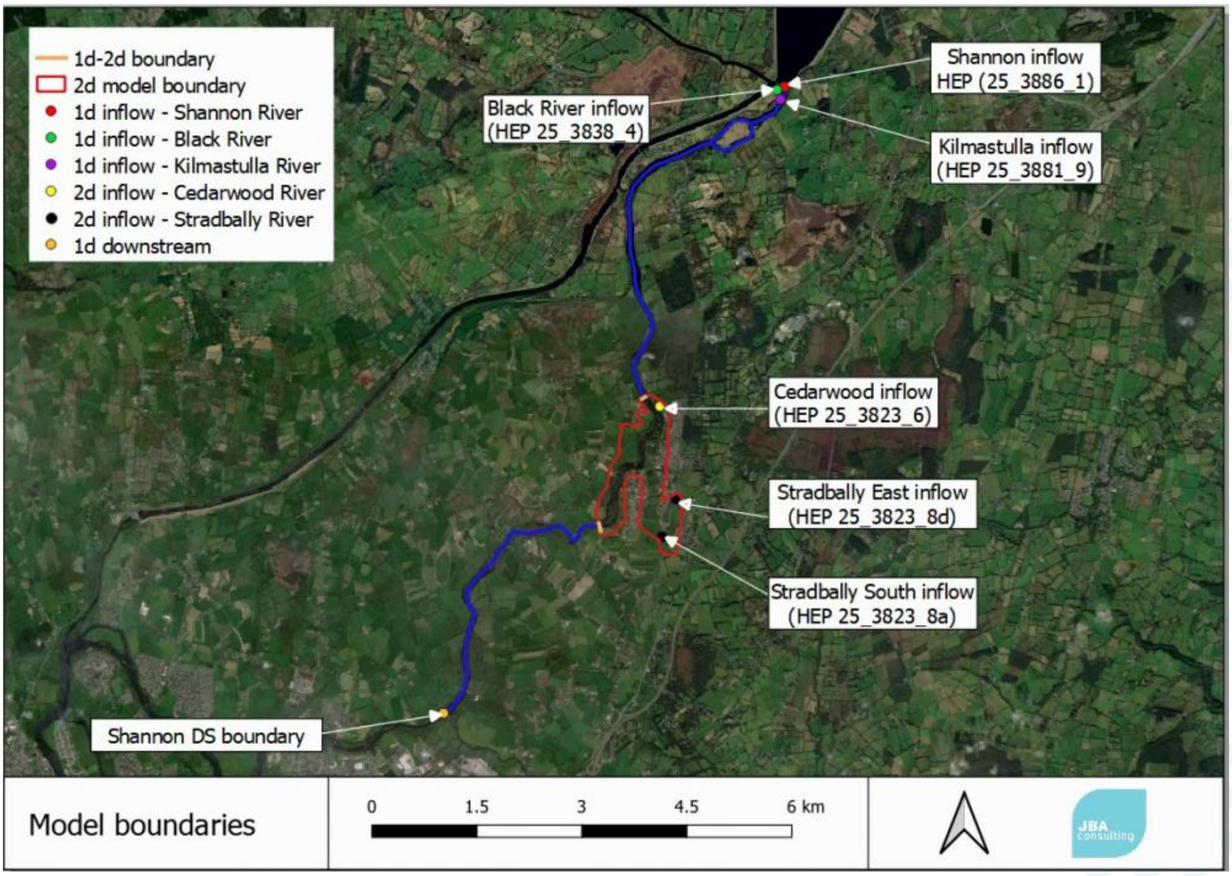


Figure 2-9: Model boundaries

2.4.2.3 1D - 2D boundaries

The 1D and 2D models have been linked using connection lines. Water level in the 2d boundary cells is determined based on the flow from the 1D node and, conversely, water level in the 1D node is determined based on the average water level along the 2d boundary cells. Flow is proportioned via depth because multiple cells are connected to a single 1D node. The boundary extends on the full width of inundation.

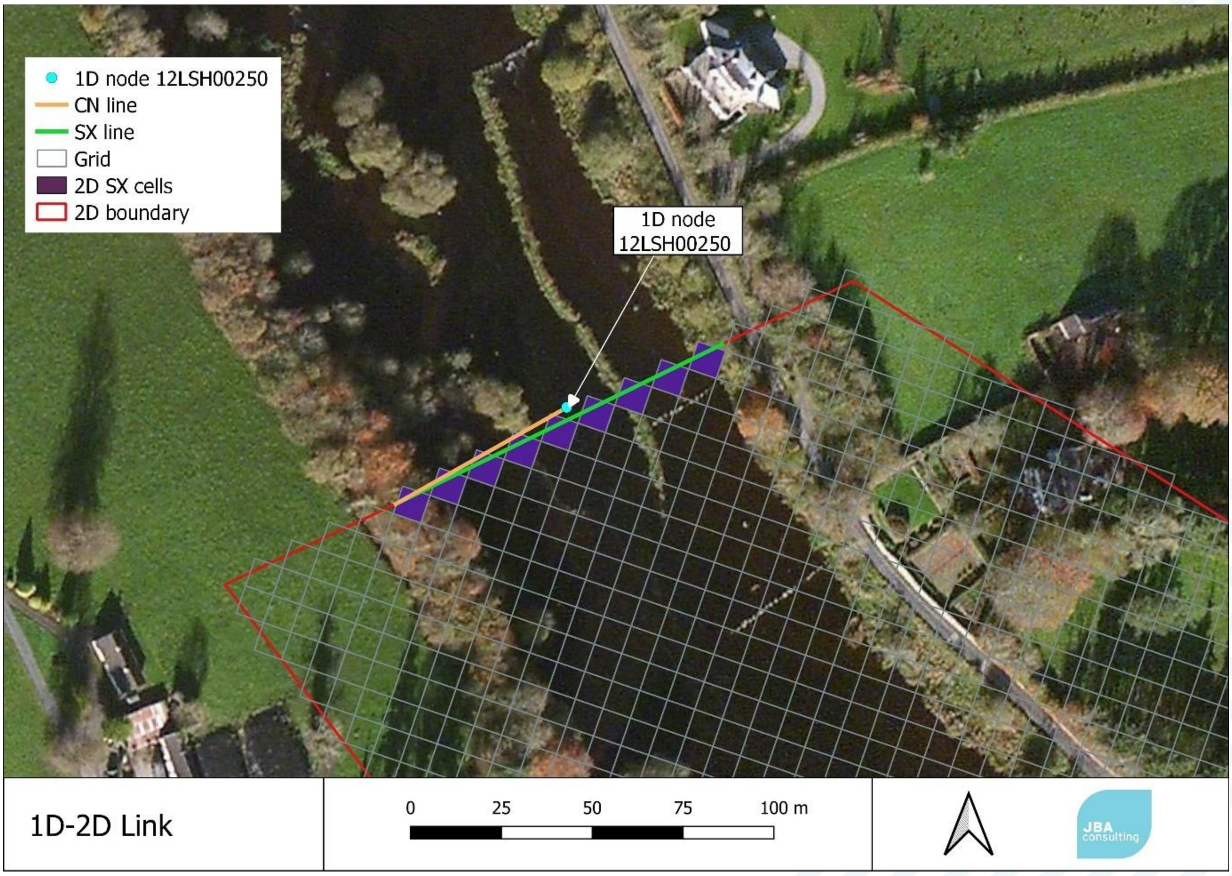


Figure 2-10: Linking between 1D and 2D models

The flow hydrographs modelled at the upstream and downstream 1D-2D boundaries are presented in Figure 2-11 and Figure 2-12, showing the flows are consistent at the boundaries between the 1D and 2D domains.

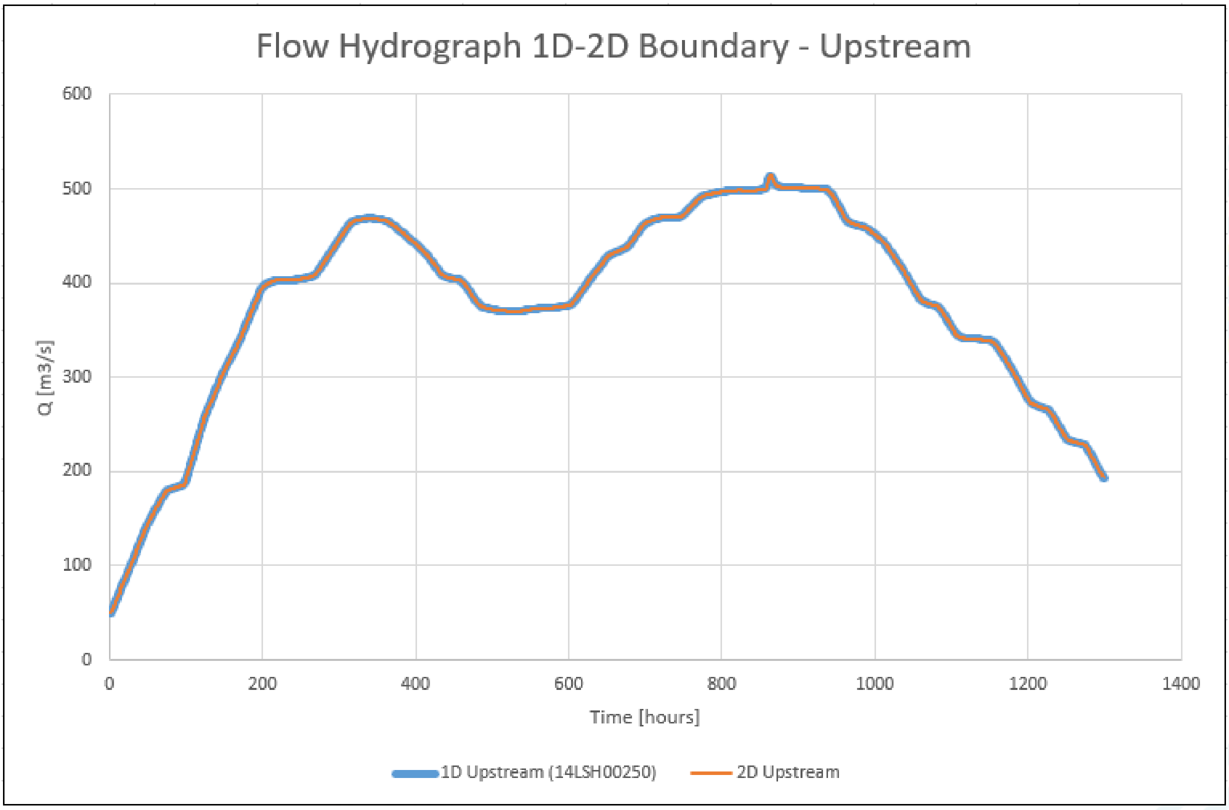


Figure 2-11: Flow Hydrographs Modelled at the Upstream 1D-2D Boundary (1% AEP with 345 m³/s headrace flow – "504 event")

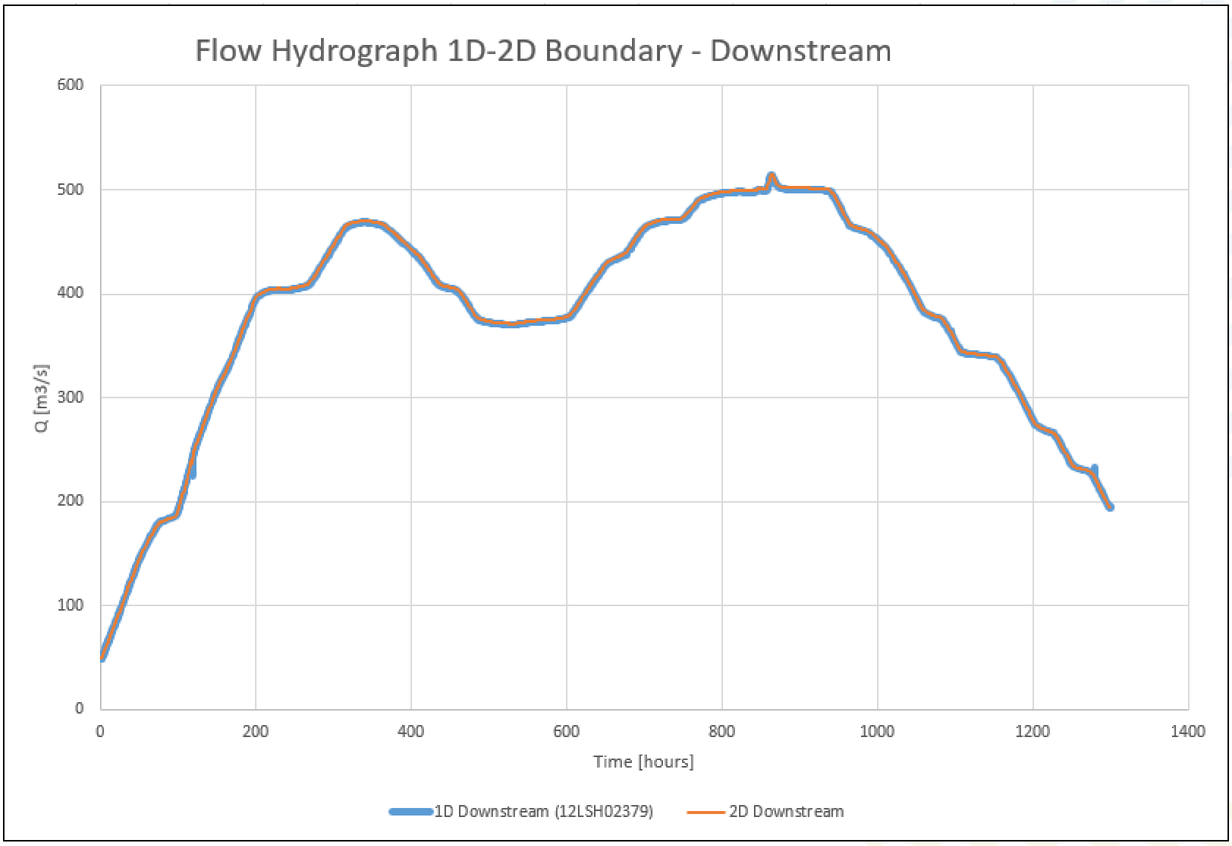


Figure 2-12: Flow Hydrographs Modelled at the Downstream 1D-2D Boundary (1% AEP with 345 m³/s diversion - "504 event")

2.4.3 Topography and DTM

The model was built using three topographic data sets:

- Cross sections of River Shannon (CFRAM survey), approximative 100 m spacing;
- 2m resolution LiDAR, stated vertical accuracy of 200 mm;
- DTM derived from survey points collected in February 2020.
- The topographic data distribution within the study area is presented in the figure below.

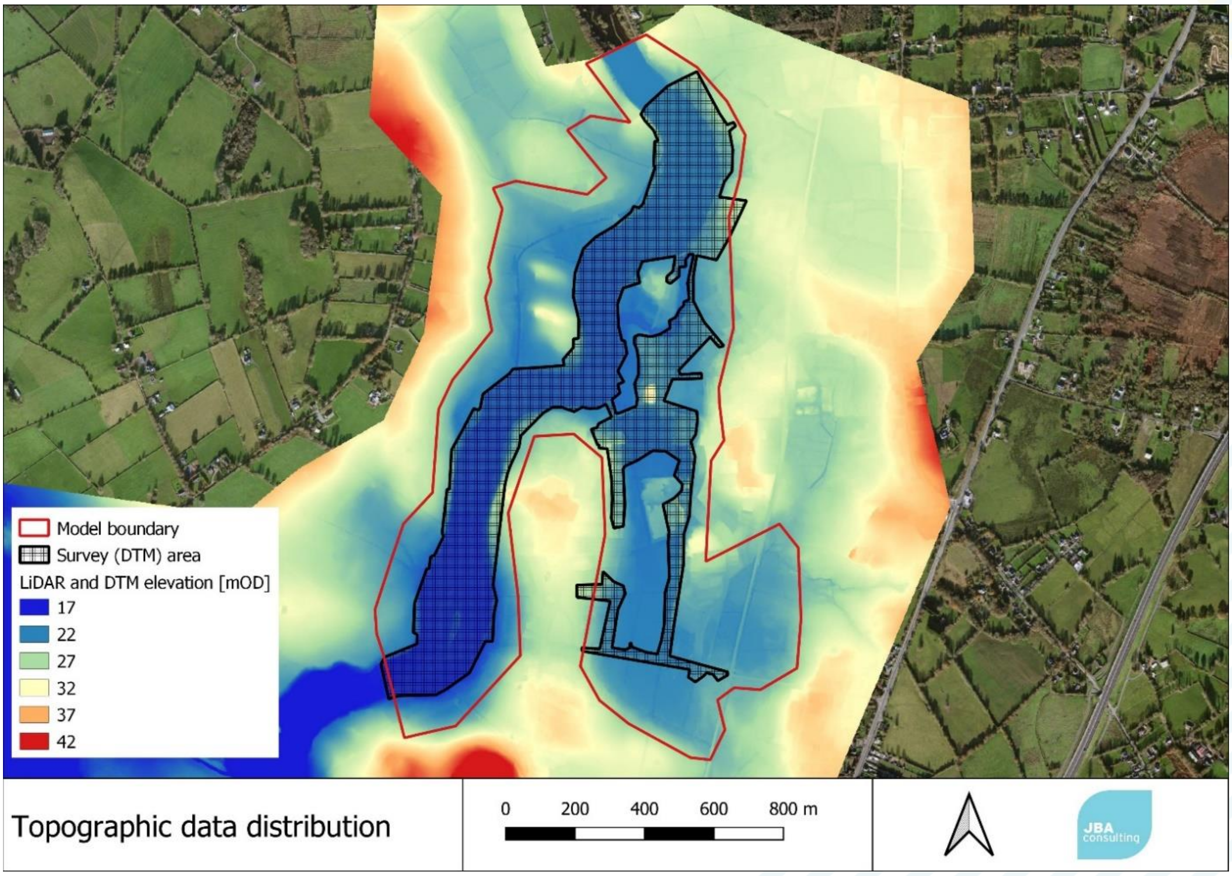


Figure 2-13: Topographic data distribution in the model

2.4.3.1 CFRAM cross sections

The cross sections of River Shannon exist from a previous model (CFRAM), that was divided as follows:

- 1D-2D in Castleconnell;
- 1D only upstream and downstream of Castleconnell.

The FRS model kept the 1D existing models, and, instead of a 1D-2D model, a 2D only model was built in the study area, considering the complexity of River Shannon in Castleconnell. Therefore, 2D datasets were used for building the by-dimensional model.

2.4.3.2 Lidar

The Lidar data used in the model has a 2m resolution and a stated vertical accuracy of +/- 200mm. The Lidar was used to represent the floodplain in the areas not covered by topographic survey (see Figure 2-13).

2.4.3.3 Survey data

The survey area extent is defined by the Shannon channel (including the top of banks) and part of the Castleconnell village – roads and areas exposed to flooding.

At the upstream boundary of the survey area, on the left bank, there are properties possible impacted by inundation. In order to avoid creating the 1D-2D boundary at this location, the 2D model has been extended 160m upstream, as seen in Figure 2-14. This area was modelled by interpolating 2 cross sections.

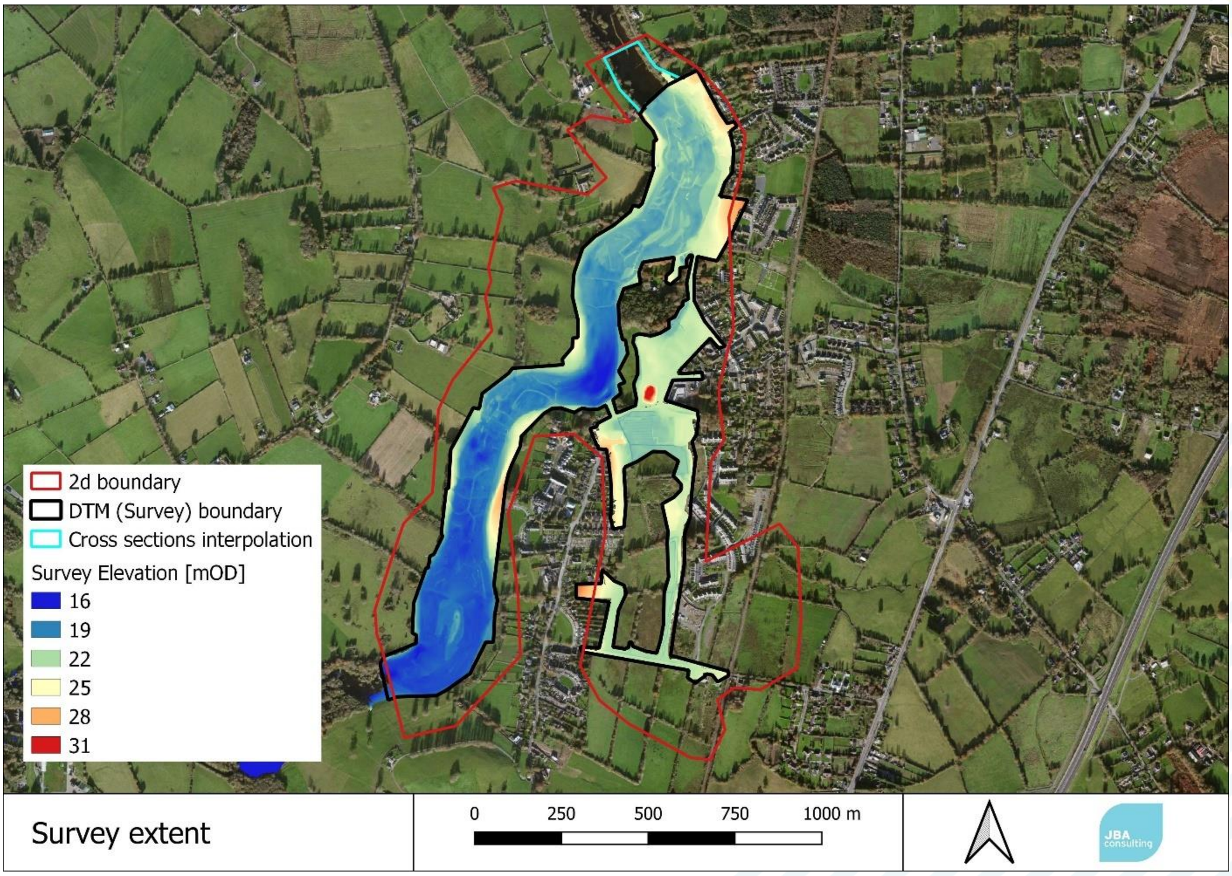


Figure 2-14: Survey extent

The survey points collected in the channel represent the riverbed, the banks and the channel features (top and base of weirs, top and base of islands etc). Breaklines were generated along the banks and the riverbed details. The survey points and the breaklines were used to generate a 3D triangulation, which was further processed to generate the DTM used in the model. An example is provided in Figure 2-15 that presents the steps undertaken from survey to DTM.

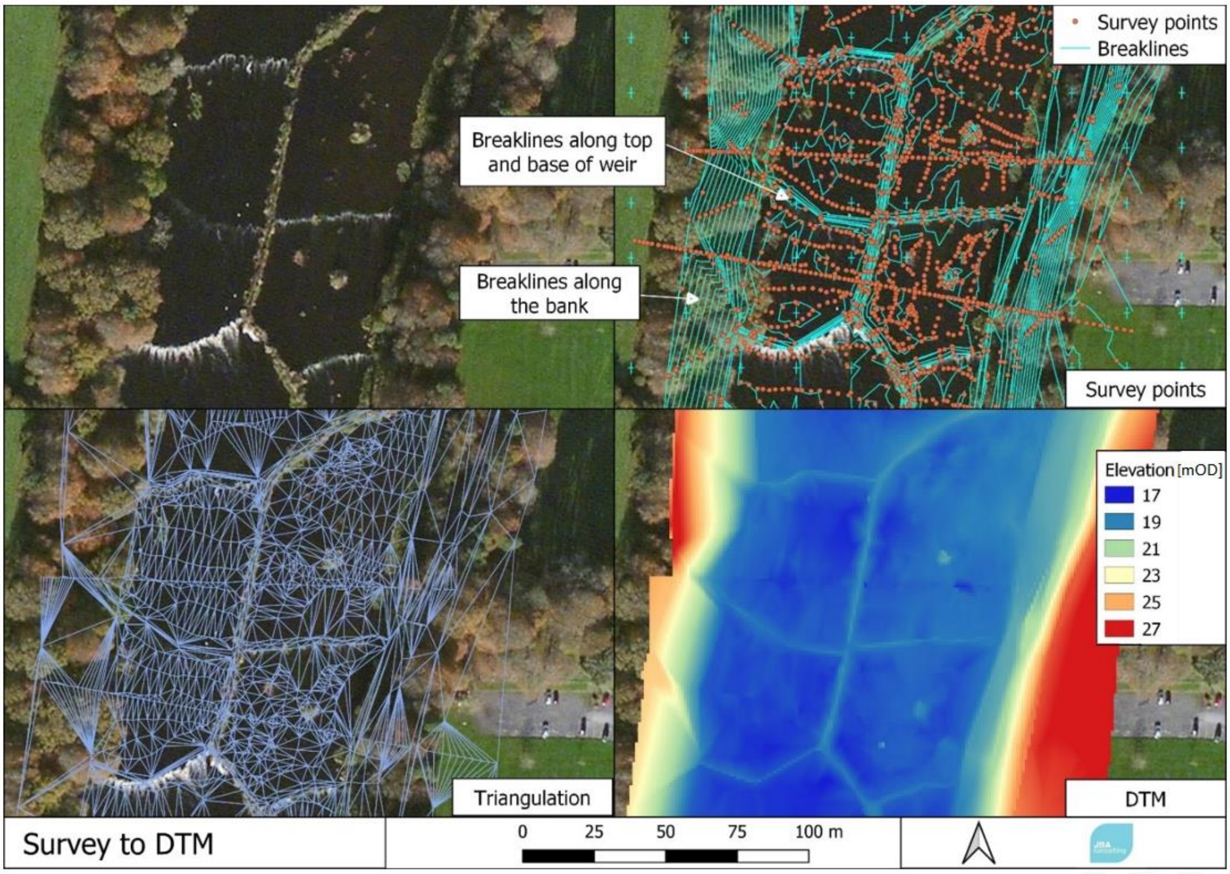


Figure 2-15: DTM generation from survey data

2.4.4 Model grid

2.4.4.1 Grid resolution

The grid used in the model is derived from the DTM described in the previous section. A multidomain with 4m and 8m grids was selected for hydraulic calculations, ensuring there is sufficient detail in the model for use in FRS development and that the model run times are not excessive. The 4m domain boundary is presented in Figure 2-16.

Approximately 17750 points have been collected in the riverbed on a total surface of 350000m^2 (0.35km^2).

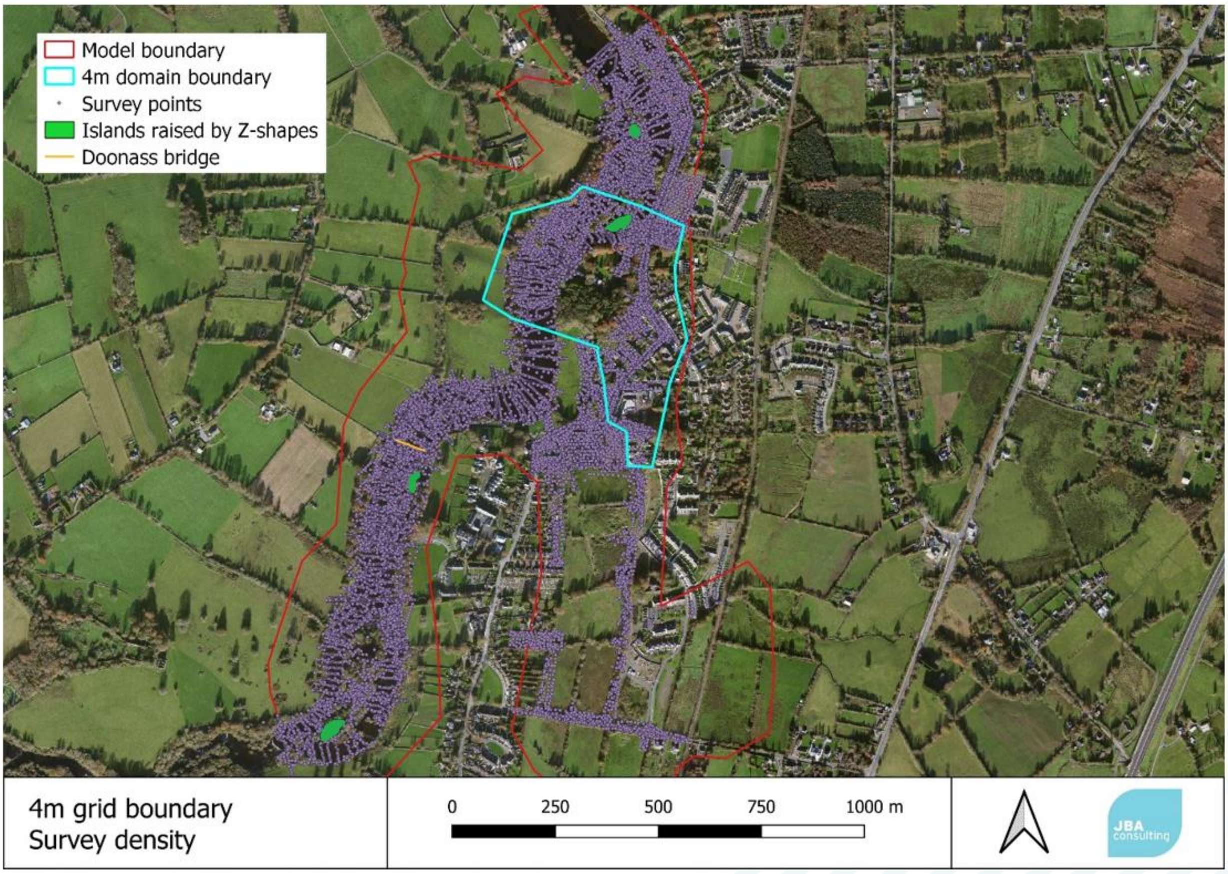


Figure 2-16: 4m grid boundary. Survey density

There is an average of 1 point per 20m^2 and the resulting average distance between points is approximately 4.5m. Tuflow samples the underline data at cell centres and at cell sides, meaning an 8m grid model samples data at every 4m, so an 8m grid is consistent with the survey detail collected and there is therefore little benefit in adopting a smaller cell resolution through much of the model. A few islands were adjusted using LiDAR points, in order to capture the high elevation unable to reached by the survey. The location of the islands is presented in the figure above.

The final model adopted a multi-domain 2d model, with upstream and downstream reaches at an 8m resolution. The area around Island House is shown to be the most sensitive to cell resolution due to the complexity of in-channel features, therefore a 4m resolution was applied at this location. The main model was tested for lower cell sizes (6m and 4m) and the differences between the different resolutions are presented in Section 4.2 on Sensitivity Testing.

2.4.4.2 Grid orientation

The model grid orientation is from North-East to South-West, aligned with the Doonass Bridge (Figure 2-17). A second grid orientation was tested – aligned with the upstream and downstream boundaries. The difference between the two grid orientations is presented in Section 4.1 on Sensitivity Testing.

The grid orientation chosen (aligned with the Doonass Bridge) is based on an increased model performance and on a more conservative approach:

- The Doonass Bridge represents a constriction in the model and the grid being aligned this way simplifies the hydraulic calculations associated with this structure, and thus increasing the model performance;
- Figure 4-1 on Sensitivity Testing shows the water levels are higher in the scenario where the grid is aligned with the Doonass Bridge, therefore it is considered the more conservative scenario.

Figure 2-17: Grid orientation

2.4.5 Manning's n - Roughness

The surface roughness, including buildings and various land uses within the 2D Shannon River model, has been applied using a 2D materials layer. OSI Prime 2 land-use polygon was used to construct the materials layer. The different Manning's n roughness values given to each land-use have been based on values from site visits, consultations of photographs, Chow 1959 and general values applied in hydrological modelling. Refer to Figure 2-18 for the modelled land use types and Table 2-3 for the corresponding Manning's n roughness values applied.

Table 2-3: Manning's roughness values applied to the 2D domain

Surface	Manning's n value applied
General Rural (baseline layer)	0.045
Riverbed	0.045
Dense vegetation	0.110
Medium vegetation	0.080
Roads	0.025
Buildings	0.300
General Urban	0.060
Large gardens	0.050

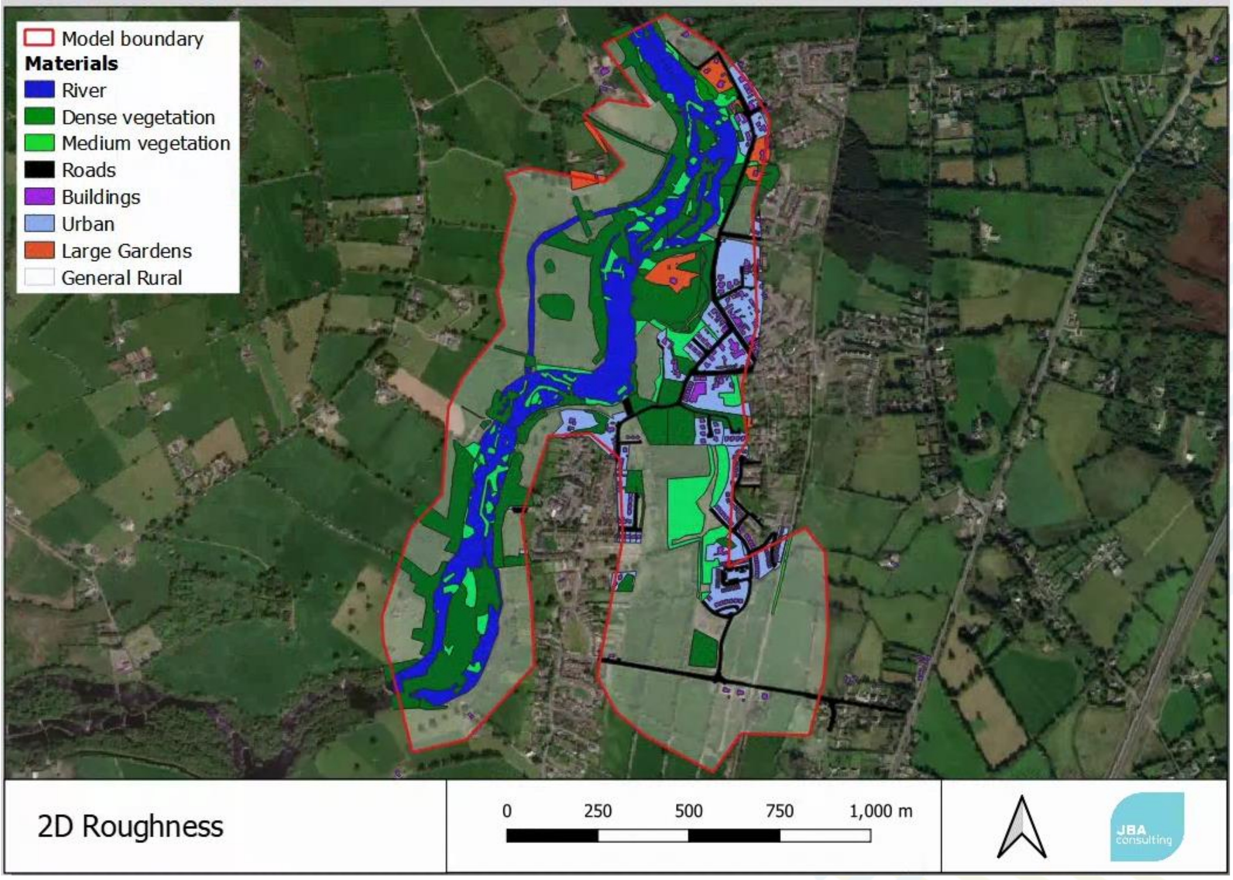


Figure 2-18: Materials layer – Spatial distribution

2.4.6 Buildings

The buildings are kept in the model and a high roughness value was applied to them (n=0.3) in order to ensure that water preferentially flows around buildings before moving through them. The buildings were modelled by manually editing the DTM using polygons assigned with the elevations of the threshold levels. The buildings modelled are presented in Figure 2-19.

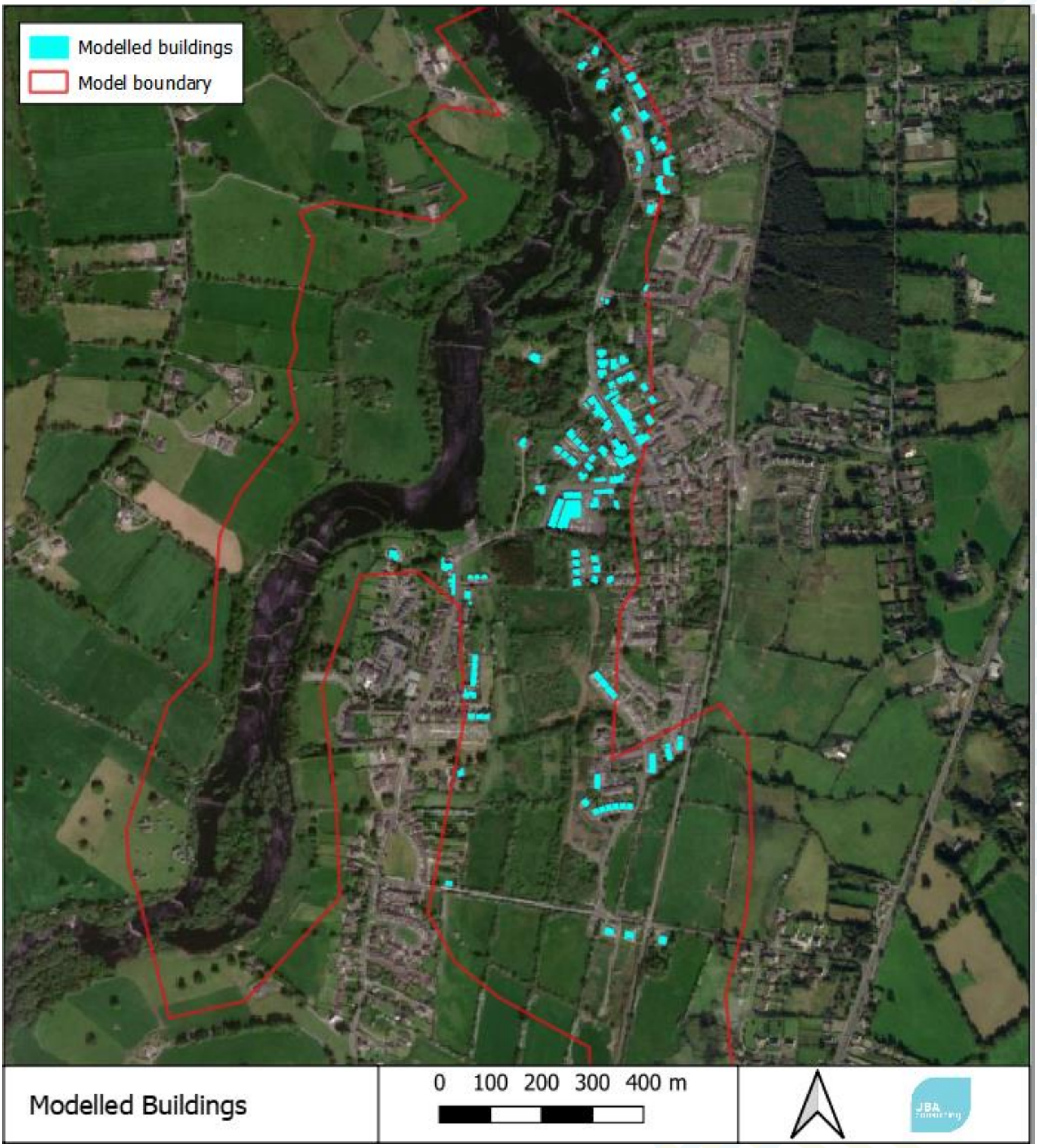


Figure 2-19: Modelled buildings

2.4.7 In-channel features

2.4.7.1 Islands

The islands within the channel are represented in the model using the processed DTM. However, a few islands have been manually adjusted with polygons and points assigned with the Lidar elevation, in order to ensure the top of the islands are captured in the model. An example is presented in the figure below, the cross section through the island showing the improvement of the island representation. The location of the islands that have been adjusted this way is presented in Figure 2-20.

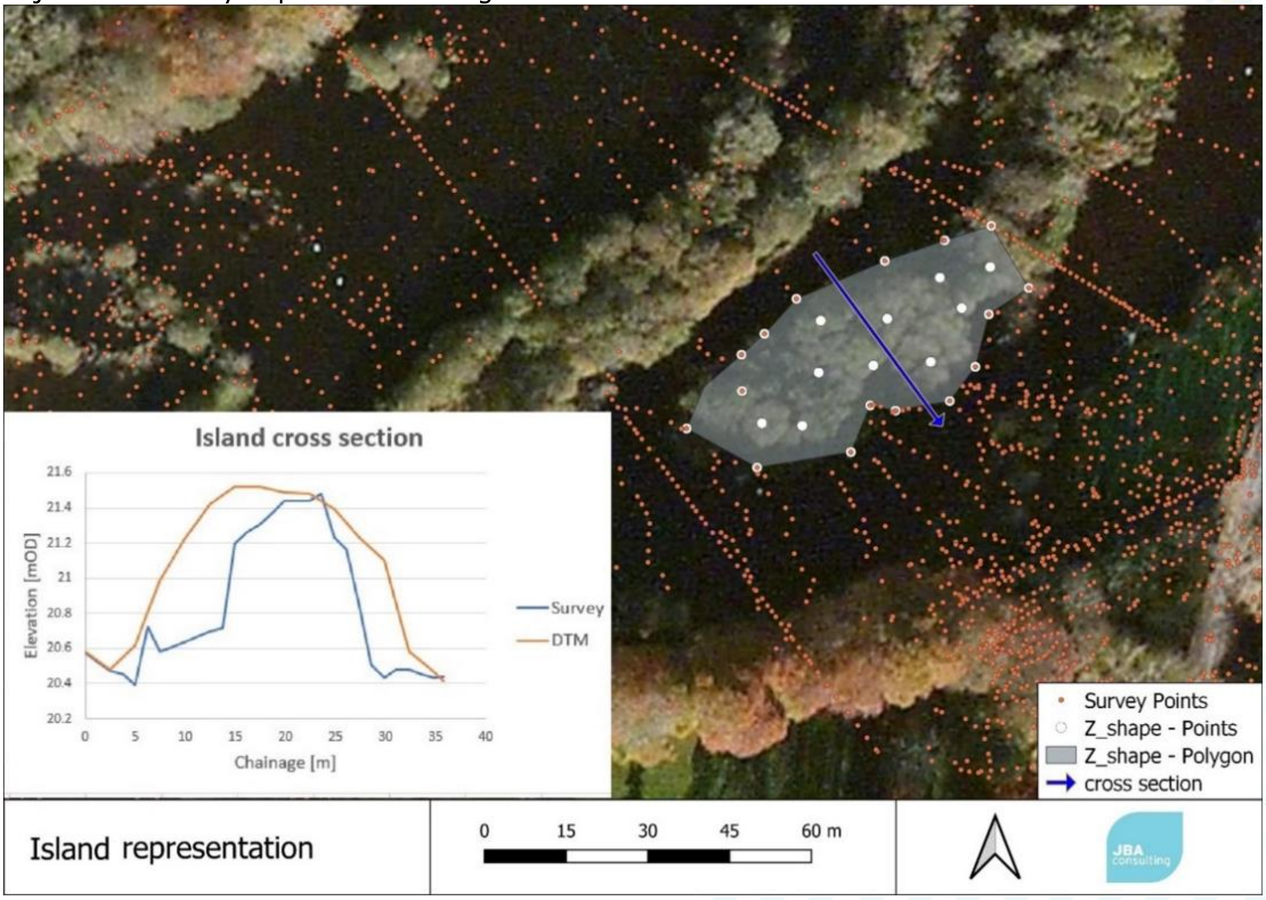


Figure 2-20: Island representation

2.4.7.2 Weirs

The weirs crests have been represented in the model using lines and points assigned with survey elevation, as shown in the figure below. Figure 2-22 presents a longitudinal section (A-A') through the channel that shows the elevation increase at the weir crests when the weirs are represented in the model using Z-shapes.

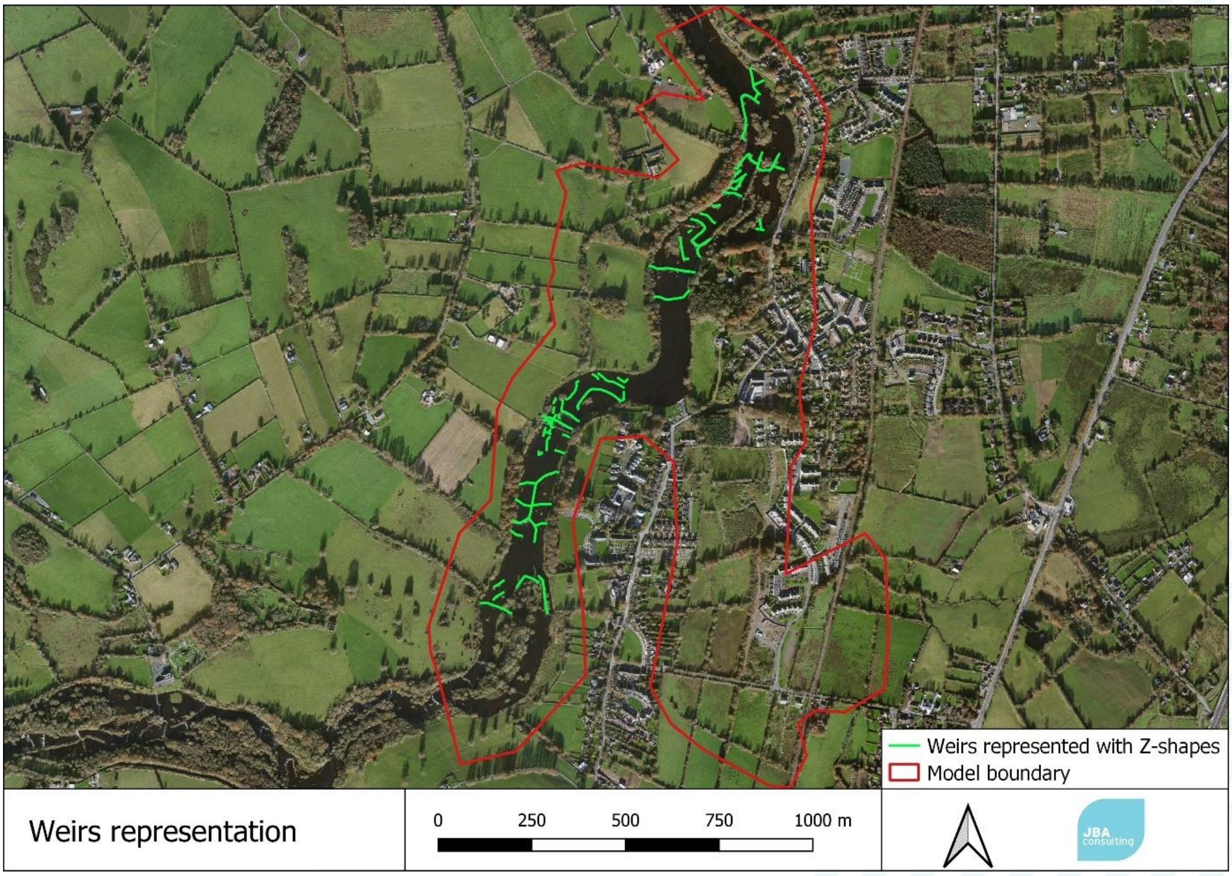


Figure 2-21: Weir Representation – Plan View

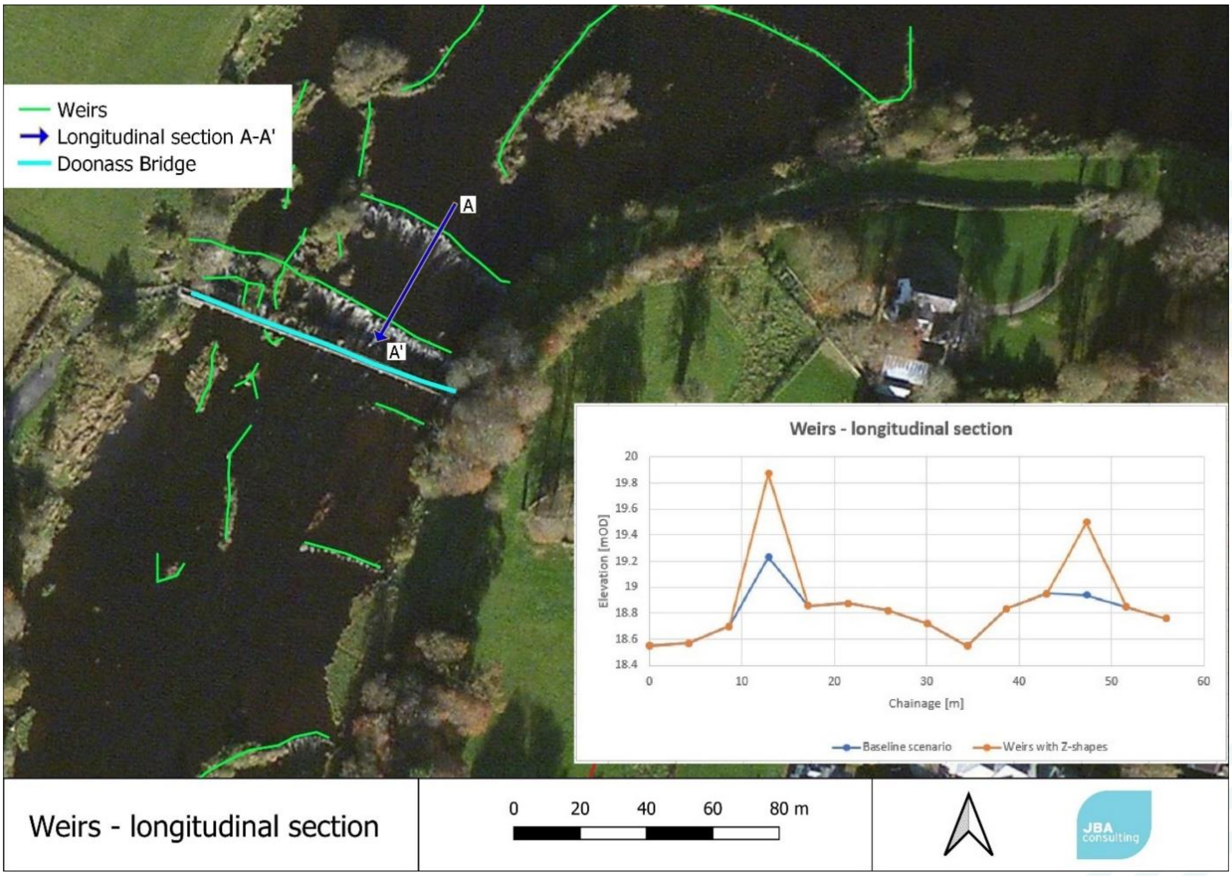


Figure 2-22: Weir representation – Longitudinal section

2.4.7.3 West Channel

The West Channel, as identified in Figure 1-1, has been modelled in the 1D domain, using the Estry software. Estry was preferred due to model stability and performance, the location of the channel being within the 2D domain, opposed to the 1D Shannon network modelled in Flood Modeller.

The channel is approximatively 800m long and it has an average width of 15m. There are 4 structures along the channel.

The 1D network was built using surveyed cross sections. The LiDAR was checked against the survey and the results for 2 cross sections are presented in Figure 2-23.

As expected, there is a significant difference at the riverbed level due to the LiDAR being unable to penetrate the water. However, there is a good match between the LiDAR and survey at the dry topography area.

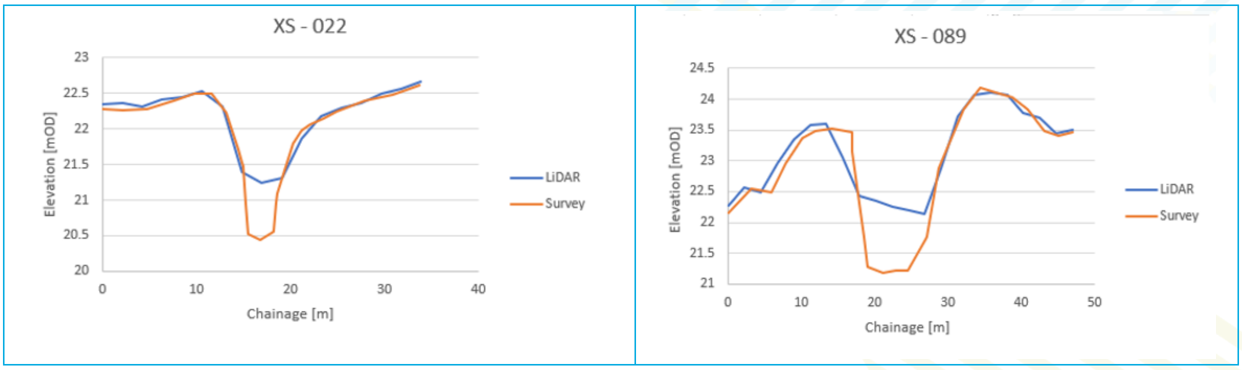


Figure 2-23: Topo-survey difference – cross sections

The channel was removed from the 2D domain and the 1D-2D link is modelled using connection lines (HX and CN lines).

Figure 2-24: West Channel 1D-2D link

The riverbed is relatively clean and straight, with some weeds and stones along the channel. Contrarily, the top of banks is heavily vegetated with trees and dense brush.

The Manning's roughness values used for the 1D network is 0.045 for the riverbed and 0.12 for the top of banks. The values are based on photo evidence and are in accordance with Chow 1959.

XS CC00012

XS CC00031

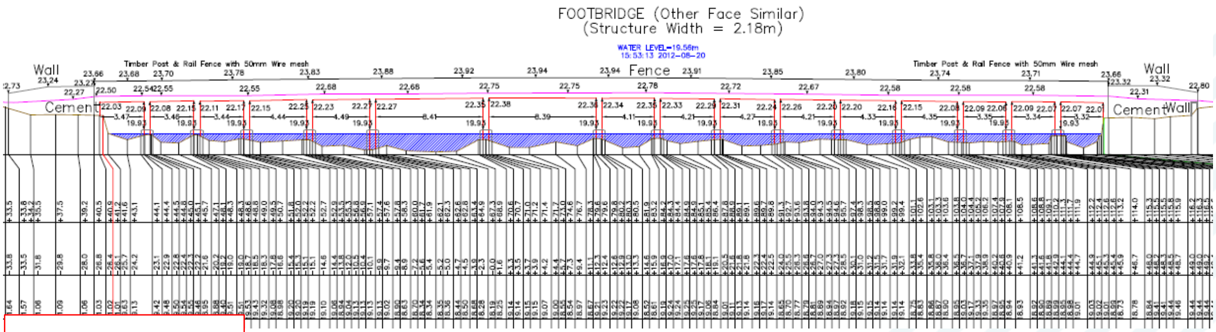
The West Channel structures are presented in Appendix A.1.

2.4.8 Structures

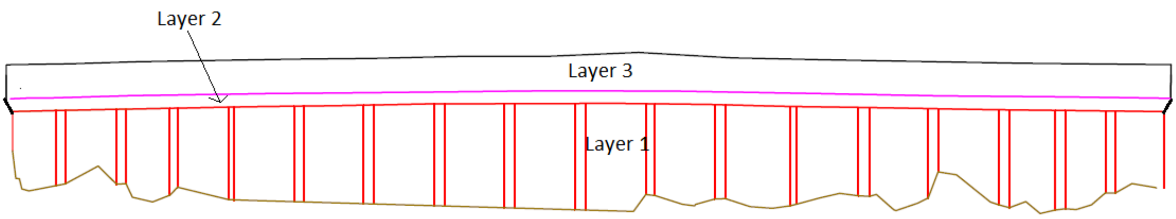
2.4.8.1 Doonass Bridge

The structure is 79m long, 2.18m wide and it has 15 piers.

DS face



DS face - survey



Model schematisation

The structure has been modelled in the 2D domain, using a Layered flow constriction shape on 3 levels:

-1st layer represents the area beneath the deck – the total area is 259.7 m^2 and the piers area plus foundation area above the bed level is 27.3 m^2, resulting a blockage of 10.5% and a loss coefficient of 0.43;

-2nd layer represents the bridge deck – 100% blockage and a loss coefficient of 1.56;

-3rd layer represents the bridge rails – 15% blockage and a loss coefficient of 0.55.

Modelling approach

Figure 2-25 shows the materials layers and the features included in the model at Doonass Bridge and the roughness values are presented in Table 2-3.

The concrete blocks and walls at the sides of the bridge are represented in the model as Z-Shape lines. The material layer applied upstream of the bridge on both banks is Dense vegetation (n=0.11) due to site visits and photographic evidence, while the weirs and the outcrops rocks are also represented in the model as Z-Shape lines.

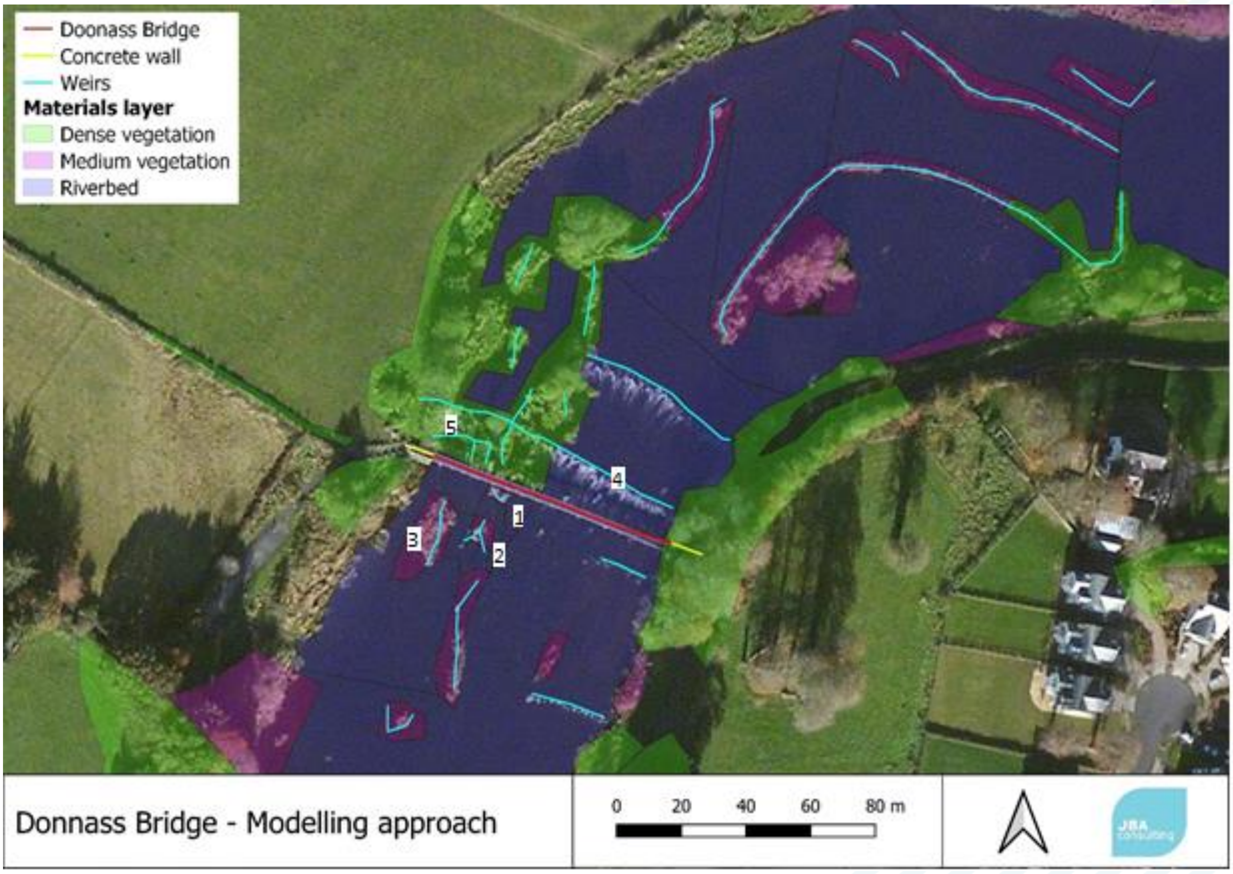


Figure 2-25: Doonass Bridge – Modelling approach

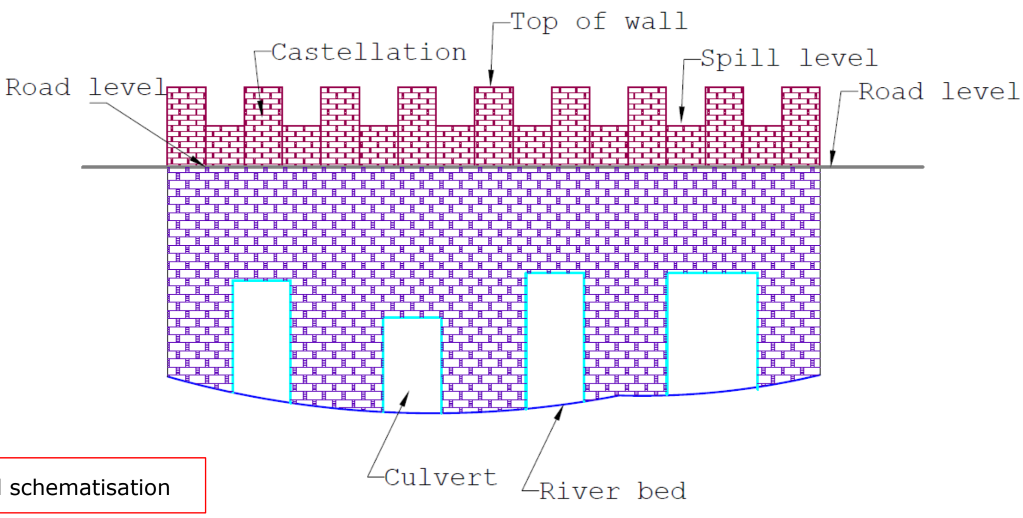
The photos below show features added into the model, with the corresponding ID numbers in Figure 2-25.

2.4.8.2 Island House Structure

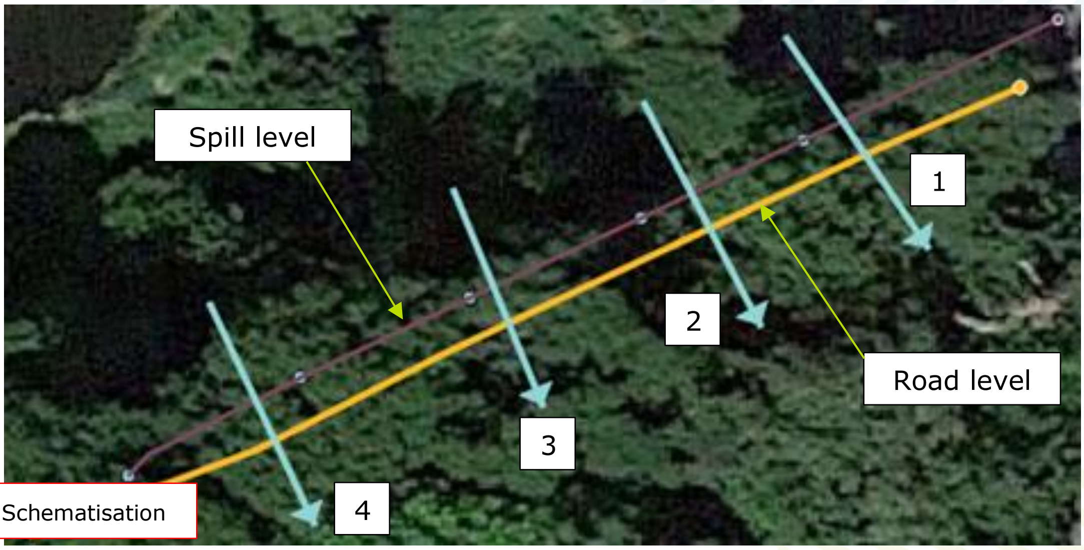
Structure type	**Dimensions**
Bridge with castellations	Width - 8m; Length – 46m
 US face	 Street view

Modelling approach

The structure has been modelled in the 2D domain with the culverts beneath the access road modelled as 1D Estry elements.
The spill is modelled using a constriction flow layer (thin line) with a 50% blockage, in order to represent the castellation wall. A thick line was used to represent the road level.



Model schematisation



2D Schematisation

The 4 culverts are equipped with flood gates, installed by LCCC after the 2009 flood event.

The model was run with the flood gates both open and closed and the results are presented in Section 4.5.

The culverts characteristics are presented in the table below, showing the downstream faces are partially blocked. The culverts were modelled with the full bore area available for the design runs.

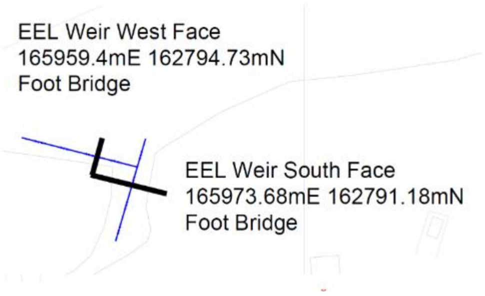
Table 2-4: Island House bridge – Culverts dimensions

Culvert ID	Face	Invert level	Soffit level	Height [m]	Width [m]	Shape
1	US	21.73	23.29*	1.56	0.6	Rectangular
	DS	21.55	22.27	0.72		
2	US	21.51	22.87	1.36	0.6	Rectangular
	DS	21.81	22.09	0.28		
3	US	21.03	23.22*	2.19	0.6	Rectangular
	DS	20.92	22.18	1.26		
4	US	20.89	22.83*	1.94	1.6	Arch
	DS	20.86	22.56	1.70		

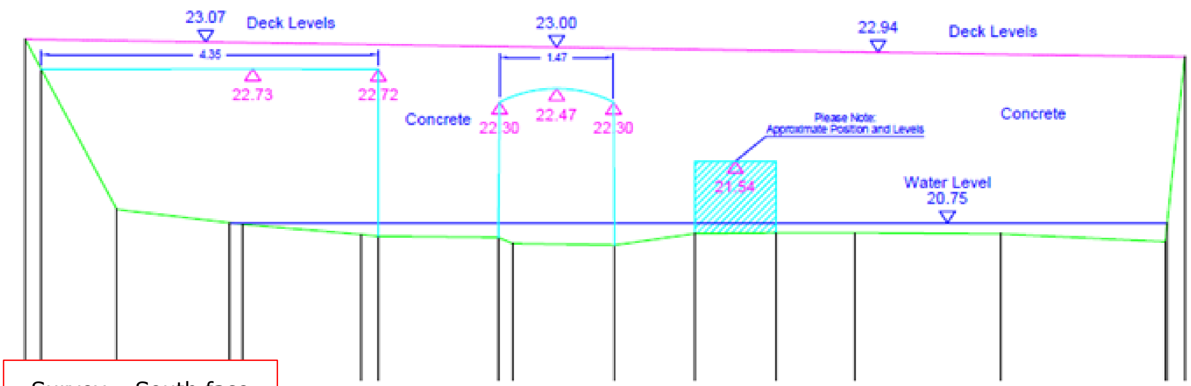
*The upstream soffit level used for the culverts 1,3 and 4 is the top of flood gate.

2.4.8.3 Eel bridge

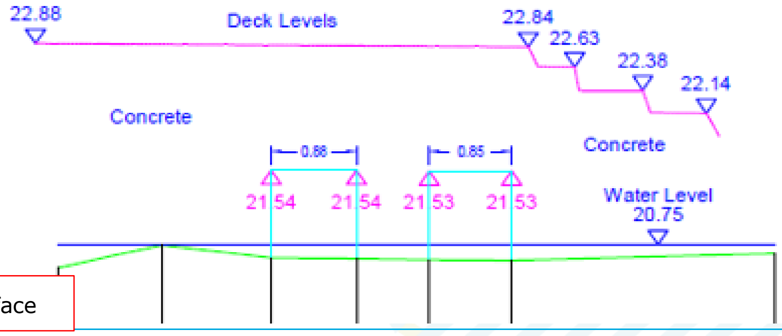
South face dimensions	West face dimensions
Length – 15m	Length – 7.5m
Width – 1.2m	Width – 1.2m



Survey – plan view

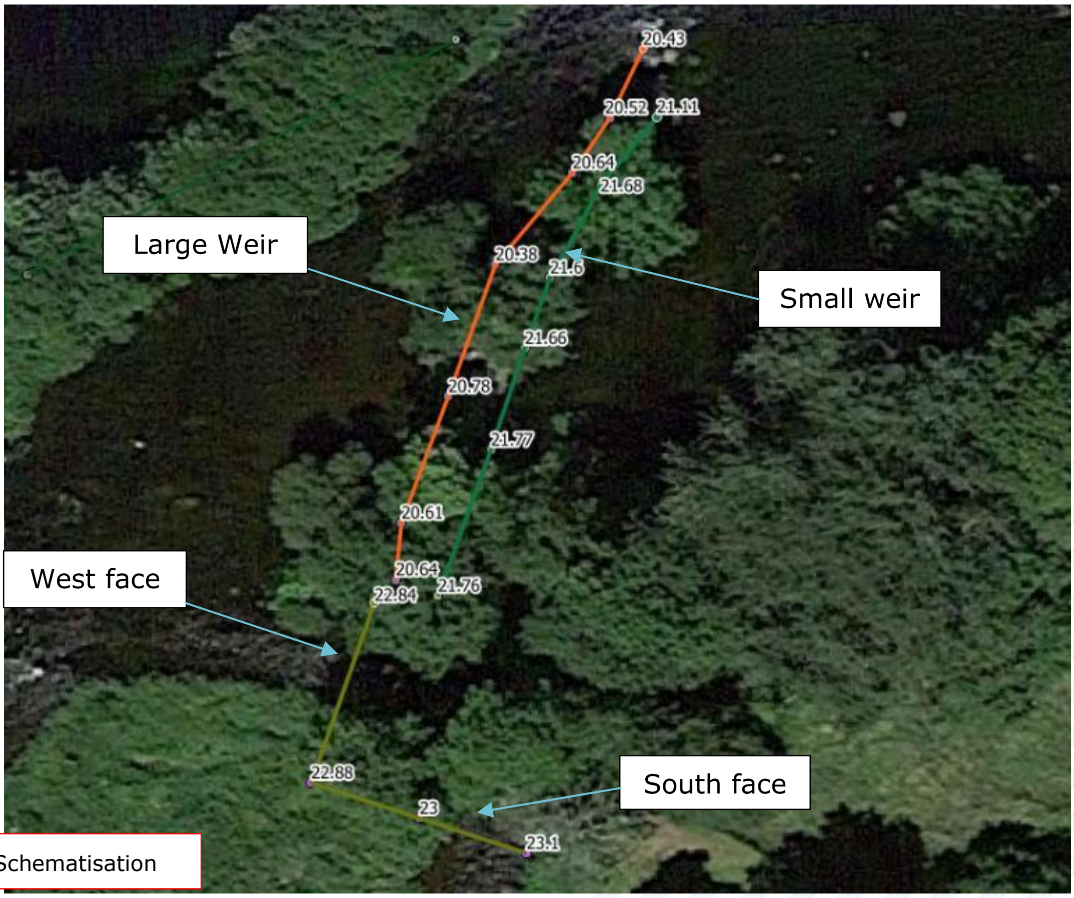


Survey – South face



Survey – West face

Modelling approach
The Eel bridge was modelled using a Z_shape line. Two scenarios were analysed: - worst case scenario: the culverts are not included in the model in order to simulate a complete blockage at the culverts - best case scenario – complete opening within the structure at the culverts – the entire cell is lowered to the bed level to represent an excessive opening in the structure. The differences between the 2 scenarios are presented in Section 4.4. The large weir next to the bridge was modelled using a thick Z_shape line and the small weir using a thin Z_shape line.



2D Schematisation

2.4.8.4 Stradbally culverts

As stated in Section 2.3, the Stradbally watercourse is included directly within the Shannon River, with an extended boundary upstream of the railway compared to the CFRAM model (refer to Figure 1-1 for the watercourse location).

There are 4 culverts on Stradbally watercourse, all of them being modelled as 1D Estry elements (Figure 2-26). Site inspections indicate that there may be dumping of debris in front of Culvert No. 2. This has not been included in the model as a blockage factor.

Culvert nr 1 and 2 are circular and have a 0.9m diameter. Nr 3 is composed of two circular culverts of 0.6m diameter and culvert nr 4 is arch but modelled as rectangular (due to pipes constrictions) with 1.88mx1.05m dimensions.

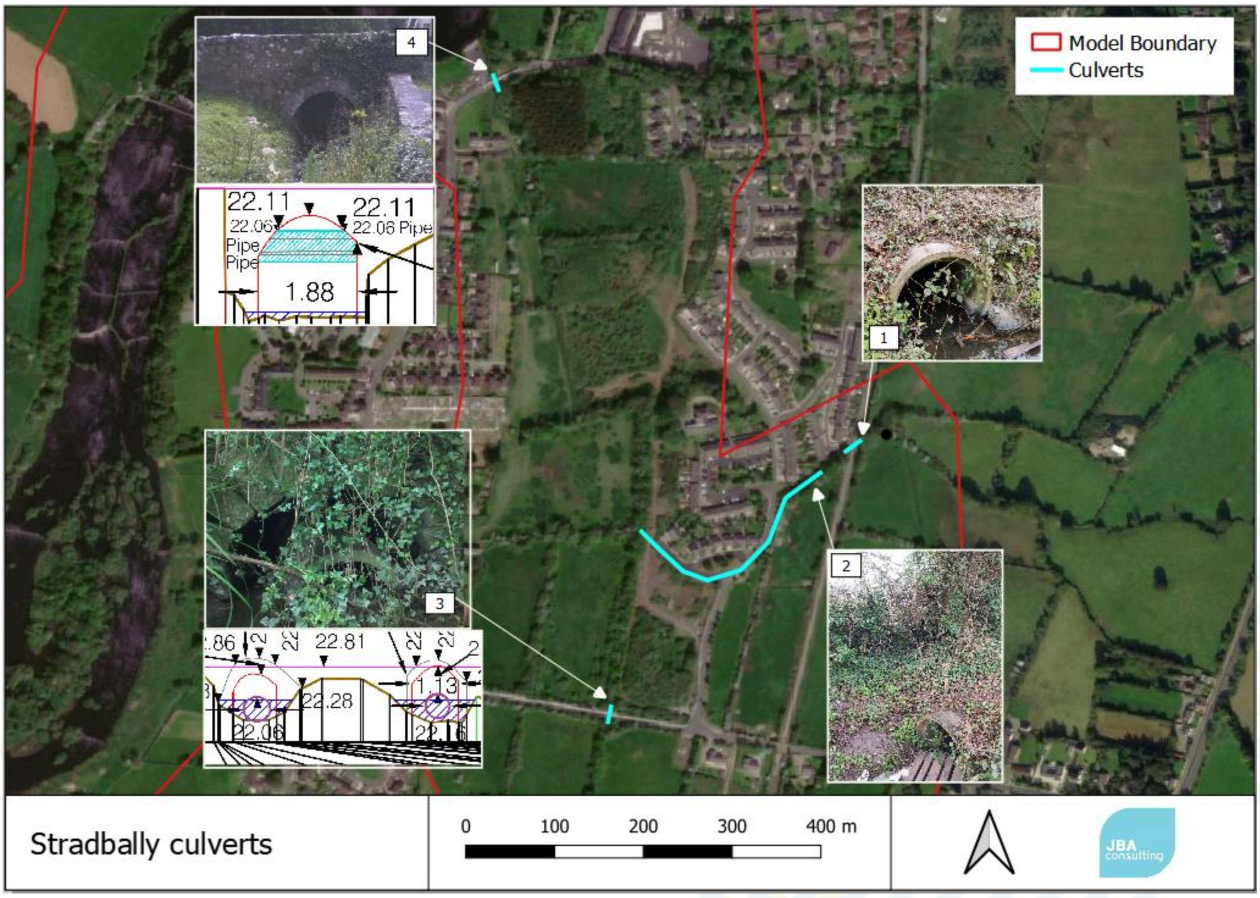


Figure 2-26: Stradbally culverts

3 Model Calibration and Validation

In Castleconnell area recent significant flood events occurred in November 2009, December 2015 and February 2020. Feb 2020 had the benefit of a comprehensive flood data collection programme.

The model has been calibrated against the 2020 event and validated against the 2009 event. Priority was given to the 2020 event due to the increased reliability and distribution of flood marks across the reach. The 2015 flood event was not assessed due to insufficient reliable data to form a calibration.

The 2020 event occurred during the lifetime of the project, which allowed retrieval of calibration data (water levels and flows) in real time: flood wracks were placed at key points along the study area and the flow over Parteen Weir was recorded daily.

The 2009 event was used because of its high scale magnitude (very similar with the 1% AEP flood event), while observed flood extents in the village and observed water levels at different locations along the model were available to form a reliable validation of the model.

The specifics of the 2009 and 2020 flood events are represented in the model based on site observations, while a conservative approach was assumed for the design runs in order to ensure the flood risk for a future flood event is assessed in the worst-case scenario. The difference between 2009 event, 2020 event and the design runs are presented in Table 3-1.

Table 3-1: Difference between events

	2009 event	2020 event	Design runs
Doonass Bridge Blockage	67% blockage applied on the left side (8m, roughly 2 opes)	No blockage	67% blockage on the left side (24m, roughly 5 opes)
Sluices at Island House bridge	Open	Closed	Open
Castellations at Island House Bridge	Included	Included	Included
Walls on the Elvers Rd	Included, gaps in the wall open	Included, gaps in the wall closed	Not included

3.1 February 2020 event – Calibration Event

The peak flow over Parteen Weir during the 2020 event was 410 m^3/s, which would be the approximate equivalent of an event between 5% AEP (394 m^3/s) and 2% AEP (458 m^3/s) during standard operational conditions. To note the heads race flow to Ardnacrusha was 376 m^3/s as a result of the levels in the basin and canal at that time. The flows and times corresponding to the Kilmastulla hydrograph are derived from recorded data at Coole Gauge during the flood event, applying routing effects from the gauge to the confluence with Shannon River. The peak flow on the Kilmastulla River at the confluence with the Shannon River was 30.75 m^3/s, slightly higher than the 5% AEP event (28.71 m^3/s). The peak flow on Kilmastulla occurred on the 22nd of February, corresponding to the 335m^3/s flow over Parteen Weir.

The Shannon and Kilmastulla hydrographs used in the 2020 event are presented in Figure 3-1.

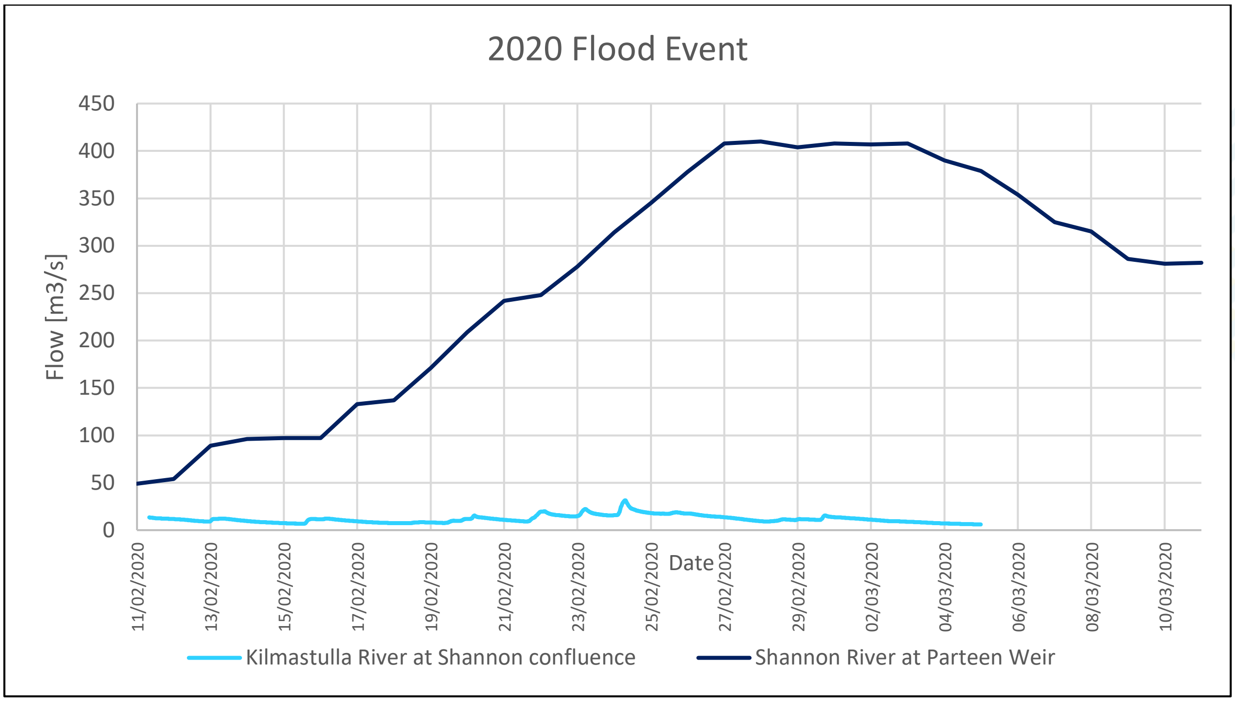


Figure 3-1: 2020 event – Flow Hydrograph

The orange wrack marks represented in Figure 3-2 were collected before the flood peak, on the 26th February and the blue wrack marks were collected at high peak, on the 1st of March.

Figure 3-2 presents the calibration results – the water level difference between the recorded water levels and modelled water levels at the wrack marks.

The model is calibrating well against the 2020 event, with most of the points being in a range of +/- 100mm and a few in an acceptable range of +/- 200mm. All model results are within the specified range of accuracy from the tender specifications.

Figure 3-2 presents the flood extent and the location of the wrack marks. During the flood event it was noted that the river water levels had a degree of wave action, therefore the wrack marks would be the peak water level observed and not the average still water level.

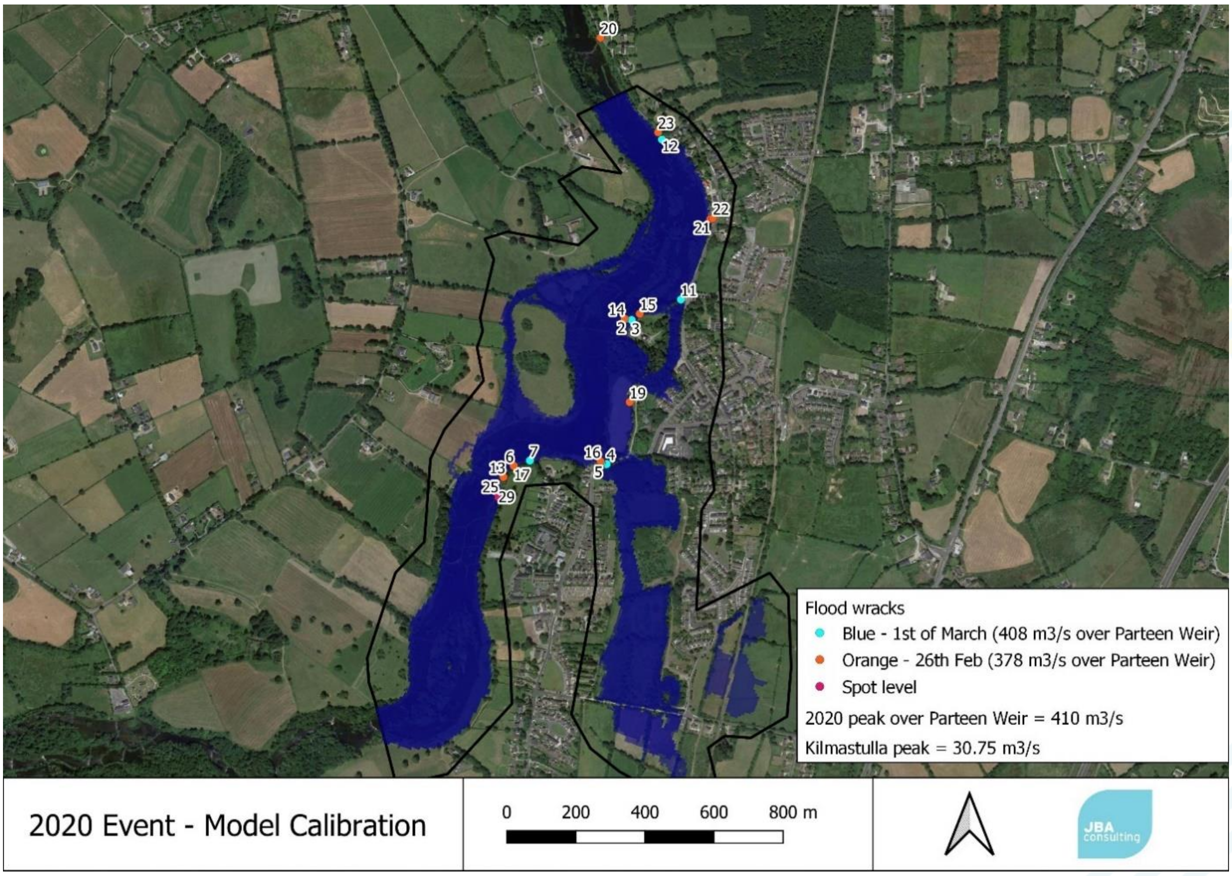


Figure 3-2: Model Calibration – 2020 event

The water levels at wrack marks are presented in Table 3-2.

Table 3-2: 2020 Event Calibration

Location	ID Flood wrack	Measured WL [mOD]	Modelled WL [mOD]	Difference [m]
1D model	20	24.44	24.52	0.08
US Cedarwood confluence	23	23.93	23.78	-0.15
	12	23.92	23.85	-0.07
US Western Channel	21	23.57	23.51	-0.06
	22	23.49	23.51	0.02
Island House Structure	11	23.29	23.29	0
US Eel bridge	15	22.92	22.96	0.04
	3	22.96	23.04	0.08
Eel Bridge	1	23.17	23.08	-0.09
	2	23.11	23.06	-0.05
	14	23.02	22.93	-0.09
DS Island House channel	19	22.47	22.48	0.01
Car park	4	22.56	22.63	0.07
	5	22.56	22.63	0.07
	16	22.46	22.48	0.02
US Doonas Bridge	7	22.25	22.3	0.05
	6	22.01	22.16	0.15
	17	21.95	22.02	0.07
	13	21.77	21.78	0.01
DS Doonas bridge	25	21.35	21.33	-0.02
	29	21.4	21.33	-0.07

3.2 November 2009 Event – Validation Event

The peak flow of 2009 event over Parteen Weir was 500 m³/s, 4 m³/s lower than the 1% AEP event (with 345 m³/s headrace flow to Ardnacrusha). The flows and times corresponding to the Kilmastulla hydrograph are derived from recorded data at Coole Gauge during the flood event, applying routing effects from the gauge to the confluence with Shannon River. The peak flow on the Kilmastulla River at the confluence with the Shannon River was 25.72 m³/s, lower than the 5% AEP event (28.71 m³/s). The peak flow on Kilmastulla occurred on the 23rd of November, corresponding to the 416m³/s flow over Parteen Weir.

The Shannon and Kilmastulla hydrographs used in the 2009 event are presented in Figure 3.3.

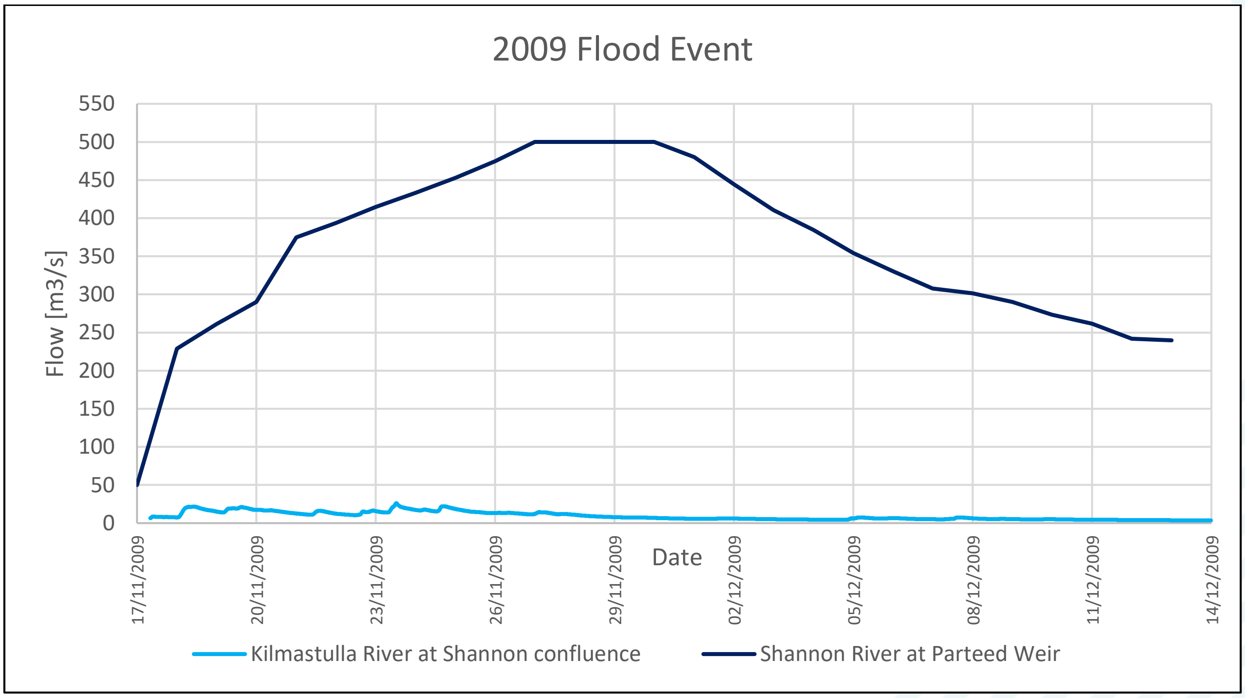


Figure 3-3: 2009 event – Flow Hydrograph

The calibration data available for the 2009 event are observed flood extents and observed water levels at properties and at car park. The calibration results are presented in Figure 3-4.

The observed water levels at the upstream end of the model are based on information provided from local residents, namely "the water levels came within 1 inch of houses' threshold levels". The threshold level of the first house from upstream (refer to Figure 3-4) is 23.99 mOD and the threshold level of the second house is 24.06 mOD. Therefore, the adopted observed water levels used for the model validation are 23.97mOD and 24.04 mOD.

Observed water level within the car park was provided by LCCC.

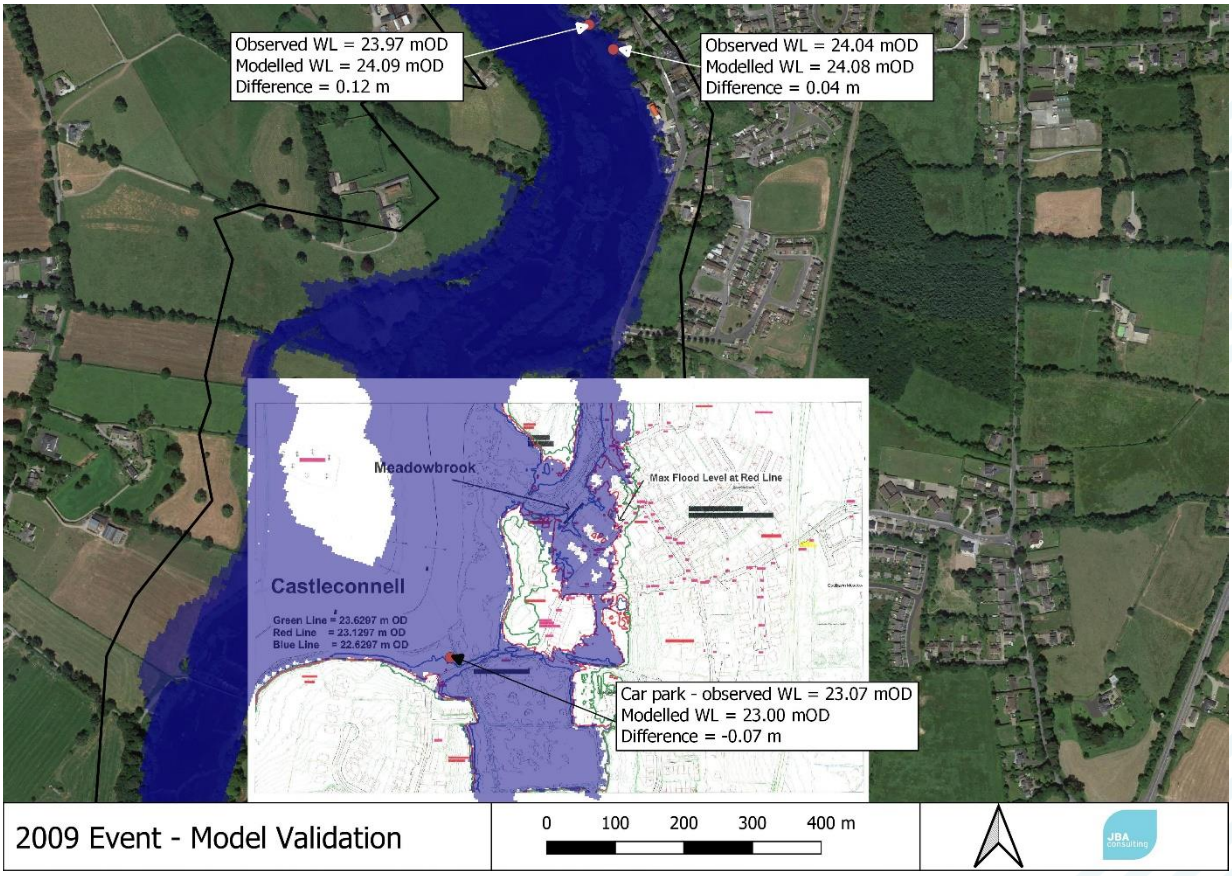


Figure 3-4: Model Validation – 2009 Event

In order to replicate the conditions during the flood event, a 67% blockage was applied on the left side of the Doonass Bridge (8m) based on site observations.

3.3 CFRAM model comparison

Immediately downstream of the study area – in the 1D only model - CFRAM model used 2 cross sections (12LSH02258 and 12LSH02084) to estimate an island, due to lack of survey availability. The island and the estimated cross section location are presented in the figure below.

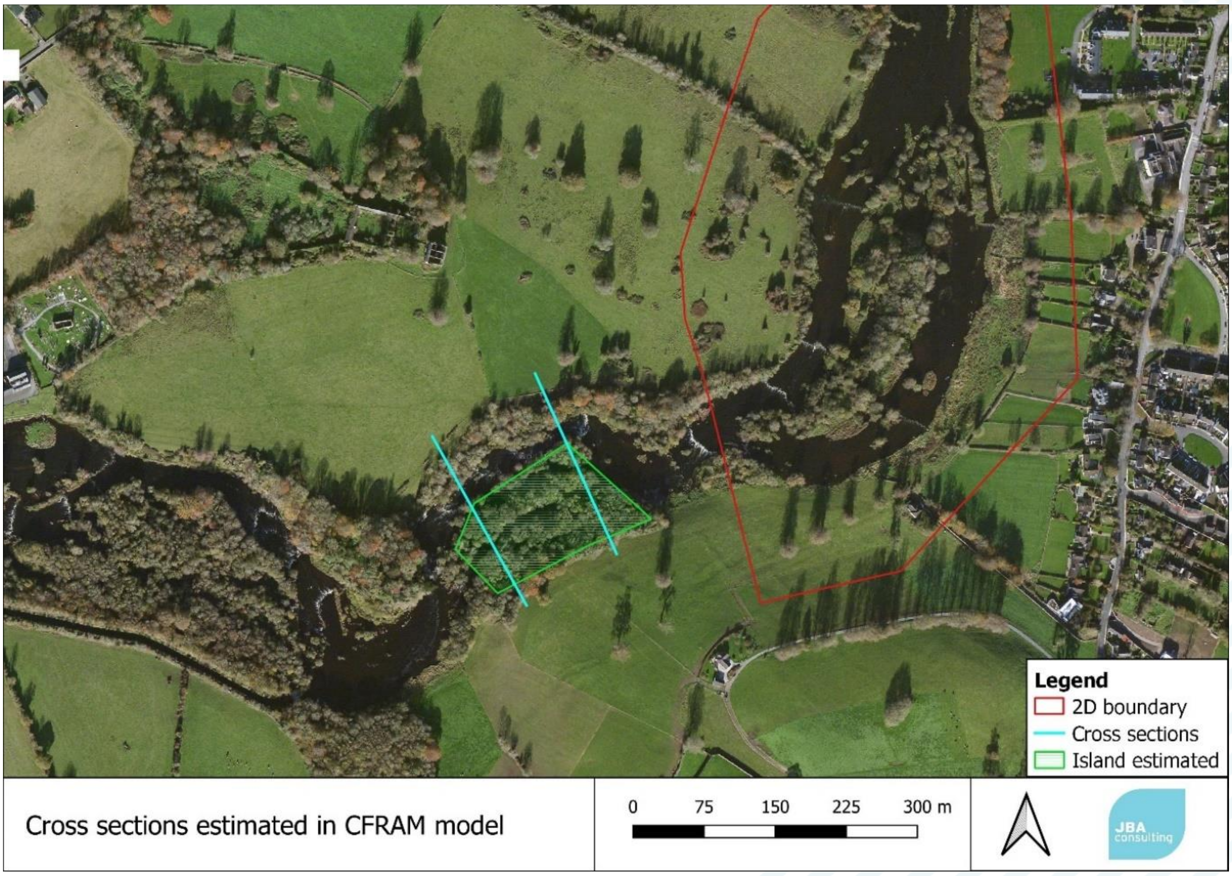


Figure 3-5: CFRAM model – cross sections estimated to replicate channel restriction

As displayed in Figure 3-8, there is a very steep slope of the riverbed in the area immediately downstream of the 2D model, causing white waters. Due to health and safety reasons, this area was not surveyed in detail for the FRS model and therefore it was not included in the 2D only model. However, survey information was managed to be collected when the water levels were low, namely 2 cross sections (at the same location as the CFRAM estimations) that were used in the 1D only model.

Figure 3-6 displays the comparison between the cross section used in the 1D FRS model (based on survey) and the 1D CFRAM model (based on estimations), showing the CFRAM model overestimated the island and the riverbed elevation, resulting in an effective blockage that increased the water levels upstream.

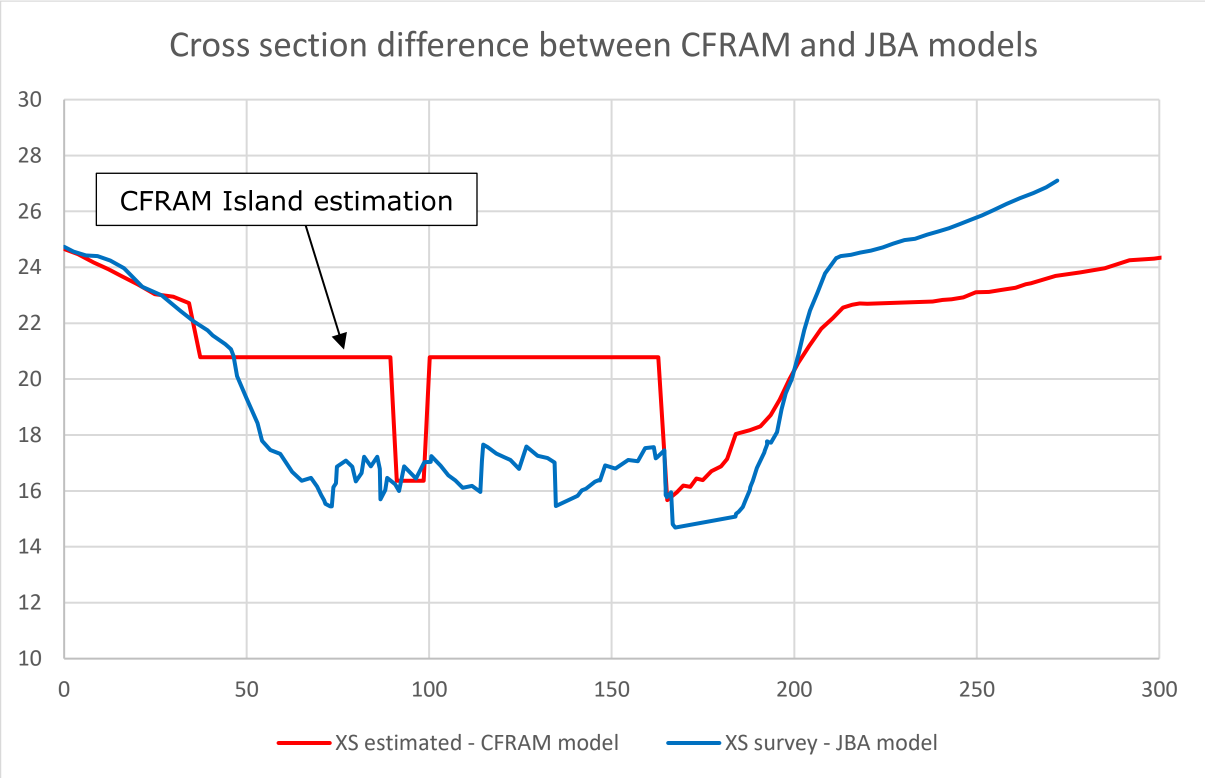


Figure 3-6: Cross section 12LSH02258 - Difference between CFRAM model and FRS model

In order to understand the effect of this estimation, CFRAM model has been run using recent survey in this area, as a test. The results are presented in Figure 3-7 and shows a comparison with the FRS and original CFRAM results. The longitudinal profile shows the CFRAM model significantly overestimates the water levels as a result of the blockage applied downstream**, which impacts on water levels in Castleconnell**.

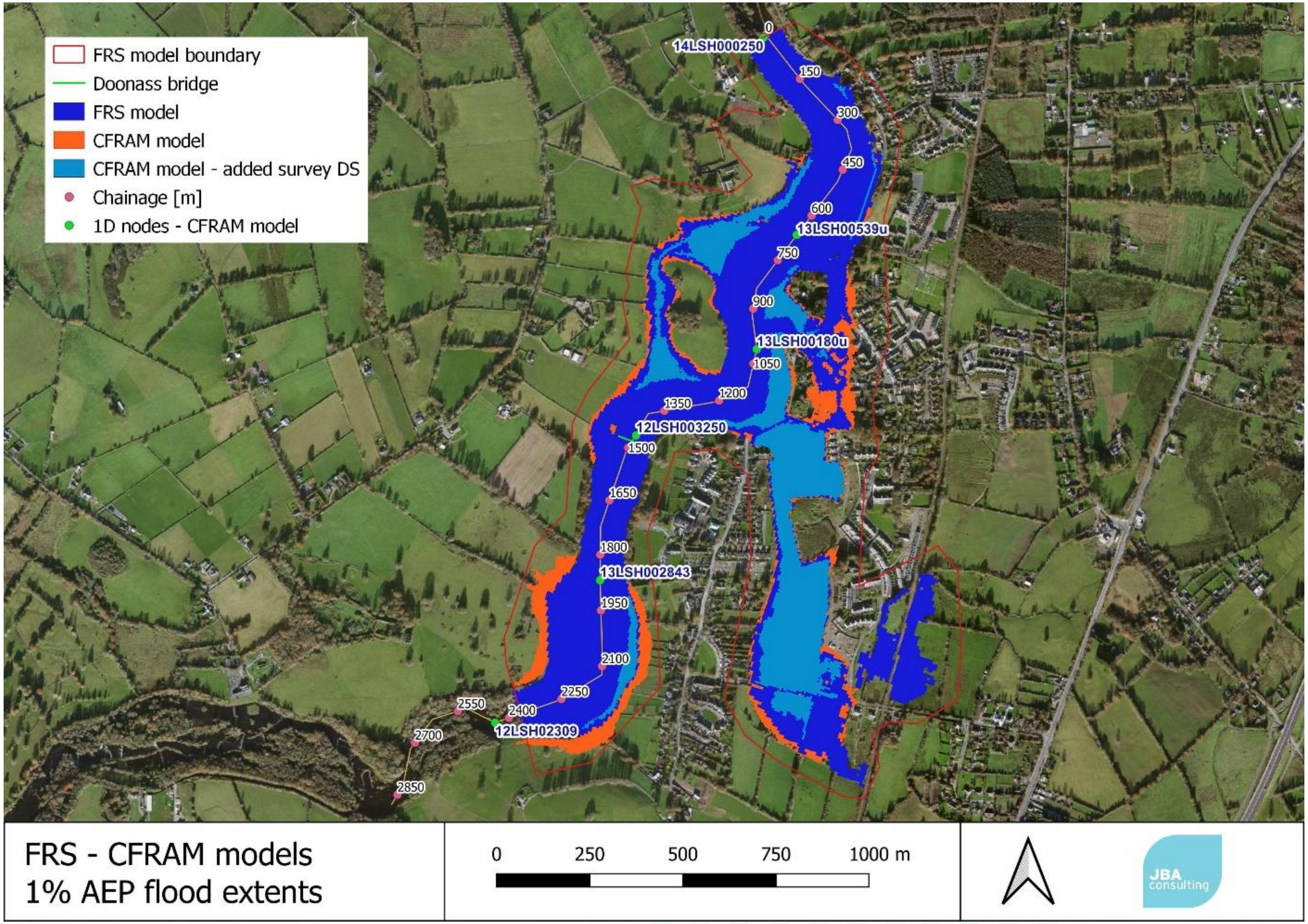


Figure 3-7: 1% AEP Flood extents – FRS and CFRAM models

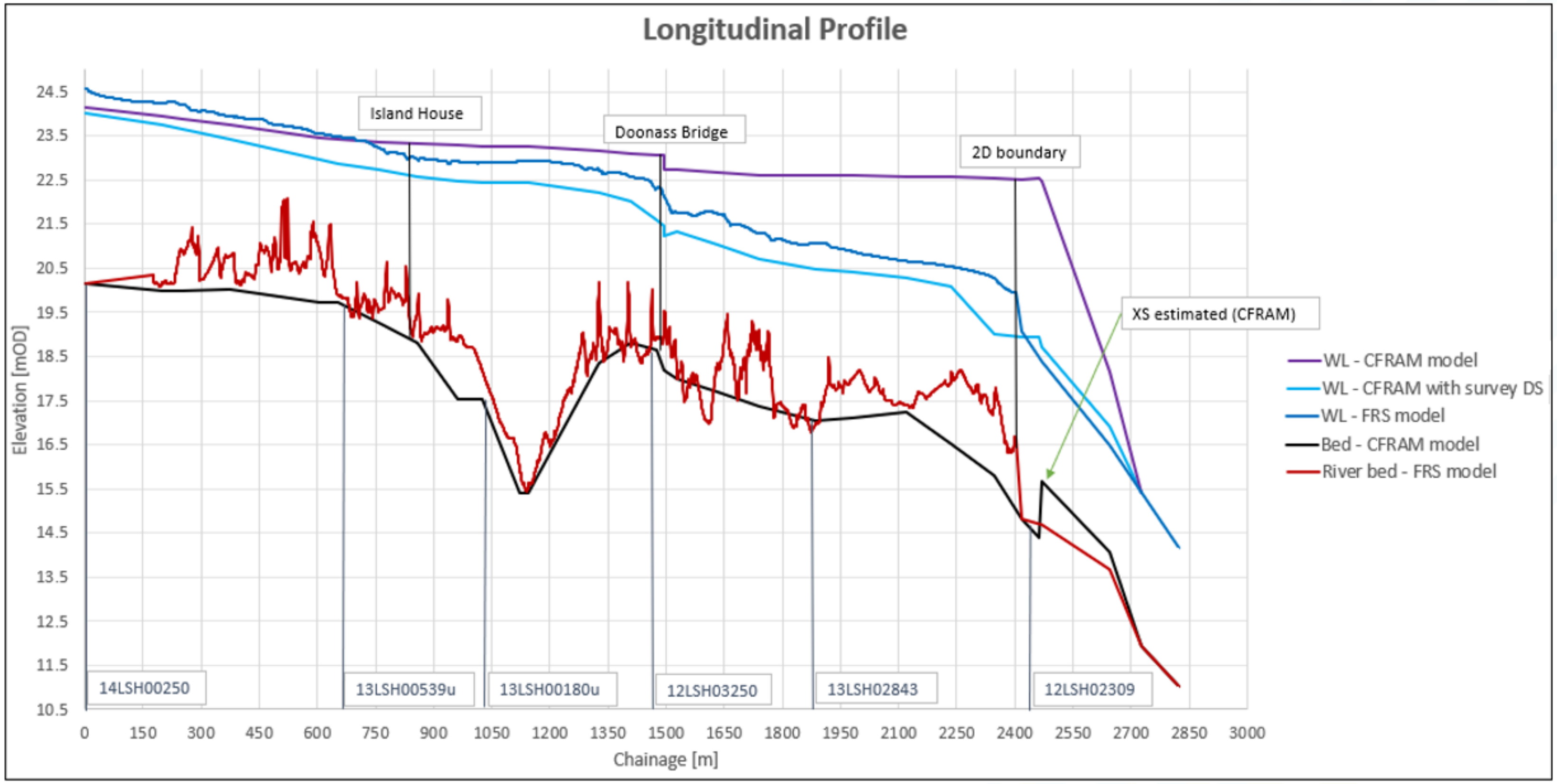


Figure 3-8: 1% AEP (with 345 m³/s headrace flow to Ardnacrusha) Longitudinal profile– FRS and CFRAM water levels

The 2 cross sections estimated by CFRAM model and surveyed in the FRS model are situated at chainage 2500m and 2650m.

The peak flows of the 1% AEP into the 2D (FRS model) and 1D-2D (CFRAM model) domains that cover the study area are 514.4 m^3/s and 524.8 m^3/s respectively (both in the standard operational conditions with 345 m^3/s headrace flow to Ardnacrusha). The CFRAM flows were extracted from node 14LSH00250, the same location as the start of the FRS 2D domain.

The difference in bed levels observed in the longitudinal profile illustrates the FRS 2D model chosen approach far better represents the variation in bed level slope and corresponding water levels than using a 1D linearly interpolated approach. The FRS 2D model samples the survey data at 4m along the study area (from chainage 150m to 2400m), while the CFRAM model interpolates cross sections located approximately every 100m.

The water level differences between FRS and CFRAM models at 6 nodes along the study area for the 1% AEP event are presented in Table 3-3.

Table 3-3: FRS and CFRAM models – water levels - 1% AEP (with 345 m^3/s headrace flow to Ardnacrusha)

Node	CFRAM WL [mOD]	FRS WL [mOD]	Difference [m]
14LSH00250	24.15	24.55	-0.4
13LSH00539u	23.46	23.44	0.02
13LSH00180u	23.28	22.88	0.4
13LSH03250	23.05	22.21	0.84
13LSH02843	22.6	21.05	1.55
12LSH02309	22.52	19.06	3.46

4 Model Sensitivity Testing

A series of sensitivity analysis have been undertaken in order to test the model results based on uncertainty in a number of hydraulic input parameters.

The following sensitivity tests have been carried out for the 1% AEP standard operational conditions (345 m^3/s headrace flow to Ardnacrusha - refer to Table 5-1). This is a significant flow under which to test the sensitivity of the model set up and model parameters.

4.1 Grid orientation

Two grid orientations were tested: the grid aligned with the upstream and downstream boundaries and the grid aligned with the Doonass Bridge. The water levels are lower by an average of 30 mm in the scenario where grid is aligned with the US and DS boundaries, as presented in the figure below.

The model grid orientation adopted for the model is from North-East to South-West, aligned with the Doonass Bridge, as discussed in Section 2.4.4.2.

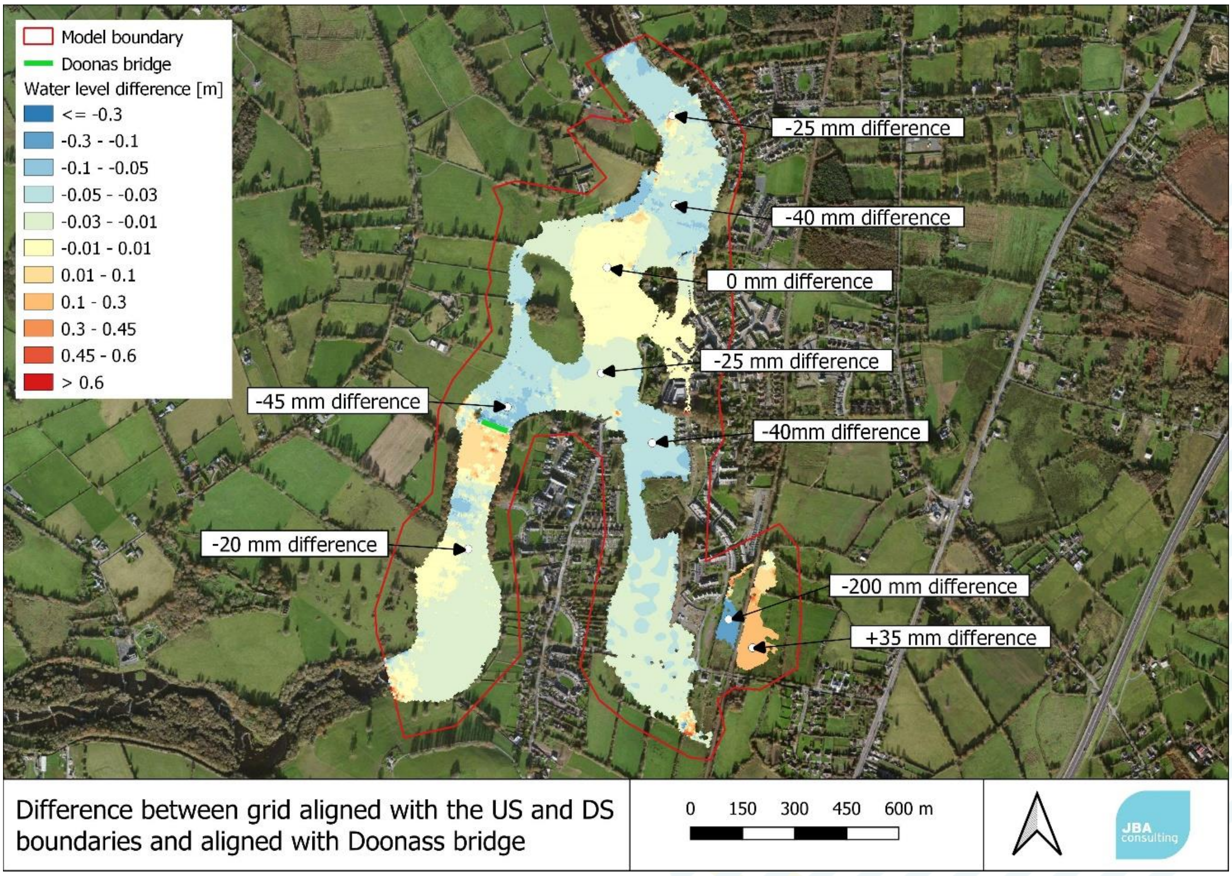


Figure 4-1: Grid orientation – water level difference (grid aligned with the US and DS boundaries - grid aligned with the Doonass Bridge)

4.2 Model resolution

The model has been run using the following grids derived from Survey and Lidar:

- 8m;
- 6m;
- 4m;

Multi-domain: 8m + 4m. The 4m domain includes the following: Island House Channel, Island House Structure, Eel Bridge, the narrow flow paths through the village, as shown in Figure 4-4.

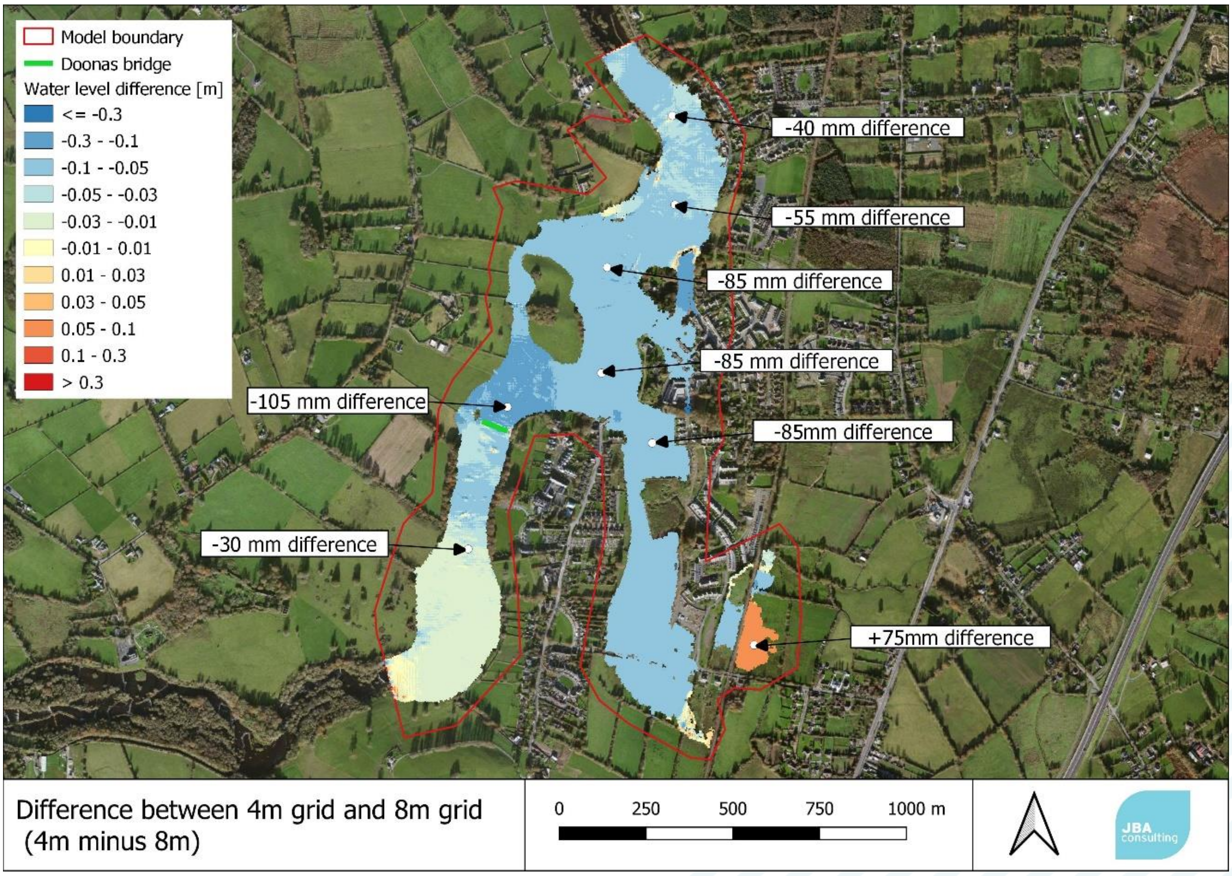


Figure 4-2: Difference between 4m and 8m scenario (4m minus 8m scenario)

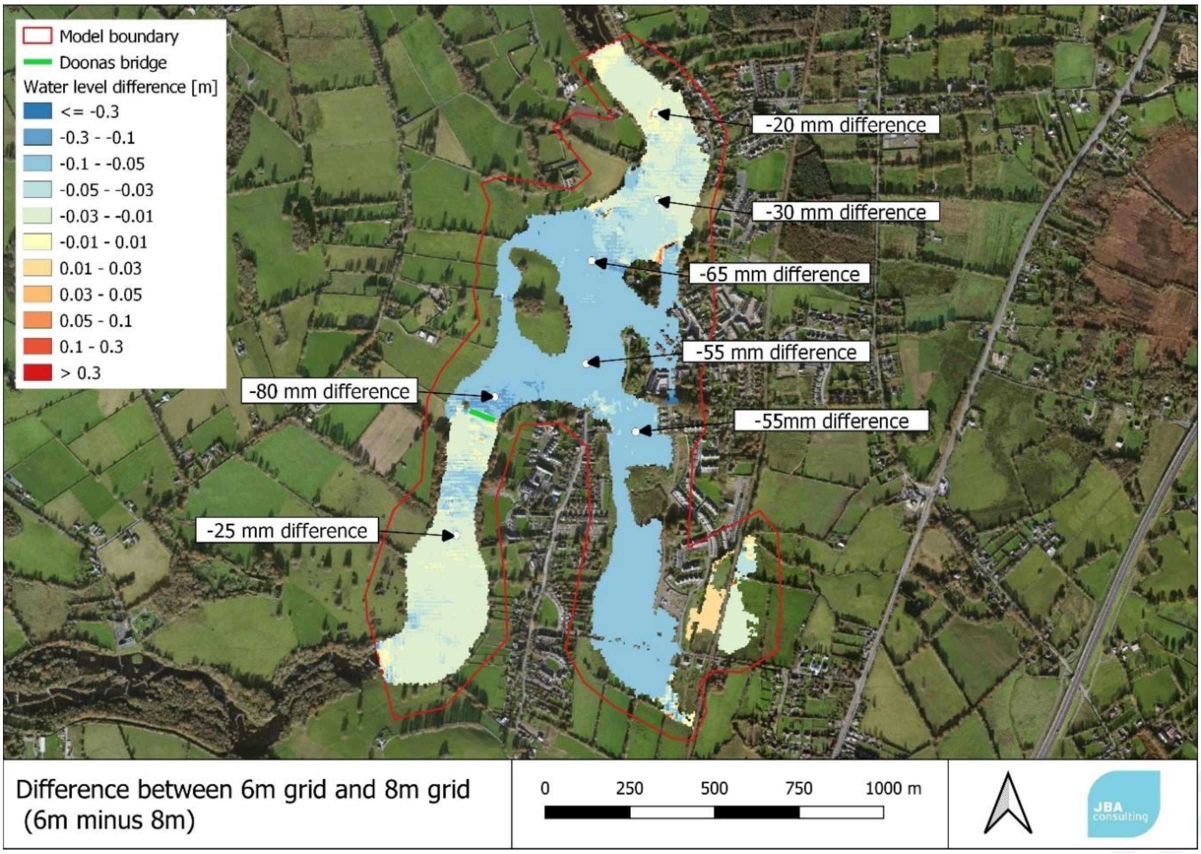


Figure 4-3: Difference between 6m and 8m scenario (6m minus 8m scenario)

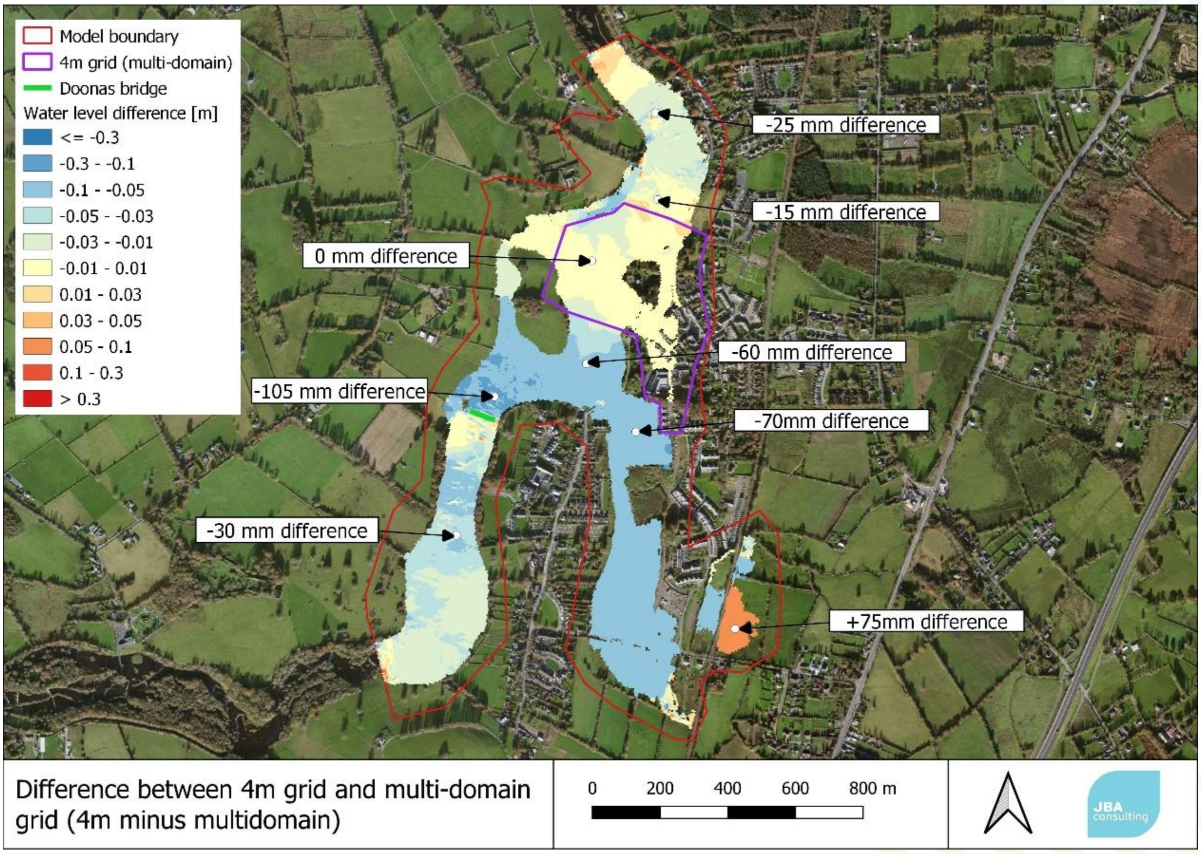


Figure 4-4 Difference between 4m and multi-domain scenario (4m minus multi-domain scenario)

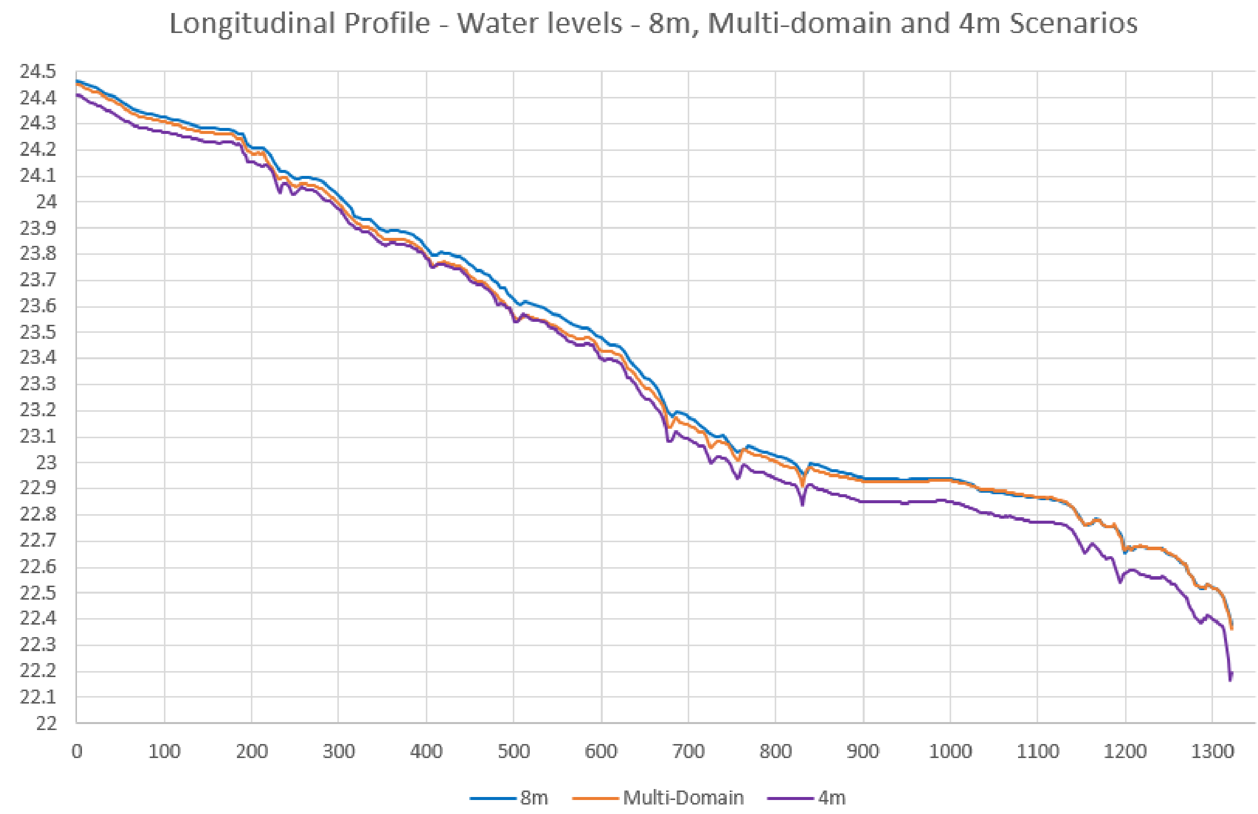


Figure 4-5:Longitudinal profile – Upstream of Doonass Bridge

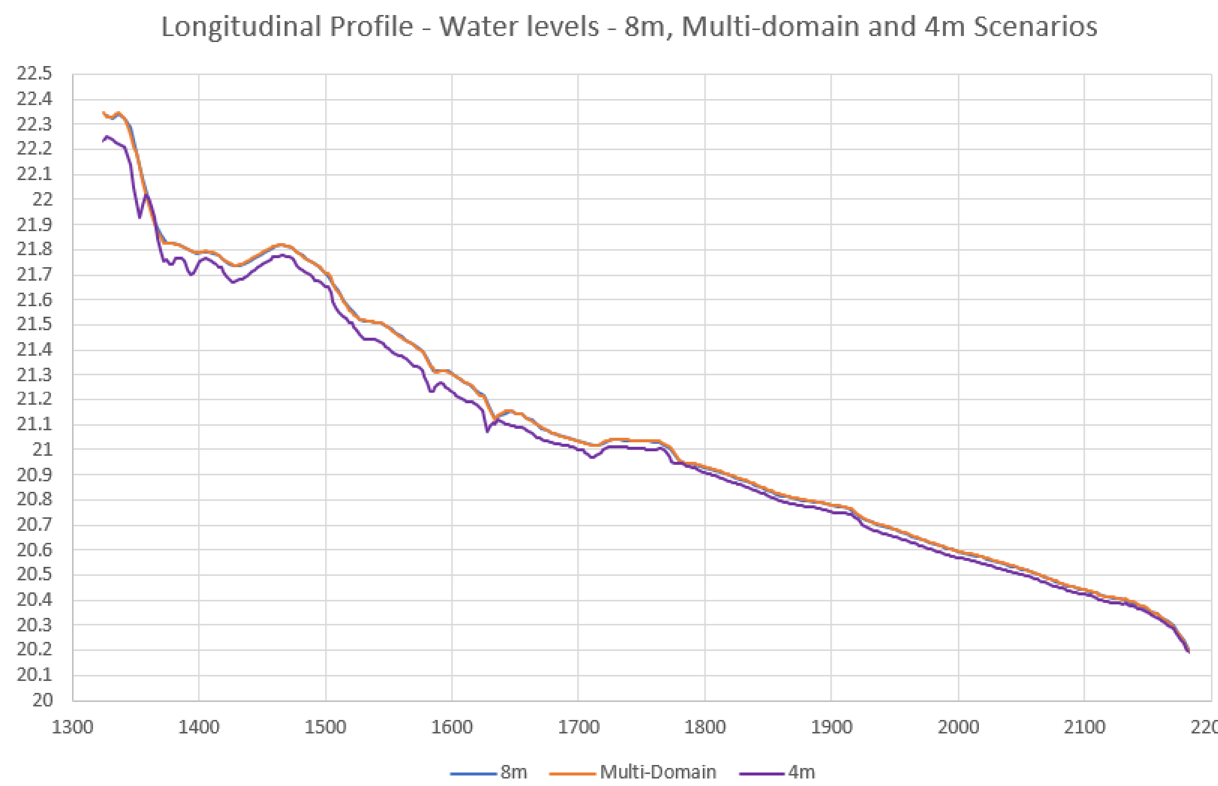


Figure 4-6 Longitudinal profile – Downstream of Doonass Bridge

The maps and the longitudinal profiles show the following:

- the water levels in the 4m scenario are always lower (up to 100mm) than the 8m, 6m and multidomain scenarios;
- the water levels in the multidomain scenario are in between 4m and 8m from upstream until Doonass Bridge (chainage 1300m in the longitudinal profile), then are identical with 8m.

The multidomain was chosen for the design runs because it is a better representation of the narrow flow paths through the village than the 8m scenario and it produces more conservative water levels than the 4m scenario.

4.3 Doonass Bridge

In this test the bridge was removed from the model and the results were compared to the extents where the bridge is in place, with no blockage applied, during the 1% AEP "504" event (504 m^3/s on the Shannon River downstream of Parteen Weir).

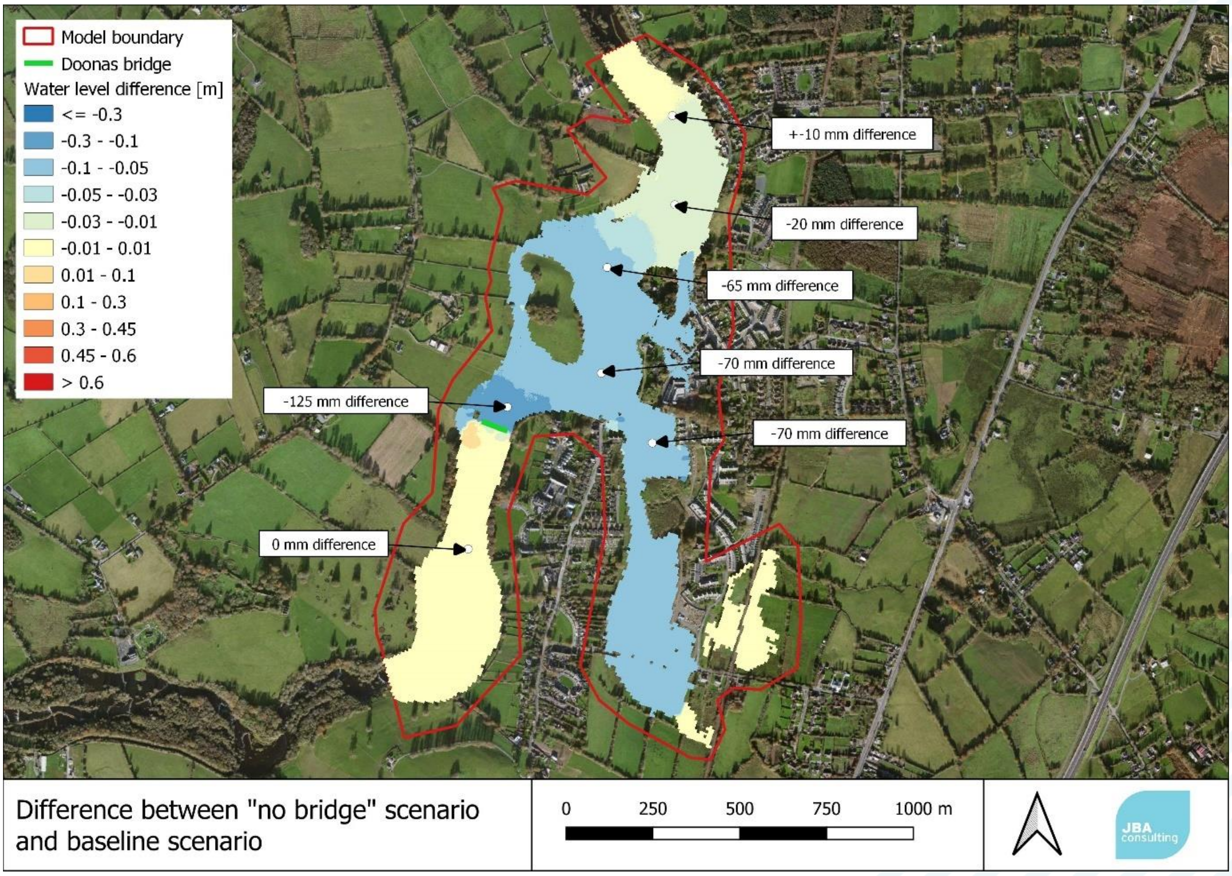


Figure 4-7: Difference between "No bridge" scenario and baseline scenario – "504" Event ("no bridge" scenario minus baseline scenario)

The differences are rather small because the water level does not reach the soffit level, therefore the Doonass Bridge obstruction is represented by the piers solely.

Doonass Bridge blockage

The model was tested with the following blockages applied to the Doonass Bridge during the 1% AEP "504" Event (504 m3/s on the Shannon River downstream of Parteen Weir):

- 33% (flow area of the structure is reduced by 33%);
- 67% (flow area of the structure is reduced by 67%);
- 67% on height (the first 2 thirds of the structure starting from the riverbed to the soffit are completely blocked);
- 100% blockage of the left half of the bridge.

The results are presented in the figures below.

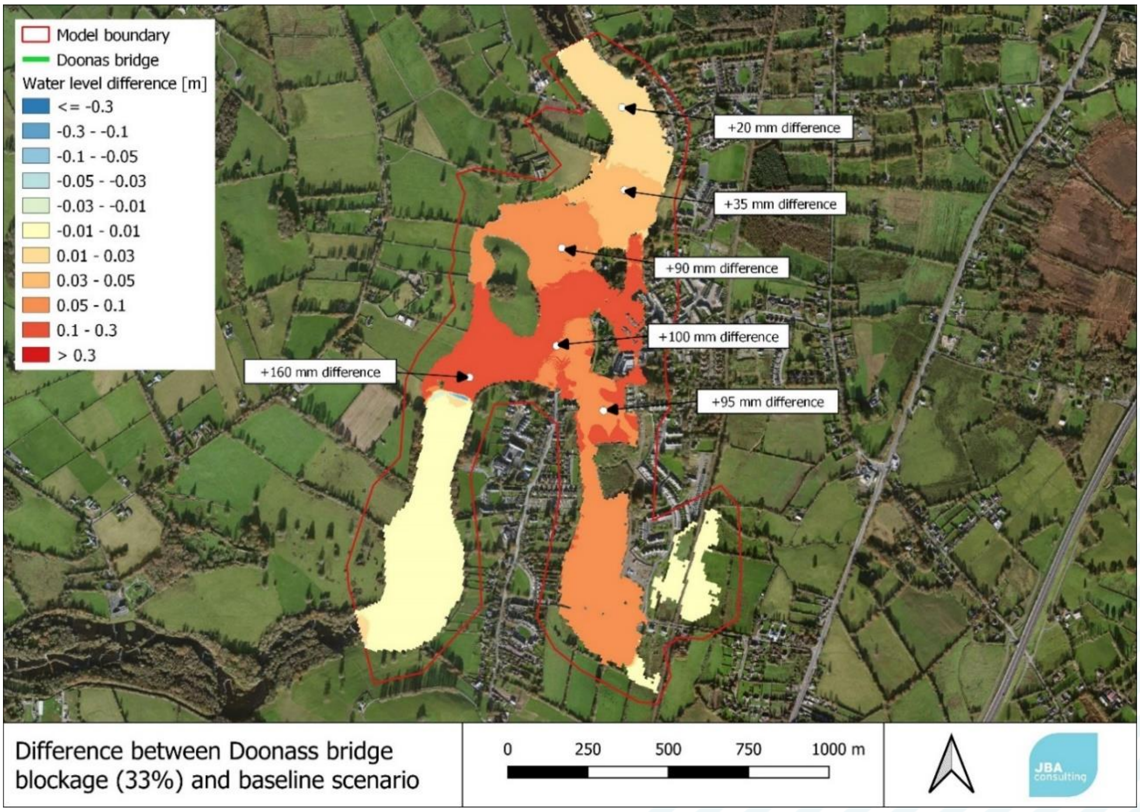


Figure 4-8: Difference between blockage scenario (33%) and baseline scenario (blockage scenario – baseline scenario)

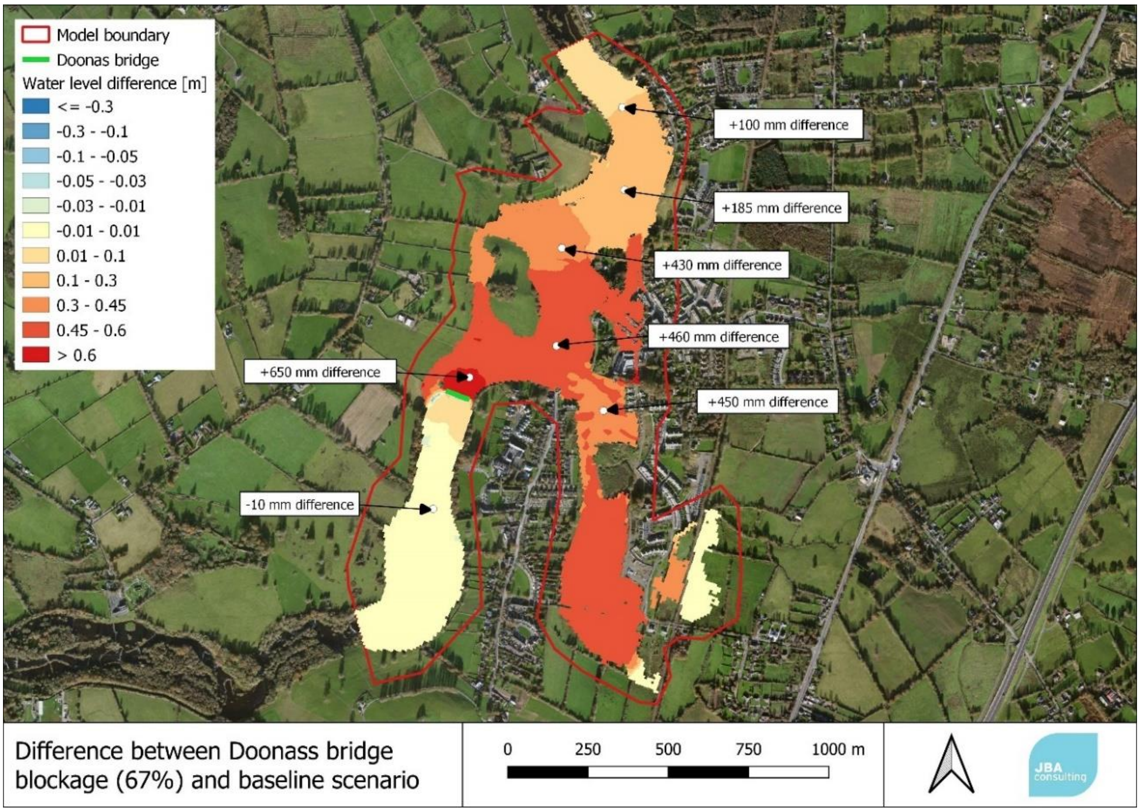


Figure 4-9: Difference between blockage scenario (67%) and baseline scenario (blockage scenario – baseline scenario)

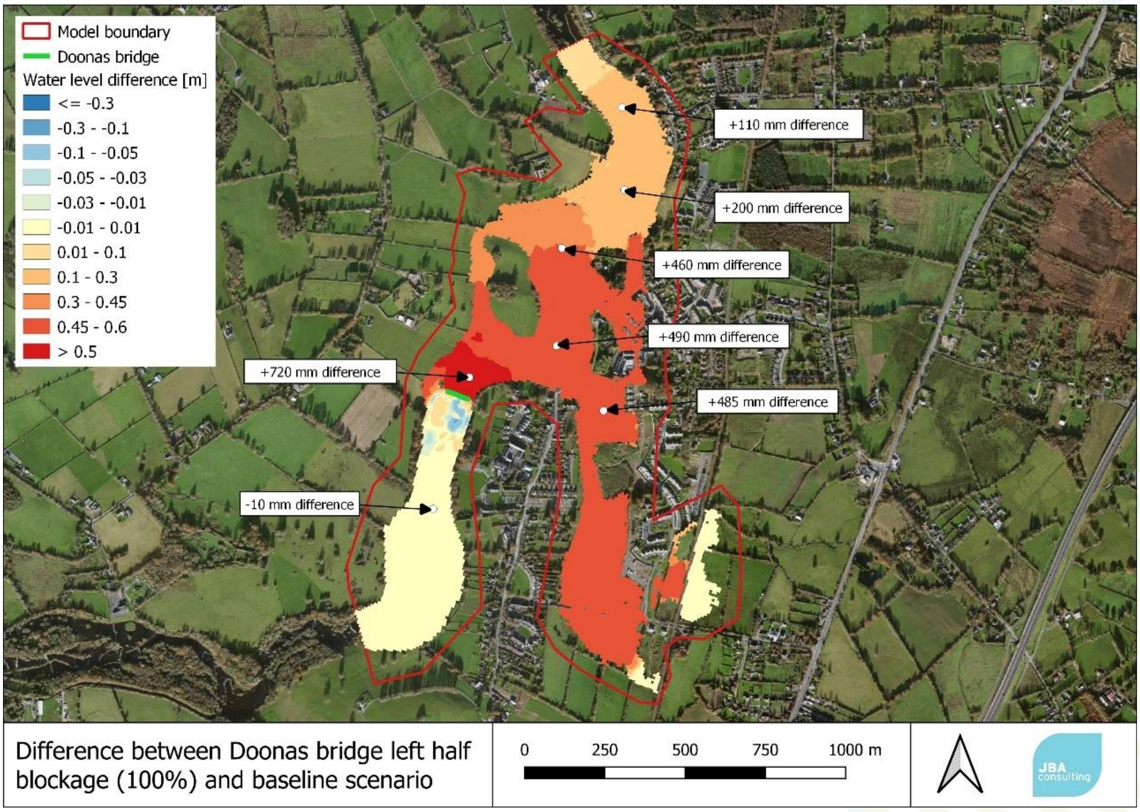


Figure 4-10: Difference between bridge blockage (left half, 100%) and baseline scenario (blockage scenario – baseline scenario)

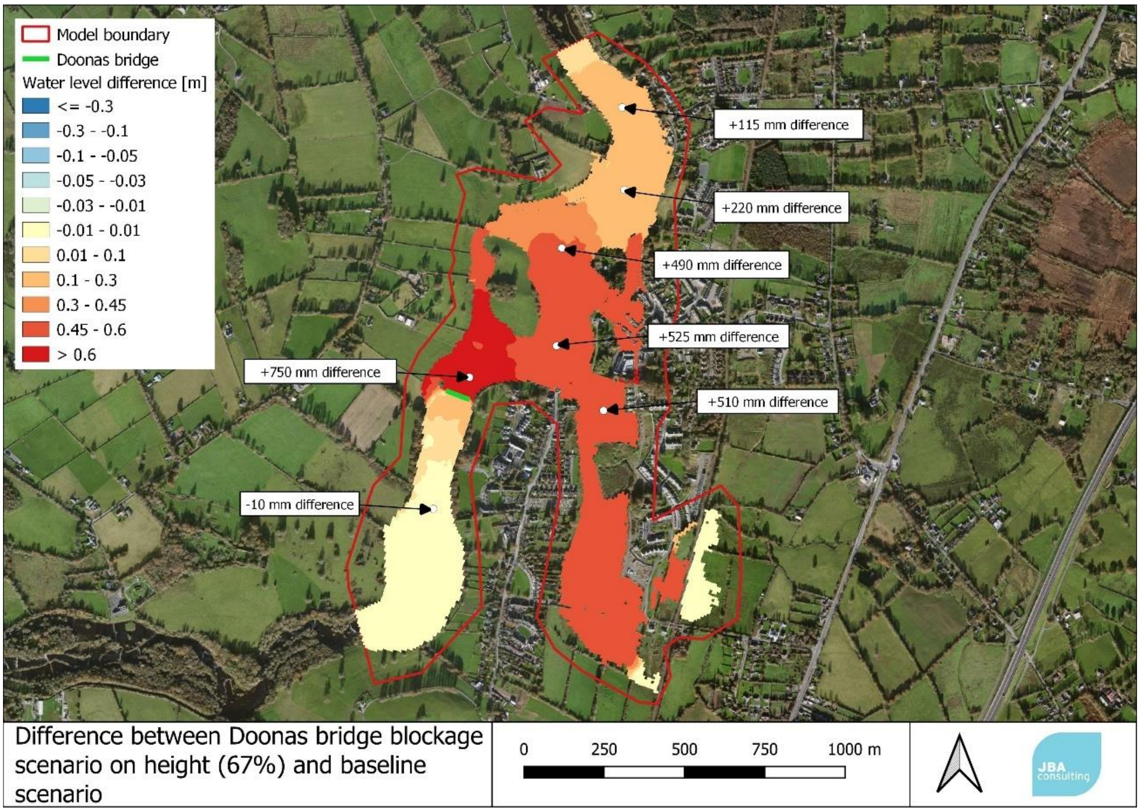


Figure 4-11: Difference between bridge blockage on height (67%) and baseline scenario (blockage scenario – baseline scenario)

4.4 Eel bridge

The Eel bridge was tested against 2 extreme scenarios in respect to flood risk:

- Worst case scenario - total blockage of the culverts
- Best case scenario - complete opening within the structure at the culverts

The figure below shows the water level differences between the 2 extreme scenarios. The water level changes only locally in case of any of the 2 scenarios.

Therefore, the Eel bridge has a minor hydraulic impact for the 1% AEP and it was modelled using the worst-case scenario (100% blockage). This approach is supported by onsite observations during large floods, namely the bridge getting blocked by fallen trees.

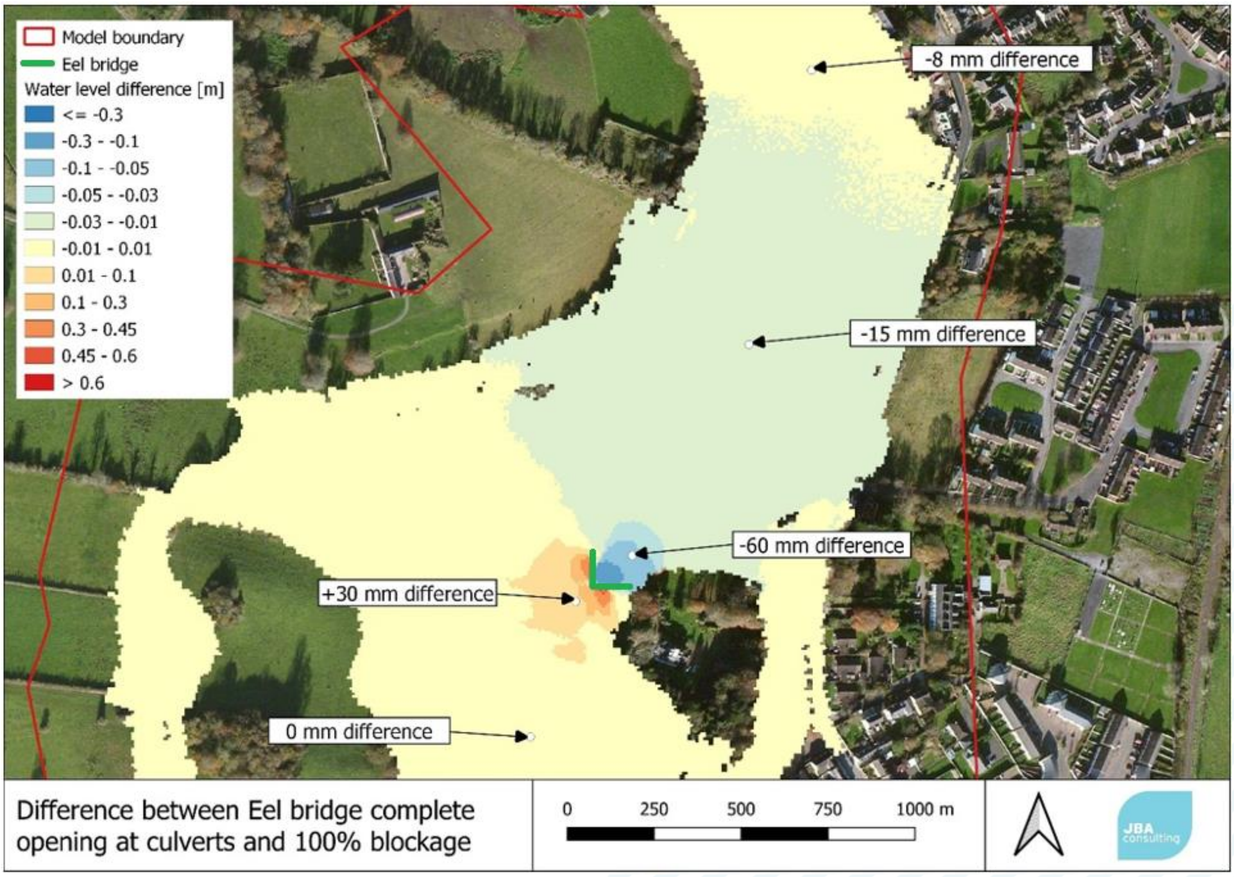


Figure 4-12: Difference between Eel bridge failure scenario and bridge blockage (100%) scenario (failure scenario minus blockage scenario)

4.5 Island House structure

The model was run with the flood gates both open and closed.

The comparison between the gates open and gates closed scenarios shows the water levels are higher by up to 150mm in the Island House channel and by 50mm within the main village when the gates are open. It should be noted that parts of the Island House channel are heavily overgrown.

The water levels are slightly reduced upstream by having the gates open.

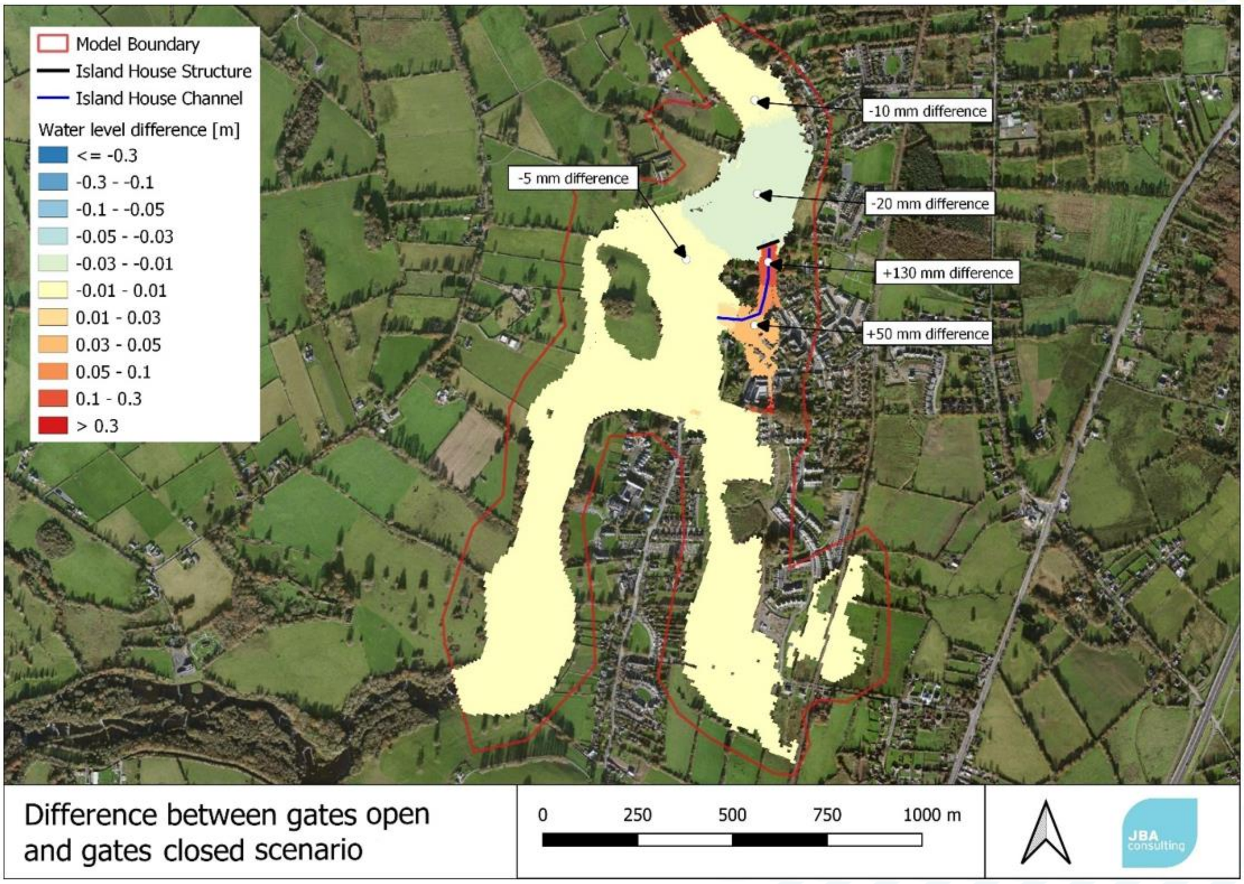


Figure 4-13: Difference between flood gates open and flood gates closed scenarios (flood gates open scenario minus flood gates closed scenario)

4.6 High and low Manning's n values

The Manning's n surface roughness applied to the model (see Section 2.4.5) was increased and decreased by 20%, as presented in Table 4-1 below.

The increase in water level is approximatively 200mm when a higher roughness value is applied and, conversely, the water level decreases by approximatively 200mm when the lower roughness coefficients are applied to the model.

The evolution of the islands has not been modelled as they are considered stable. Vegetation is managed by Castleconnell Fisheries Association to avoid any stands of bankside vegetation encroaching into the river. Standard sensitivity tests around Manning's n assessment of the islands have been undertaken, as outlined above.

Table 4-1: Manning's n values used in the sensitivity tests

Material	Baseline values	High n (+20%)	Low n (-20%)
General Rural	0.045	0.054	0.036
Riverbed	0.045	0.054	0.036
Dense vegetation	0.110	0.132	0.088
Medium vegetation	0.080	0.096	0.064
General Urban	0.060	0.072	0.048

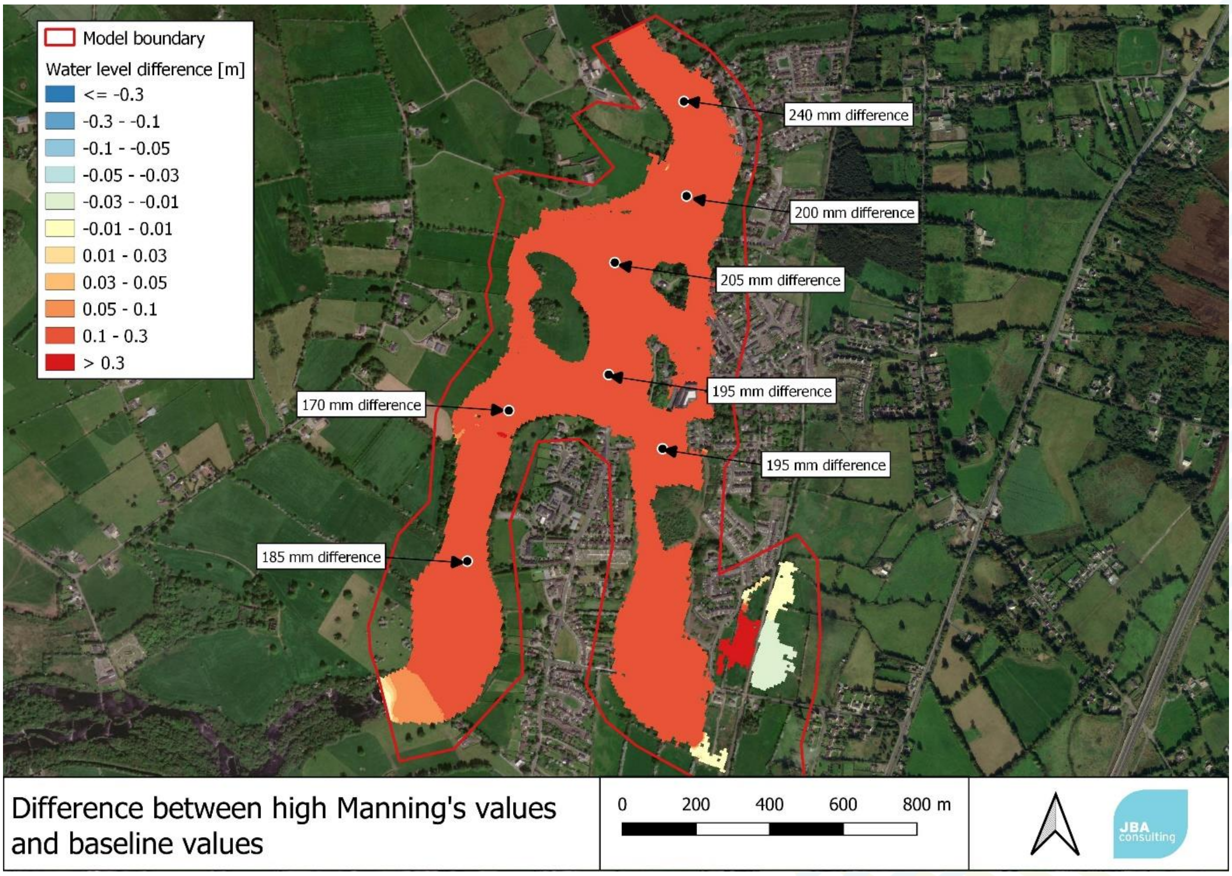


Figure 4-14: Difference between high Manning's n values and baseline scenario (high Manning's minus baseline values)

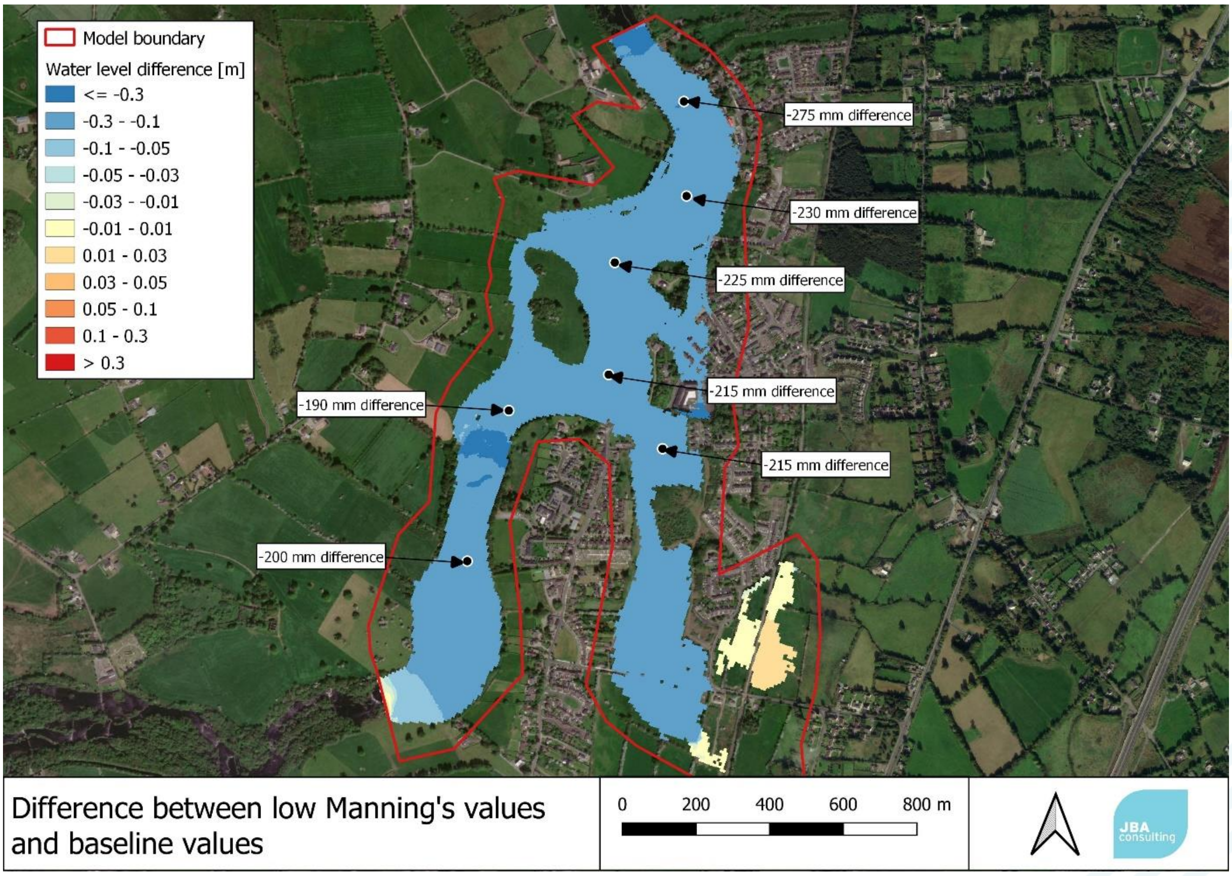


Figure 4-15: Difference between low Manning's n values and baseline scenario (low Manning's minus baseline values)

4.7 Culvert Blockage

A blockage scenario was run on the model in order to estimate a 67% blockage at the culvert under the railway line on the Stradbally Stream.

In the eventuality of a blockage at the culvert, the water level increases by 80mm upstream of the railway line. Blockage of the next culvert downstream was not tested as the flow would continue to expand out into the floodplain to the south, and little difference would be expected.

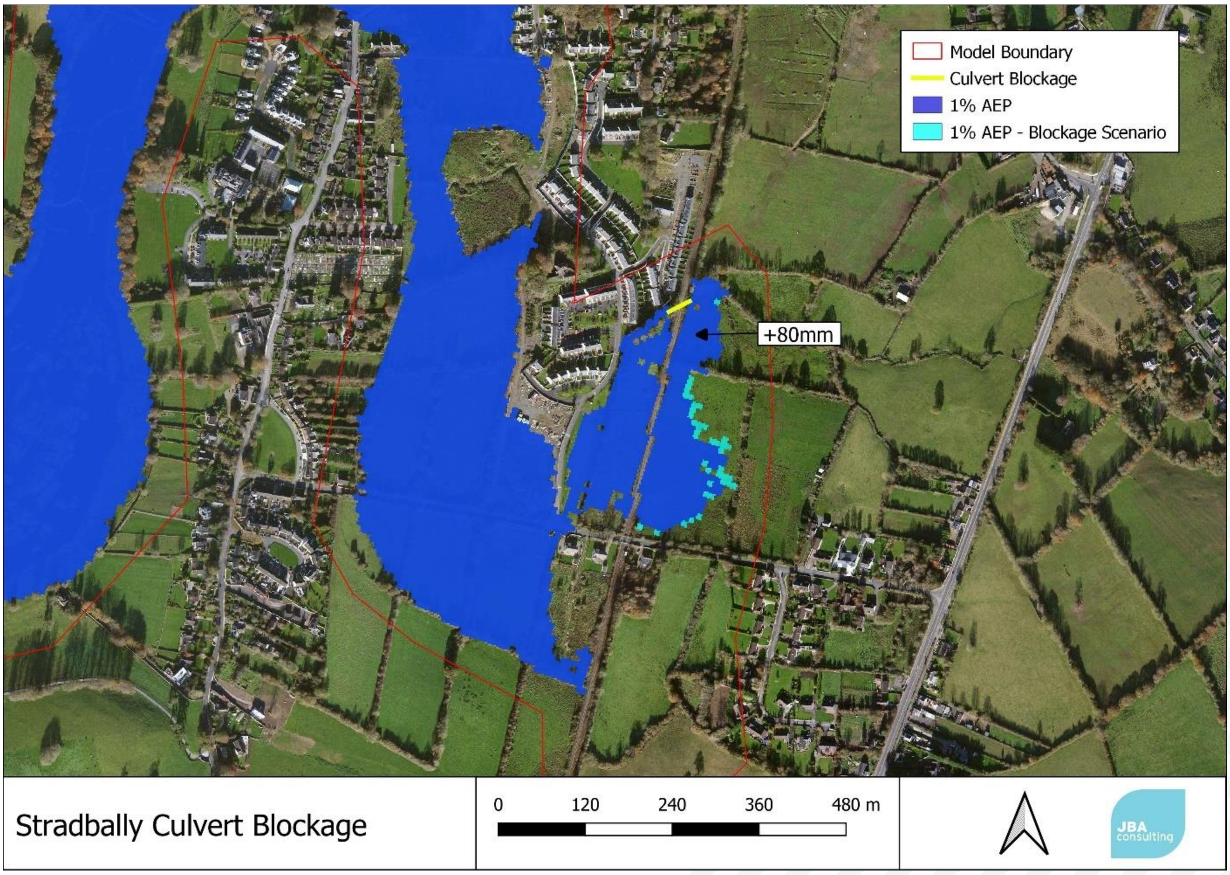


Figure 4-16: Stradbally Stream – Blockage Scenario

5 Model Runs

The model runs undertaken are presented in the Table 5.1. The flow used to represent different operational conditions are named by the rate of headrace flow to Ardnacrusha.

The main assumptions used in the model runs are as follows:

The flood gates at the Island House structure are considered open, being the more conservative scenario for the main at-risk properties. A 67% blockage was applied on the left side of Doonass Bridge (24m – approximately 5 opes) and the walls along the Elvers Rd are not included.

According to the hydrological Joint Probability assessment in the Hydrology Report, the 1% AEP is composed of the following inflows:

- 1% AEP on Shannon
- 5% AEP on Kilmastulla
- 5% AEP on Black River
- 1% AEP on Stradbally
- 1% AEP Cedarwood

Flow estimates for this HEP 25_3886_1 are strongly influenced by the assumptions relating to the operation of turbines and spillway at Ardnacrusha.

During flood events under "standard operational conditions", we have assumed four turbines are in operation and $345m^3/s$ is regulated to the turbines. The headrace flow assumption of $345m^3/s$ is based upon previous estimates in the Shannon CFRAM studies as informed by the ESB. The operational conditions of the power station were discussed in a meeting held between JBA, ESB, OPW and LCCC on 22/04/20. In this meeting the ESB advised that in high flow conditions, $345m^3/s$ can be delivered down the head race to the power station, but a number of factors should be taken into account and this is not a fixed quantity and could be lower. With this assumed head race flow a **"504" Event** was established for the River Old River Shannon at the HEP downstream of Parteen Weir (HEP ref 25_3886_1), with a 1% AEP peak flow of 504 m3/s. This flow is similar in scale to that experienced in the 2009 flood event.

For the purpose of the design of the Castleconnell FRS, an allowance has been made for operational conditions at Ardnacrusha that could, within reasonable contemplation, occur. In the event of one turbine being out of operation for maintenance or as a result of a mechanical failure, ¾ of the $345m^3/s$ ($258\ m^3/s$) has been assumed to continue down the head race and the rest, ¼ ($87m^3/s$) would pass over Parteen Weir into the Old River Shannon. In a planned situation, a spillway can be opened at Ardnacrusha and the flows along the canal maintained. However, as the spillway is not automatic, in an unplanned situation it cannot pass the full flow immediately. Therefore, a reduced flow down the head race must be considered in the design of the scheme. This scenario was discussed with ESB and based on their past operational experience the design team adopted a suite of operational conditions to define the potential uncertainties within the design flow. Extended turbine maintenance has been necessary during previous flood seasons , in February/March 2020 for example, where one turbine was out of commission during the 2022 winter season. This supports why the design team has had to consider the headrace inflow quantum carefully in selecting the design flow in the Old River Shannon.

These limitations in operational conditions outlined above will result in greater discharge passing over the weir at Parteen into the River Shannon resulting in a 1% AEP peak flow of $591\ m^3/s$. This is adopted as the **Baseline Design Event** for the River Shannon at the HEP downstream of Parteen Weir (HEP ref 25_3886_1).

This approach has been adopted to ensure that appropriate contingency is accommodated in the design of the flood relief scheme to afford a high level of flood protection to Castleconnell Village and the scheme area, allowing for limitations in operational conditions at the power station.

These determine the peak flow estimates for the Old Shannon downstream of Parteen Weir and have been tested in the hydraulic model. These scenarios are referred to by the amount of flow regulated in the headrace to the Ardnacrusha power station. Table 5-1 and Table 5-3 present the design event flows for each of the operating and climate scenarios.

To give context to this, the 2009 event experienced in Castleconnell was approximately the 1% AEP peak (504m^3/s) which occurred during "standard operational conditions" at Ardnacrusha.

Table 5-1: 1% AEP Old Shannon flow downstream of Parteen Weir (HEP ref: 25_3886_1) based on operational conditions at Ardnacrusha

Flow to Ardnacrusha (m^3/s)	1% AEP peak flow Old Shannon (m^3/s)	Name and description of operational conditions at Ardnacrusha
345	504	**Standard operational conditions ("504" Event)** The 504 m^3/s flow is comparable to the 2009 flood event peak flow in the Old Shannon. This is the residual flow after ~345 m^3/s is regulated to the headrace from the 1% AEP total Shannon upstream of Parteen Weir. All turbines in operation, previous scale of inflow to Ardnacrusha during flood conditions without any operational limitations.
258	591	**Limitations in operational conditions (Baseline Design Event)** Addresses operational uncertainty and represents possible situations such as: - 1 turbine down with Ardnacrusha spillway not in operation, or - 2 turbines down with spillway in operation - reduced inflow along headrace as a result of wind set up conditions increasing the hydraulic gradient along the canal or reduced throughput at the station due to high tide levels at the outfall
0	849	**Complete Outage** Assumes complete operational failure of Ardnacrusha or head race system with the total Shannon upstream of Parteen Weir (HEP ref: 25075) passing over Parteen Weir

For the **Climate Change** scenarios, the increase in flow (10% for the MRFS and 30% for the HEFS) is applied to the Total Shannon flow upstream of Parteen Weir (HEP ref: 25075) and then the Ardnacrusha headrace flow is deducted. This estimate assumes there is no natural or artificial change in the routing, attenuation or alteration of operating procedures in response to climate change impacts upstream.

Table 5-2. Old Shannon flows downstream of Parteen Weir (HEP ref 25_3886_1)

	HEP and scenario peak flow estimates [m³/s]		
Name of scenario	"504" event (standard operational conditions)	Baseline design event (limitations in operational conditions)	Complete Outage
Headrace flow to Ardnacrusha [m³/s]	345	258	0
50%	181.9	**268.9**	526.9
20%	281.3	**368.3**	626.3
10%	340.5	**427.5**	685.5
5%	393.5	**480.5**	738.5
2%	458.2	**545.2**	803.2
1%	**504.4**	**591.4**	**849.4**
0.5%	549	**636**	894
0.1%	648.5	**735.5**	993.5

Table 5-3. Old Shannon flows downstream of Parteen Weir (HEP ref 25_3886_1) - Climate change scenarios

	HEP and scenario peak flow estimates [m³/s]					
Name of scenario	"504" event (standard operational conditions)			Baseline design event (limitations in operational conditions)		
Headrace flow to Ardnacrusha [m³/s]	345			258		
Climate scenario	Present day	MRFS	HEFS	Present day	MRFS	HEFS
50%	181.9	287.3	340.0	268.9	374.3	427
20%	281.3	406.6	469.2	368.3	493.6	556.2
10%	340.5	477.6	546.2	427.5	564.6	633.2
5%	393.5	541.2	615.1	480.5	628.2	702.1
2%	458.2	618.8	699.2	545.2	705.8	786.2
1%	**504.4**	**674.3**	**759.2**	**591.4**	**761.3**	**846.2**
0.5%	549	727.8	817.2	636	814.8	904.2
0.1%	648.5	847.2	946.6	735.5	934.2	1033.6

The Complete Outage scenario was not incorporated into the climate change considerations, as it was deemed excessively extreme to be evaluated under potential climate change impacts.

The model assumptions for the "504" event and the baseline design event are presented in the Table 5-4.

Table 5-4: Model assumptions for the "504" event and the Baseline Design Event

	"504" event 345 m3/s headrace flow	Baseline design event 258 m3/s headrace flow
Doonass Bridge Blockage	67% blockage on the left side (24m, roughly 5 opes)	67% blockage on the left side (24m, roughly 5 opes)
Sluices at Island House bridge	Open	Open
Castellations at Island House Bridge	Included	Included
Walls on the Elvers Rd	Not included	Not included
Grid Orientation	Aligned with the Doonass Bridge	Aligned with the Doonass Bridge
Eel Bridge openings	Closed	Closed

The flood extent corresponding to the 1% AEP "504" event (345 m^3/s headrace flow) is presented in below and the 1% AEP for the Baseline Design Event (258 m^3/s headrace flow) is displayed in .

Modelled water levels at different locations across the study area are listed in Table 5-5, while the corresponding results for 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.1% AEP in both the "504" event (345 m^3/s headrace flow) and Baseline Design Event (258 m^3/s headrace flow) scenarios are presented in Appendix B.

Figure 5-1: 1% AEP for "504" event (345 m^3/s headrace flow)

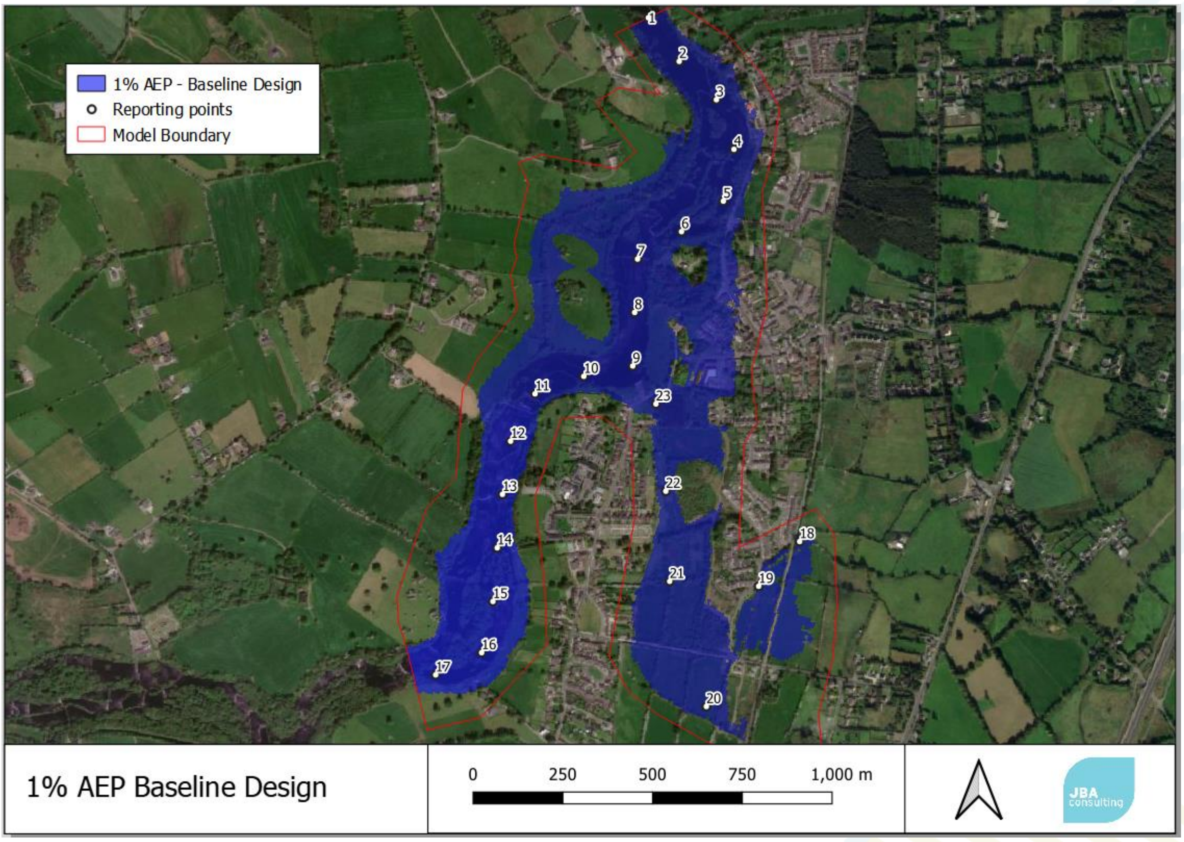


Figure 5-2: 1% AEP Baseline design event (258 m^3/s headrace flow)

Table 5-5: Shannon and Stradbally Watercourses - Modelled Water Levels (baseline design event levels in bold)

	"504" event (standard operational conditions) [mOD] 345 m^3/s headrace flow				Baseline design event (limitations in operational conditions) [mOD] 258 m3/s headrace flow				Complete Outage [mOD] Zero headrace flow
Reporting Location	1% AEP Present Day	0.1% AEP Present Day	1% AEP MRFS	1% AEP HEFS	1% AEP Present day	0.1% AEP Present Day	1% AEP MRFS	1% AEP HEFS	1% AEP Present Day
1	24.59	25.03	25.11	25.34	**24.86**	25.26	25.34	25.54	25.53
2	24.24	24.64	24.71	24.92	**24.48**	24.85	24.92	25.10	25.09
3	24.09	24.48	24.55	24.76	**24.33**	24.69	24.76	24.94	24.93
4	23.93	24.31	24.38	24.59	**24.17**	24.53	24.59	24.77	24.76
5	23.70	24.08	24.16	24.39	**23.94**	24.33	24.39	24.57	24.56
6	23.53	23.92	23.99	24.22	**23.77**	24.15	24.22	24.39	24.39
7	23.24	23.68	23.76	24.01	**23.51**	23.94	24.01	24.20	24.19
8	23.14	23.57	23.65	23.91	**23.40**	23.83	23.91	24.09	24.09
9	23.14	23.56	23.65	23.90	**23.40**	23.83	23.90	24.09	24.08
10	22.94	23.36	23.44	23.70	**23.19**	23.62	23.70	23.88	23.87
11	22.75	23.16	23.25	23.52	**23.01**	23.44	23.51	23.69	23.68
12	21.72	22.13	22.22	22.43	**21.97**	22.36	22.43	22.63	22.62
13	21.19	21.59	21.66	21.87	**21.43**	21.80	21.87	22.06	22.05
14	20.99	21.40	21.47	21.68	**21.24**	21.61	21.68	21.87	21.86
15	20.75	21.17	21.25	21.46	**21.01**	21.39	21.46	21.66	21.65
16	20.61	21.03	21.11	21.33	**20.87**	21.26	21.33	21.53	21.52
17	20.37	20.79	20.87	21.08	**20.63**	21.01	21.08	21.27	21.26
18	23.89	23.91	23.91	23.91	**23.89**	23.91	23.91	24.09	24.08
19	22.87	23.56	23.65	23.90	**23.20**	23.83	23.90	24.09	24.08
20	23.14	23.56	23.65	23.90	**23.40**	23.83	23.90	24.09	24.08
21	23.14	23.56	23.65	23.90	**23.40**	23.83	23.90	24.09	24.08
22	23.14	23.57	23.65	23.90	**23.40**	23.83	23.90	24.09	24.08
23	23.14	23.56	23.64	23.90	**23.40**	23.83	23.90	24.09	24.08

6 Model Performance

This section summarises the general performance of the Shannon hydraulic model.

6.1 Timestep and model run time

Model timestep

- Model runs were run in double precision with the following timesteps:
- a 2 second 1D FM and ESTRY timestep and a 4 second 2D TUFLOW timestep for the 8m grid;
- a 1 second 1D FM and ESTRY timestep and a 2 second 2D TUFLOW timestep for the 4m grid.

Model run time:

- approximately 11 hours for a 1000-hour simulation for the final model (multi-domain);
- approximately 4 hours for a 1000-hour simulation in the 8m grid scenario;
- approximately 22 hours for a 1000-hour simulation in the 4m grid scenario.

PC specification:

- Intel Core i7-8700 CPU @ 3.2 GHz Processor
- 16 GB RAM memory

6.2 Model stability

6.2.1 1D Flood Modeller stability

The 1D model is stable with no points of non-convergence and the convergence of flow through the model is good.

Flood Modeller used 3 iterations per timestep and the peak mass error recorded is 0.02%, respectively -0.12%. Refer to Figure 6-1 and for the graphic representation of the 1D model stability for Shannon and Cedarwood models.

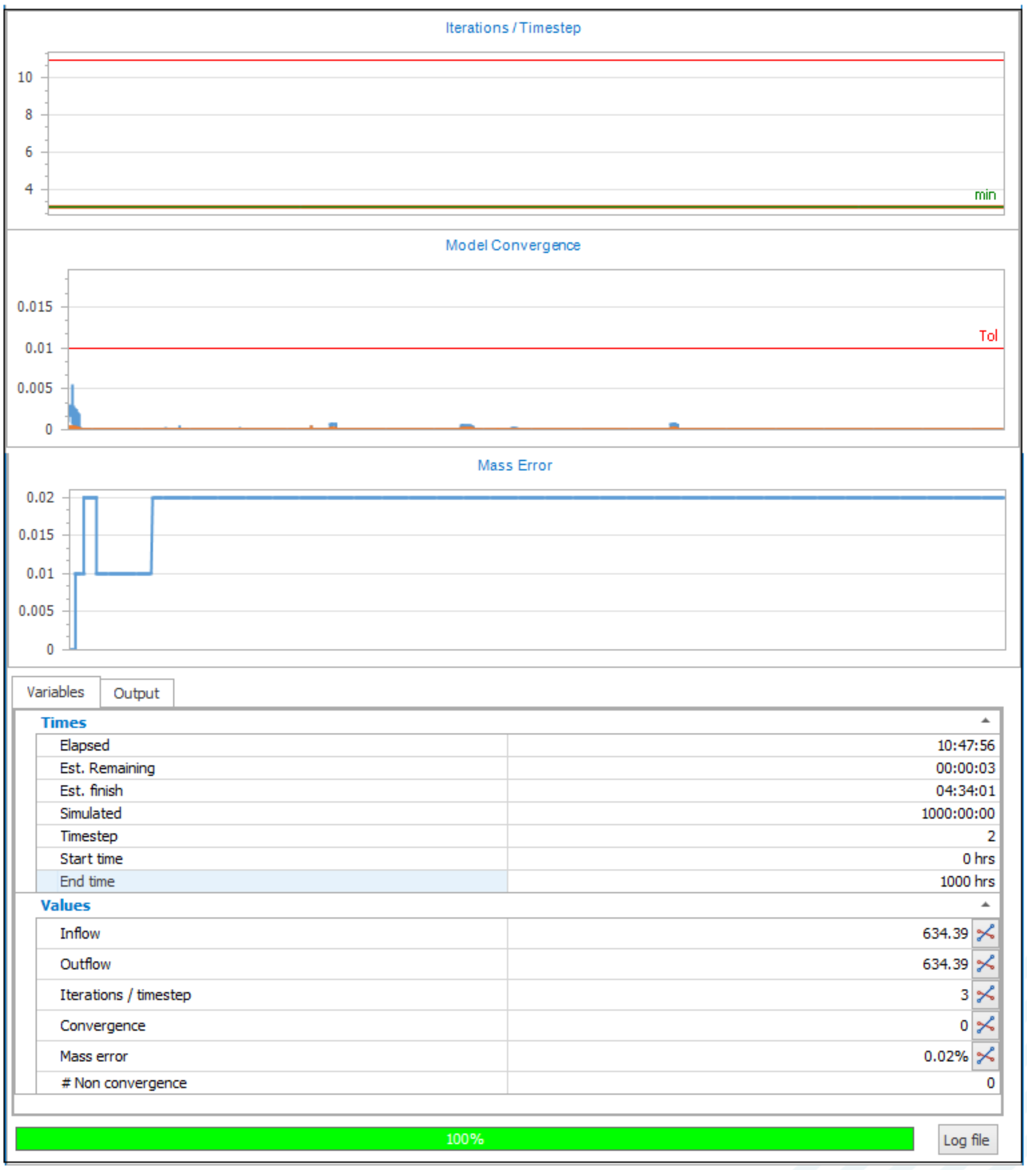


Figure 6-1: Flood Modeller stability plots – Shannon Model

6.2.2 2D Tuflow stability

Within the 2D domain, to ensure that the model is stable and performing adequately, three main factors are examined:

- Checks and warnings recorded;
- Number of negative depths;
- Mass balance error (MBE).

6.2.2.1 Checks and warnings

- WARNING 2073 – Object ignored. Only Points, Lines, Polylines & Regions used. GIS Object = Null Shape
- This warning occurs when there is an object without a geometry in a GIS layer. It does not affect model performance.
- WARNING 2117 – Inactive 2D cells made active by 2D SX link

- The 2D cells at the edge of the code boundary became active in order to create the link with the 1D model. The connection is working as intended.

6.2.2.2 Negative depths

No negative depths were recorded during the model runs in Shannon and Cedarwood models.

6.2.2.3 Mass balance error (MBE)

The highest MBE value recorded is 0.24% in the Shannon Model at 3.99h and 1.12% in the Cedarwood model at 0.00h.

```
Start Time (h): 0.
End Time (h):   1000.
Computational Steps (based on largest 2D timestep): 899999
CPU Time:       10:39:42  [10.66 h]
Clock Time:     10:48:11  [10.8 h]

Simulation FINISHED

Classic 1D Negative Depths: 0
Classic 2D Negative Depths: 0

Peak Flow In (m3/s):    513.0 at Time 863.51
Peak Flow Out (m3/s):   512.1 at Time 864.06
Volume at Start (m3):   354317
Volume at End (m3):     1269332
Total Volume In (m3):   1425208455
Total Volume Out (m3):  1424039258
Volume Error (m3):      -254183 or -0.0% of Volume In + Out
Final Cumulative ME:    -0.01%

                                   Whole Simulation          Qi+Qo > 5%
Peak +ve dV (m3):                   86.1 at  119.62h        86.1 at  119.62h
Peak -ve dV (m3):                  -11.9 at  961.27h       -11.9 at  961.27h
Peak ddV over one timestep:         12.2 at  960.01h        12.2 at  960.01h
Peak ddV as a % of peak dV:        14.2%                   14.2%
Peak Cumulative ME:                0.24% at    3.99h       0.24% at    4.00h
```

Figure 6-2: Tuflow Simulation Summary – Shannon Model

7 Pluvial Flood Risk

As part of the exercise in defining flood risk, an assessment on the extent of potential risk due to pluvial flood flows was undertaken. This involved assessing the stormwater network for varying flood events and joint probability events to gauge whether there was a flood risk due to the existing networks capacity, the relationship between the network and the downstream fluvial receptors or both.

In total, six outfall locations and their associated networks were assessed as part of the model testing. These areas were then grouped into three areas namely, North, Central and South.

The tests that were conducted involved the following conditions:

- Free Outfalls, No pumping – This tested the network's capacity to cater for its own catchment
- Surcharged Outfalls, no pumping – This tested the consequence of the network not being able to discharge to the Shannon or the Cedarwood Stream due to river levels surcharging the network
- Blocked outfalls, pumping and overflow intervention – These tested installations of pluvial mitigation measures to gauge the quantum of intervention required during a surcharged event

Further to this, a number of joint probability combinations were tested to capture the reality of a concurrent pluvial event occurring during the critical (1% AEP) event in the Shannon. The results of these are presented in Figure 7-1, 7-2 and 7-3.

Figure 7-1 Pluvial Risk Areas - North

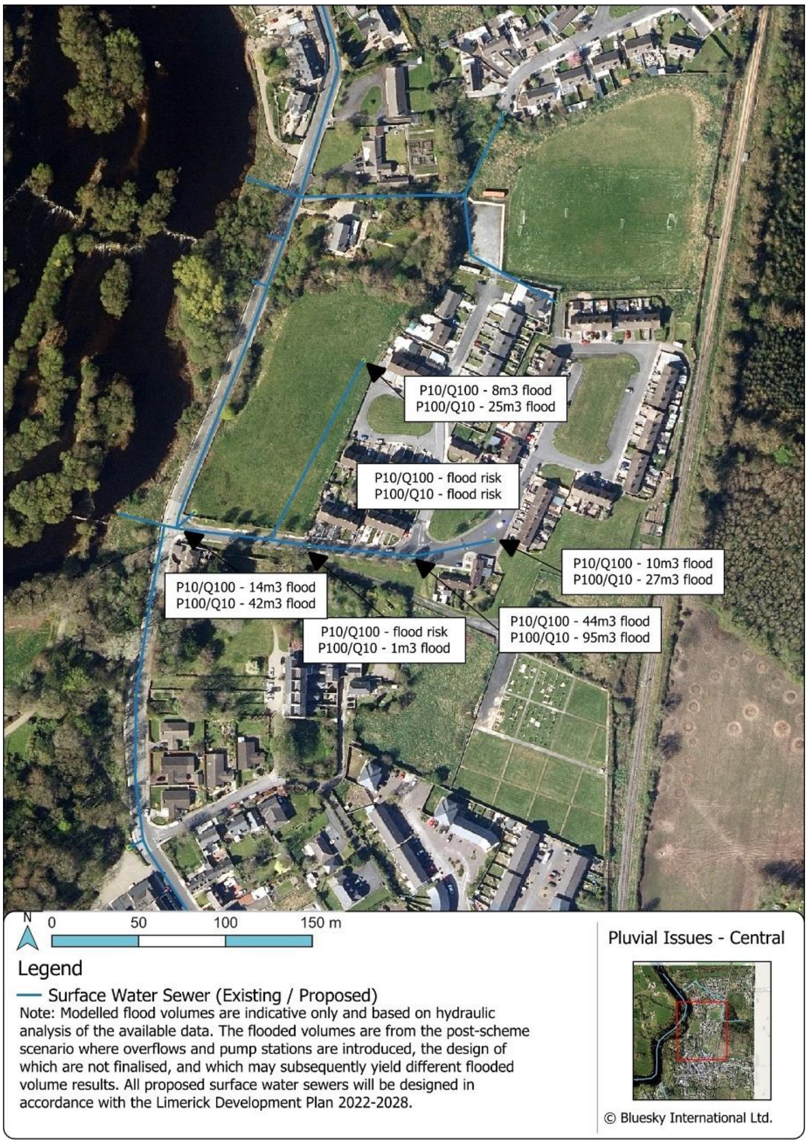


Figure 7-2 Pluvial Risk locations - Central

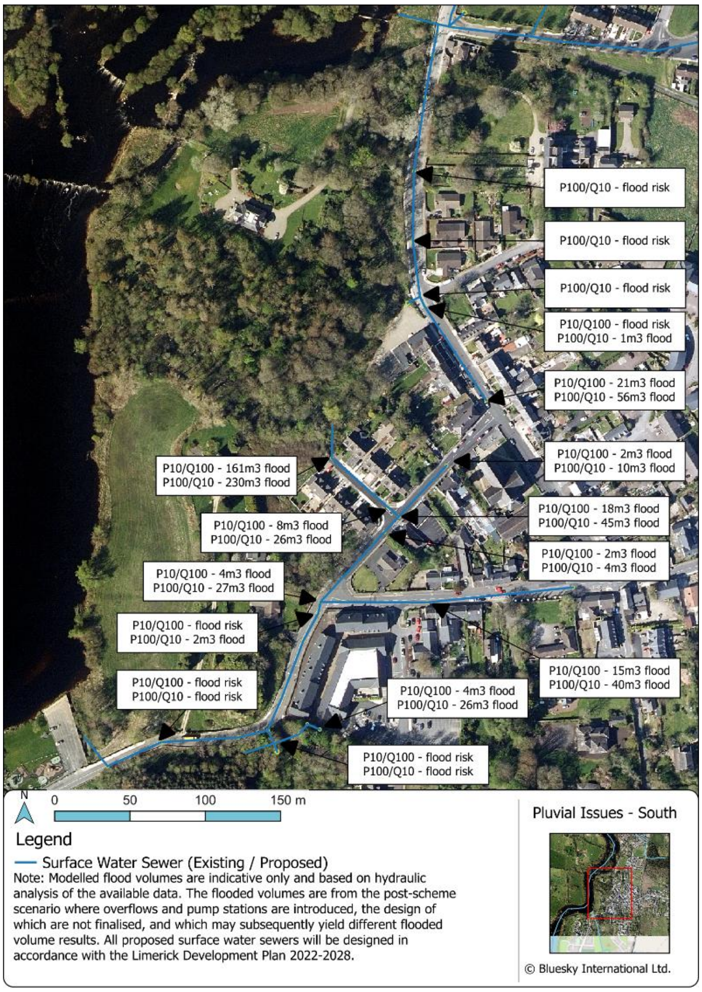


Figure 7-3 Pluvial Risk Areas - South

7.1 Pluvial Summary

Running the model for all available storm up to 100-year with no flood event in the Shannon shows there are existing capacity issues on the networks. Part of this is down to inadequate pipe sizes but also the limits on the available records we have. There is a need therefore, to develop further these datasets in the detailed design stage.

Blocking the outfalls (i.e. a flood event) shows extensive flooding from the networks in the order of 1000's of m3 of flooding. These are assessed against the levels in the Shannon post-installation of fluvial measures to ensure the worst-case downstream conditions are considered.

Introducing two new pump stations (initially tested at Scanlon Park and at Maher's Pub) is seen to reduce the flooding volumes but does not solve the flood risk in its entirety. This is part due to existing capacity issues in the numerous stormwater networks where upstream nodes still exceed capacity and result in overland flows. Where previously these would, in some cases, be able to discharge into the Shannon, the flow paths will be now blocked by the fluvial defences. Thus, a solution is needed to cater for these overland flows.

Due to the lack of available datasets, the intended interventions are not yet fully defined, and further data collection is required to define these measures.

8 Summary

The Hydraulics Report outlines the construction and schematisation process of the hydraulic model for Castleconnell and its surrounding area, used in the development of the Castleconnell Flood Relief Scheme (FRS). The goal of the FRS project is to design and implement flood defences that will safeguard properties and essential infrastructure against future flood events, with a standard of protection up to a 1% Annual Exceedance Probability (AEP).

The River Shannon, heavily influenced by the Parteen Weir and Lough Derg, is the main source of flood flows at Castleconnell. Other contributing fluvial sources include the Kilmastulla River, Black River, Cedarwood Stream and Stradbally Stream. The Shannon River channel through the study area is characterised by high hydraulic complexity, with many islands, pools and weirs present along the channel, influencing the hydraulic regime. Given this complexity, a 2-dimensional (2D) model was constructed to directly represent the hydraulic behaviour of these features, with additional topographical and river survey commissioned for this purpose.

The Cedarwood Stream, a significantly smaller watercourse than the Shannon River, has been shown by CFRAM modelling to be a source of flood risk to Castleconnell. A separate 1D-2D FM-Tuflow model was built for Cedarwood Stream.

The hydraulic model was developed using Flood Modeller and TUFLOW software packages, creating a linked model with 1D and 2D components. It utilises three topographic data sets: cross sections of the River Shannon, 2m resolution LiDAR, and a Digital Terrain Model (DTM) derived from survey points collected in 2020. The final model adopted a multi-domain 2D model, with upstream and downstream reaches at an 8m resolution. An area around Island House, sensitive to cell resolution due to the complexity of in-channel features, was modelled at a 4m resolution.

The hydraulic model for Castleconnell area was calibrated and validated using significant flood events from November 2009 and February 2020. Priority was given to the 2020 event due to the availability of reliable and distributed flood marks across the area.

The 2020 event, which happened during the project's timeline, allowed for real-time data collection of water levels and flows. The 2009 event, notable for its high magnitude, was used for model validation due to the availability of observed flood extents and water levels at different locations. The model was found to calibrate well against the 2020 event, with most points falling within a range of +/- 100mm and a few within an acceptable range of +/- 200mm. The validation of the model using the 2009 event, which recorded a peak flow of 500 m3/s over the Parteen Weir (slightly less than the 1% AEP event), demonstrated a good correlation with the observed flood patterns during that event.

The model was also compared with the CFRAM model to understand the impact of estimations in the CFRAM model, which overestimated an island and riverbed elevation downstream of Castleconnell, causing increased water levels upstream. After replacing CFRAM estimations with recent survey data, the model showed significantly improved water level estimations.

Sensitivity analyses were carried out to test the model results based on uncertainty in a number of hydraulic input parameters. This included grid orientation, model resolution, bridge assessments, Manning’s n values and culvert blockages.

Finally, the design runs considered operational conditions at Ardnacrusha, taking into account situations where one turbine might be out of operation for maintenance or due to mechanical failure. This comprehensive analysis ensured the design of the scheme accommodates appropriate contingencies, providing a high level of flood protection to Castleconnell Village and the scheme area.

Appendices

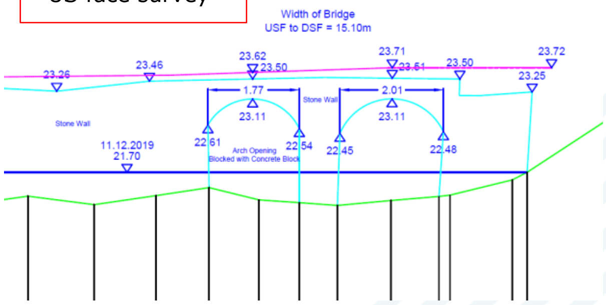
Appendix A – Model Structures

A1 - West Channel Structures

CC00071 – Arch culvert			
Width	2.01m	**Length**	15.1m
Soffit	23.11mOD	**Height**	2.05m
Type	Irregular culvert (type I) linked to a height-width table	**Overtopping**	Modelled in 1D (minimum height 23.60mOD)
Notes	The left opening is blocked with a concrete block and it was not included in the model		

US face

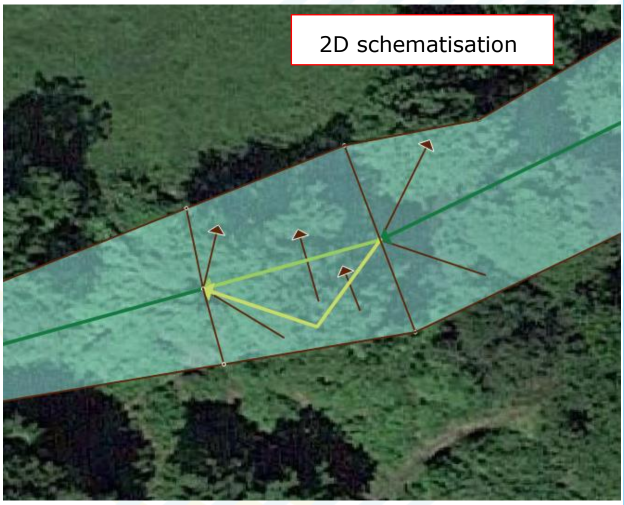
US face survey



Height - width table

H	W
0	0
0.17	1.97
1.39	2.01
1.71	1.6
1.97	0.83
2.05	0.01

2D schematisation

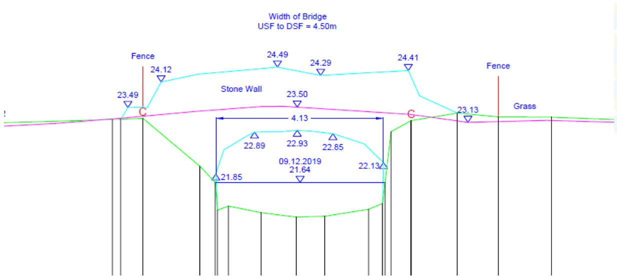


CC00067 – Arch bridge

Width	4.13m	**Length**	4.5m
Soffit	23.89mOD	**Height**	2.14m
Type	Arch bridge (type BB) linked to a height-width table	**Overtopping**	Modelled in 1D (minimum height 23.28mOD)

DS face

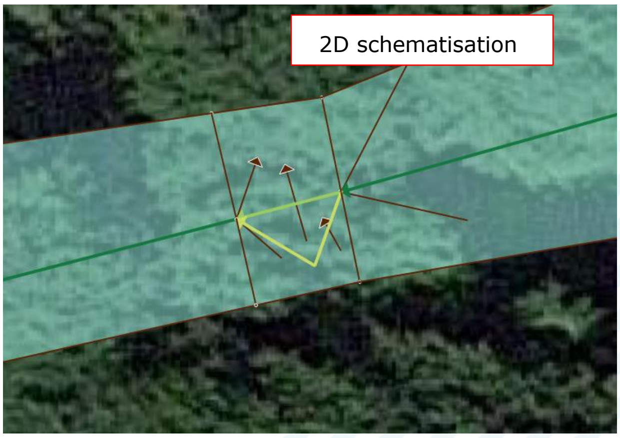
US face survey



Height - width table

H	W
0	0
0.19	3.45
0.34	4.13
1.35	4.04
2.06	1.93
2.14	0.01

2D schematisation

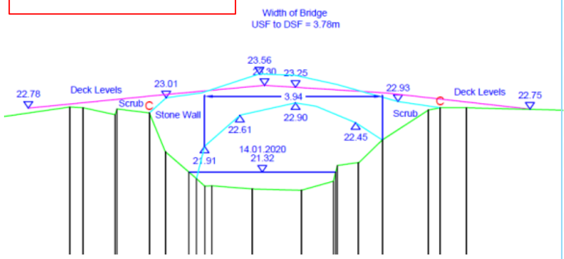


CC00026 – Arch bridge			
Width	3.94m	**Length**	3.78m

Soffit	22.90mOD	**Height**	1.99m
Type	Arch bridge (type BB) linked to a height-width table	**Overtopping**	Modelled in 1D (minimum height 22.80mOD)

DS face

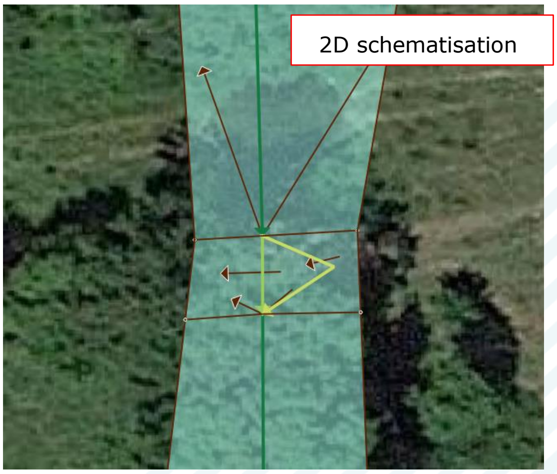
US face survey



Height - width table

H	W
0	0
0.1	2.14
0.57	3.05
0.62	3.51
1.15	3.94
1.54	2.57
1.91	0.8
1.99	0.01

2D schematisation

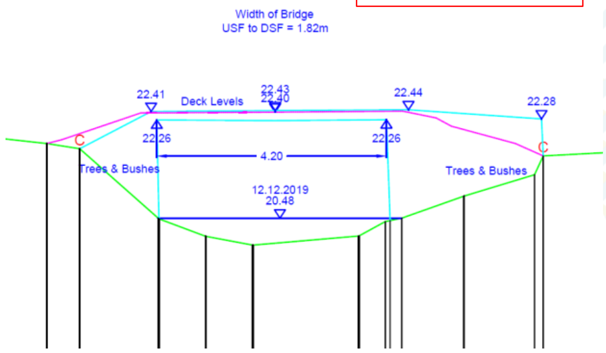


CC00009 – Bridge			
Width	4.2m	**Length**	1.82m

Soffit	22.26mOD	**Height**	2.27m
Type	Bridge (type BB) linked to a height-width table	**Overtopping**	Modelled in 1D (minimum height 21.67mOD)

DS face

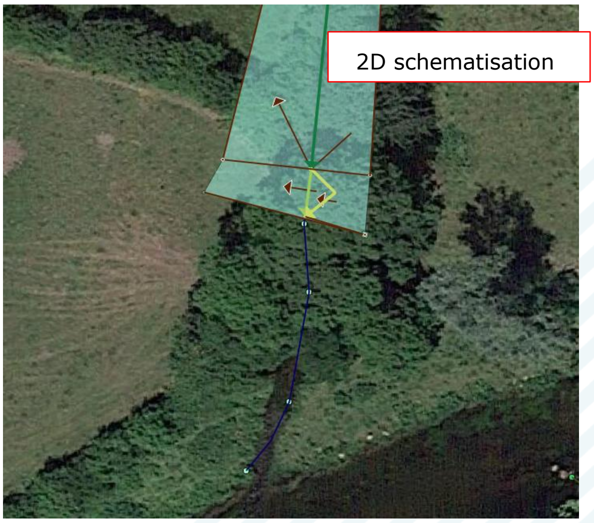
US face survey



Height - width table

H	W
0	0
0.16	2.8
0.48	4.2
2.27	4.2

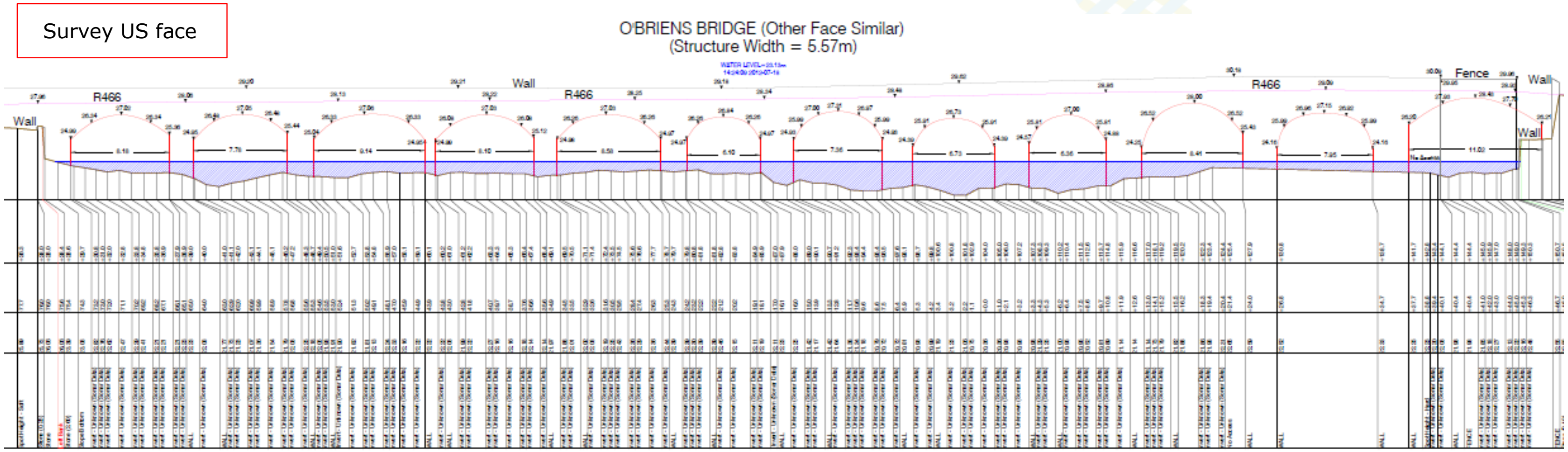
2D schematisation



A2 - Shannon 1D model

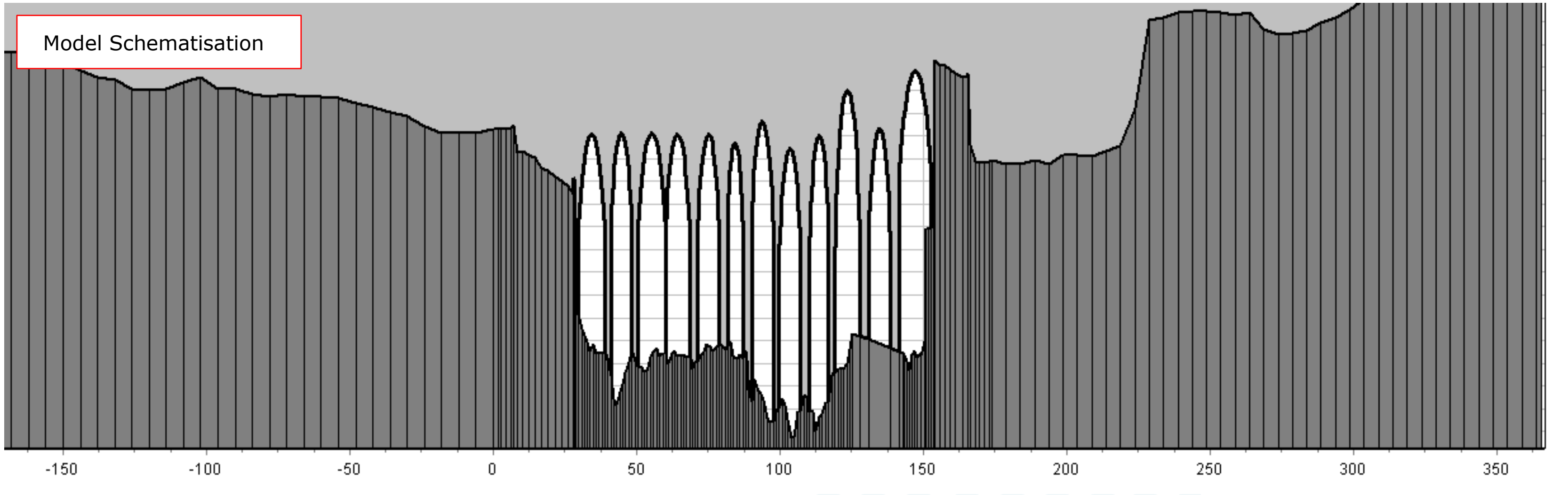
15LSH02270bu – Arch bridge (12 openings)			
Structure Width	124m	**Length**	5.57m
Soffit	min: 26.73 mOD max: 28.43 mOD	**Springing height**	min: 24.39 mOD max: 26.20mOD
Opening height	min: 4.63m max: 6.72m	**Invert level**	20.37 mOD

Survey US face



US face

Model Schematisation



Appendix B – Hydraulic Results

Table B-1: Shannon and Stradbally Watercourses Modelled Water Levels – "504" event - 345 m³/s headrace flow

Reporting location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
1	23.27	23.74	23.98	24.21	24.44	24.59	24.73	25.03
2	23.05	23.46	23.69	23.90	24.11	24.24	24.37	24.64
3	22.85	23.29	23.52	23.74	23.95	24.09	24.21	24.48
4	22.70	23.13	23.37	23.59	23.80	23.93	24.05	24.31
5	22.42	22.88	23.13	23.35	23.57	23.70	23.83	24.08
6	22.24	22.70	22.95	23.18	23.40	23.53	23.66	23.92
7	21.84	22.35	22.62	22.86	23.10	23.24	23.38	23.68
8	21.76	22.26	22.52	22.76	22.99	23.14	23.28	23.57
9	21.76	22.26	22.52	22.76	22.99	23.14	23.28	23.56
10	21.62	22.09	22.34	22.57	22.79	22.94	23.07	23.36
11	21.47	21.93	22.15	22.39	22.61	22.75	22.89	23.16
12	20.54	20.95	21.15	21.35	21.58	21.72	21.85	22.13
13	20.02	20.43	20.64	20.84	21.05	21.19	21.32	21.59
14	19.79	20.22	20.43	20.64	20.86	20.99	21.12	21.40
15	19.50	19.94	20.16	20.38	20.61	20.75	20.89	21.17
16	19.32	19.79	20.01	20.24	20.47	20.61	20.75	21.03
17	19.44	19.55	19.77	20.00	20.24	20.37	20.51	20.79
18	23.84	23.86	23.87	23.88	23.89	23.89	23.89	23.91
19	23.45*	23.46*	22.86	22.87	22.87	22.87	22.87	23.56
20	22.88	22.88	22.89	22.89	22.99	23.14	23.28	23.56
21	21.92	22.26	22.51	22.76	22.99	23.14	23.28	23.56
22	21.82	22.26	22.51	22.76	23.00	23.14	23.28	23.57
23	21.77	22.25	22.51	22.76	23.00	23.14	23.28	23.56

*No flooding occurs at this location, therefore the peak water levels were extracted at the culvert inlet (refer to Figure 2-26 - culvert 2), which is located 100m upstream of reporting location number 19.

Table B-2: Shannon and Stradbally Watercourses Modelled Water Levels – Baseline Design Event - 258 m^3/s headrace flow

Reporting location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
1	23.69	24.11	24.34	24.52	24.72	24.86	24.99	25.26
2	23.41	23.81	24.01	24.19	24.37	24.48	24.61	24.85
3	23.24	23.64	23.86	24.03	24.21	24.33	24.45	24.69
4	23.08	23.49	23.70	23.87	24.05	24.17	24.28	24.53
5	22.83	23.25	23.47	23.64	23.82	23.94	24.05	24.33
6	22.65	23.08	23.30	23.47	23.66	23.77	23.89	24.15
7	22.30	22.76	22.99	23.18	23.38	23.51	23.64	23.94
8	22.21	22.66	22.89	23.07	23.27	23.40	23.53	23.83
9	22.21	22.66	22.89	23.07	23.27	23.40	23.53	23.83
10	22.05	22.47	22.69	22.87	23.07	23.19	23.32	23.62
11	21.90	22.29	22.51	22.69	22.88	23.01	23.13	23.44
12	20.91	21.26	21.48	21.65	21.85	21.97	22.10	22.36
13	20.39	20.75	20.96	21.13	21.32	21.43	21.56	21.80
14	20.17	20.55	20.76	20.93	21.12	21.24	21.36	21.61
15	19.89	20.29	20.51	20.69	20.89	21.01	21.14	21.39
16	19.73	20.14	20.37	20.55	20.75	20.87	21.00	21.26
17	19.50	19.91	20.13	20.31	20.51	20.63	20.76	21.01
18	23.84	23.86	23.87	23.88	23.89	23.89	23.89	23.91
19	23.45*	23.46*	22.86	22.87	22.87	23.20	23.53	23.83
20	22.88	22.88	22.90	23.07	23.27	23.40	23.53	23.83
21	22.21	22.66	22.89	23.07	23.27	23.40	23.53	23.83
22	22.21	22.66	22.89	23.08	23.27	23.40	23.53	23.83
23	22.21	22.66	22.89	23.09	23.27	23.40	23.53	23.83

*No flooding occurs at this location, therefore the peak water levels were extracted at the culvert inlet (refer to Figure 2-26 - culvert 2), which is located 100m upstream of reporting location number 19.

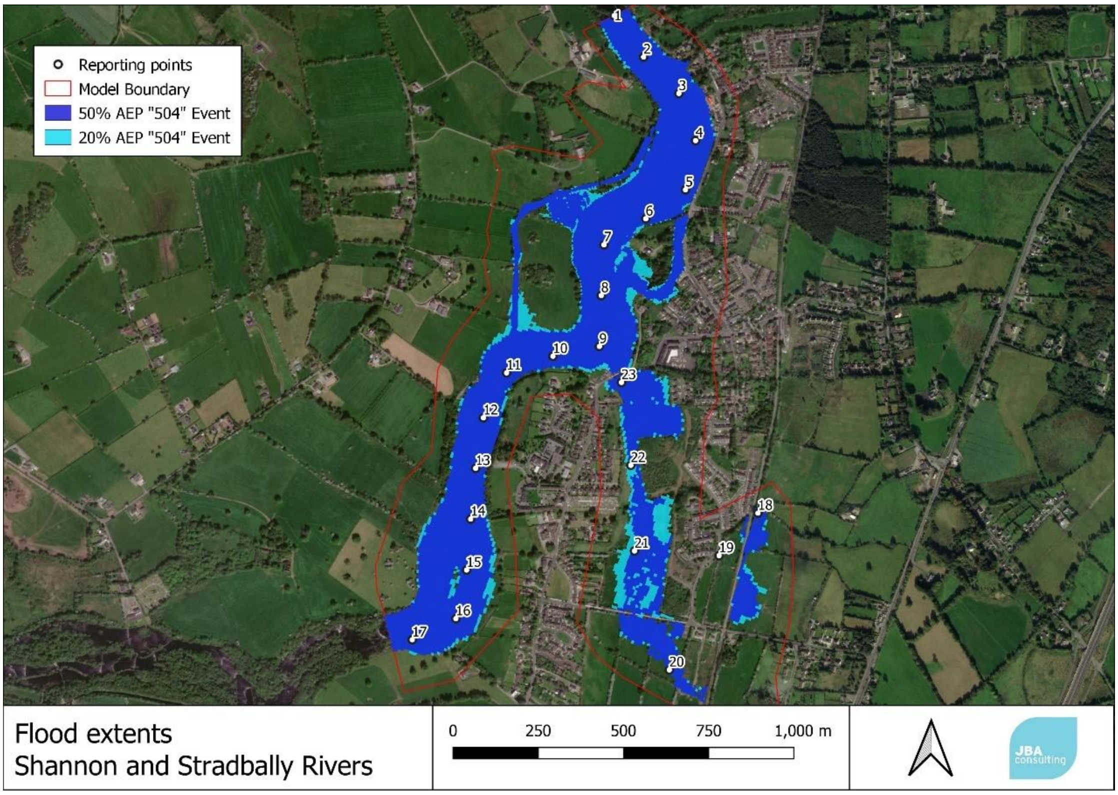


Figure B-1: Shannon and Stradbally Watercourses Flood Extents – 50% and 20% AEP "504" event (345 m³/s headrace flow)

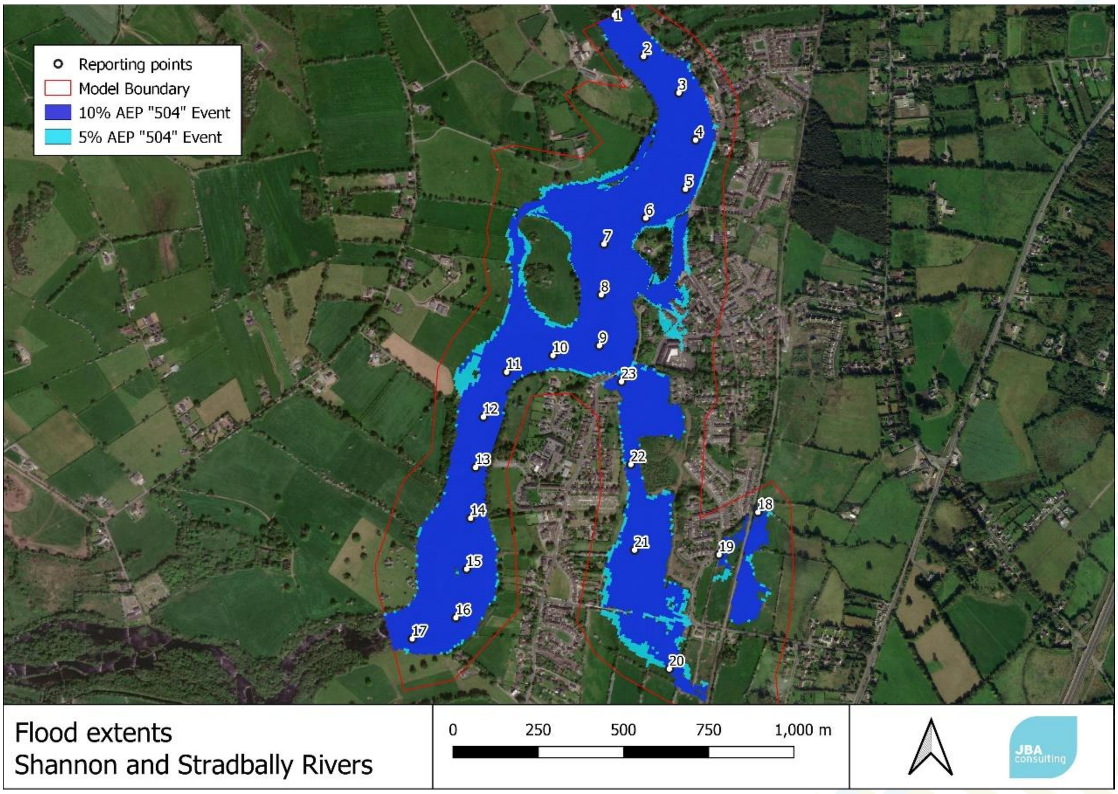


Figure B-2: Shannon and Stradbally Watercourses Flood Extents – 10% and 5% AEP "504" event (345 m³/s headrace flow)

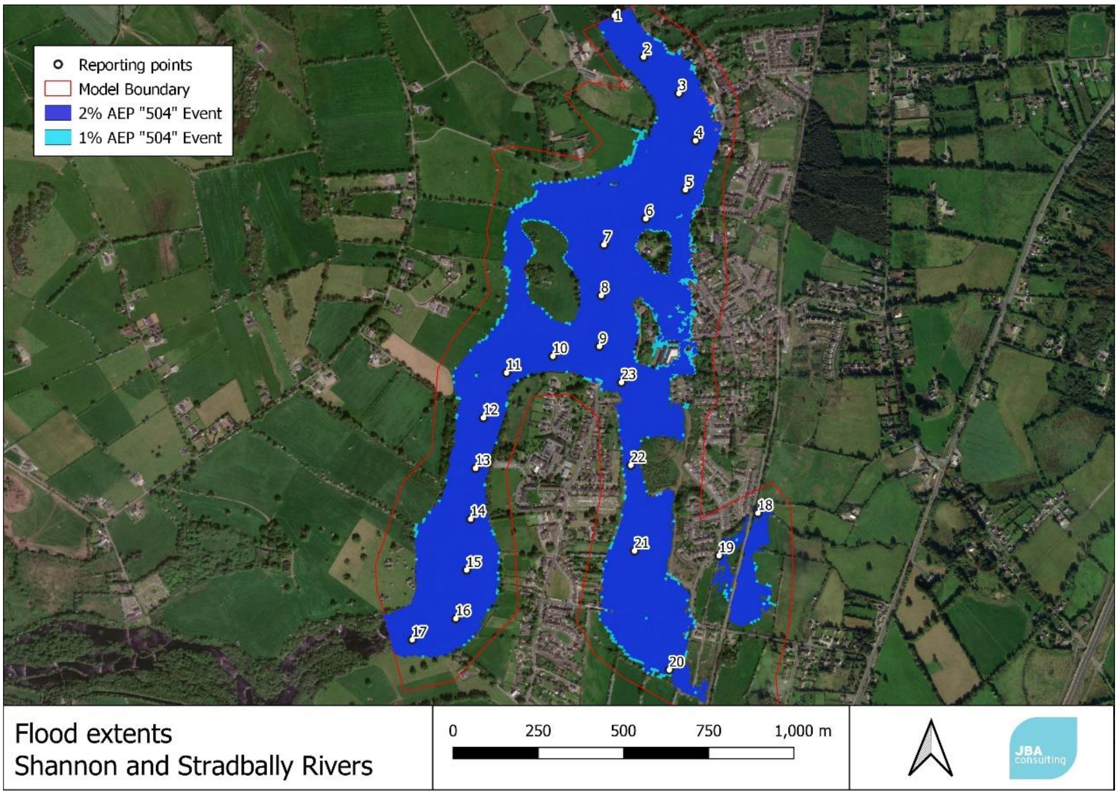


Figure B-3: Shannon and Stradbally Watercourses Flood Extents – 2% and 1% AEP "504" event (345 m³/s headrace flow)

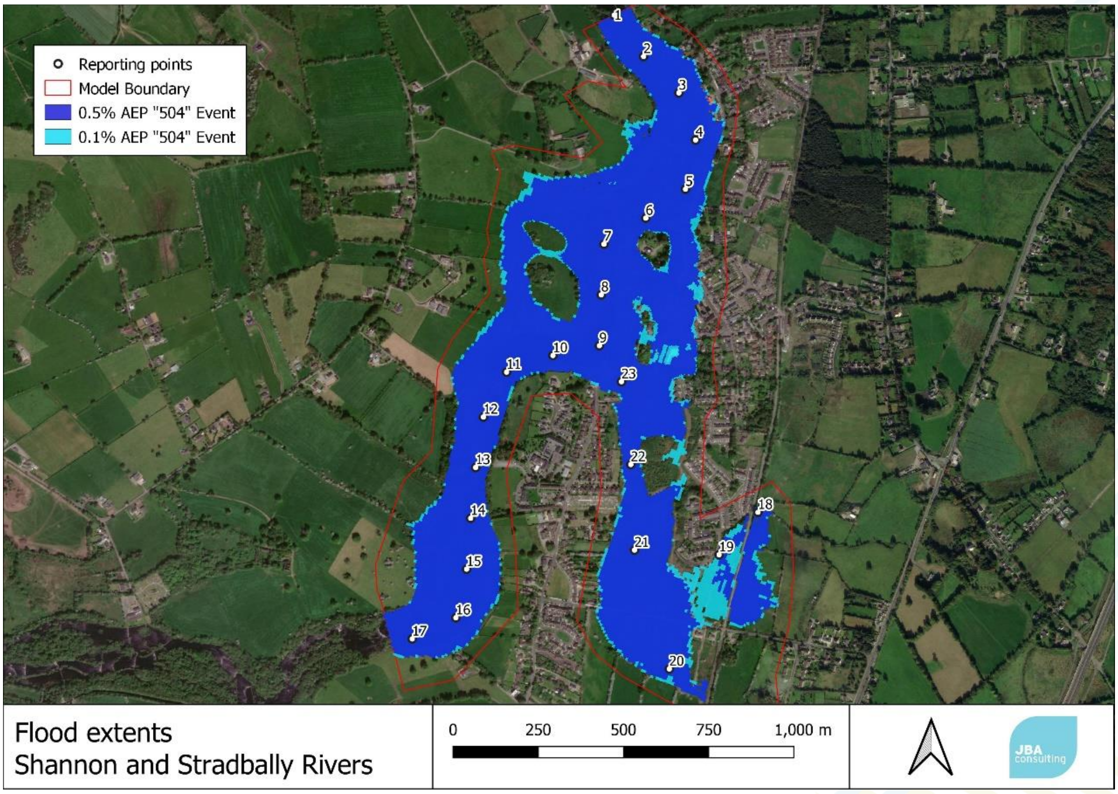


Figure B-4: Shannon and Stradbally Watercourses Flood Extents – 0.5% and 0.1% AEP "504" event (345 m³/s headrace flow)

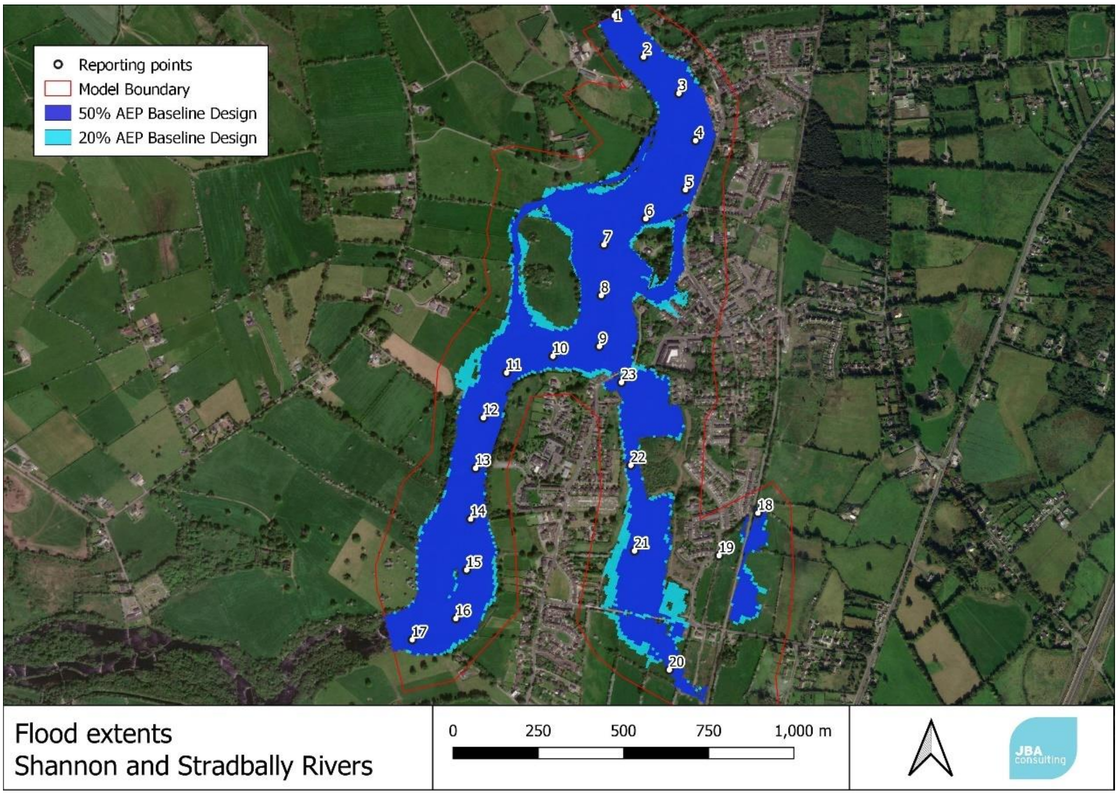


Figure B-5: Shannon and Stradbally Watercourses Flood Extents – 50% and 20% AEP Baseline Design Event (258 m³/s headrace flow)

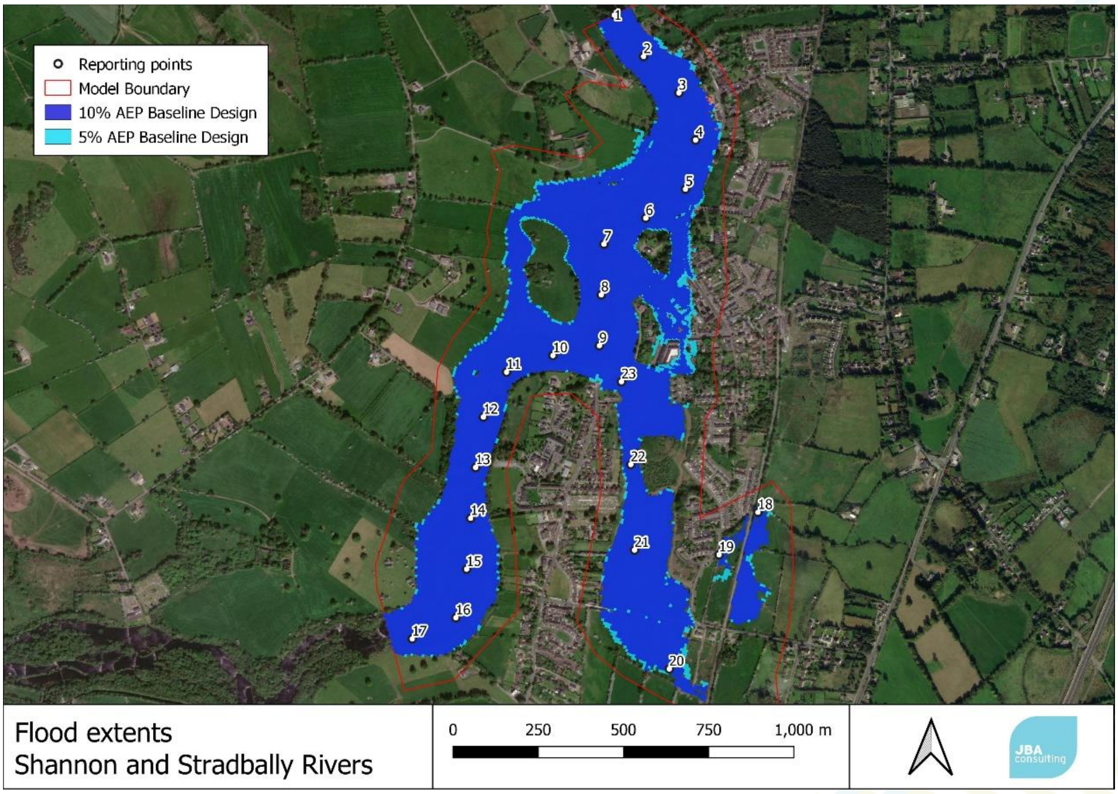


Figure B-6: Shannon and Stradbally Watercourses Flood Extents – 10% and 5% AEP Baseline Design Event (258 m³/s headrace flow)

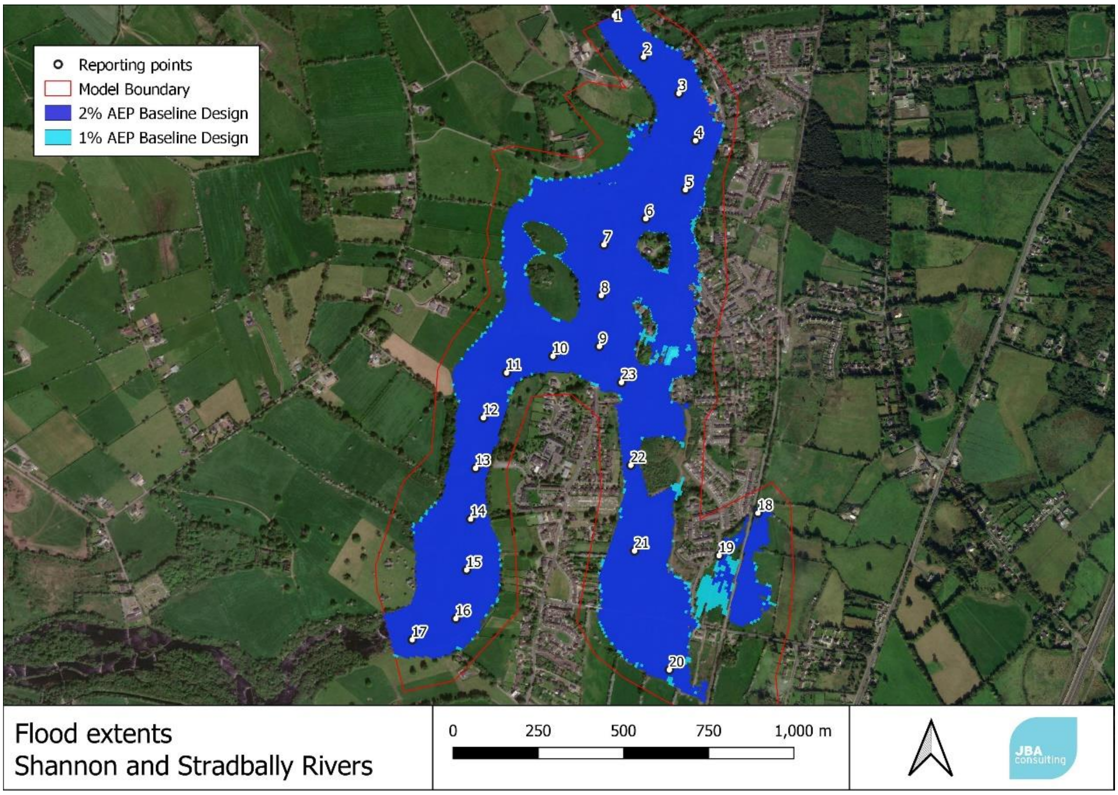


Figure B-7: Shannon and Stradbally Watercourses Flood Extents – 2% and 1% AEP Baseline Design Event (258 m^3/s headrace flow)

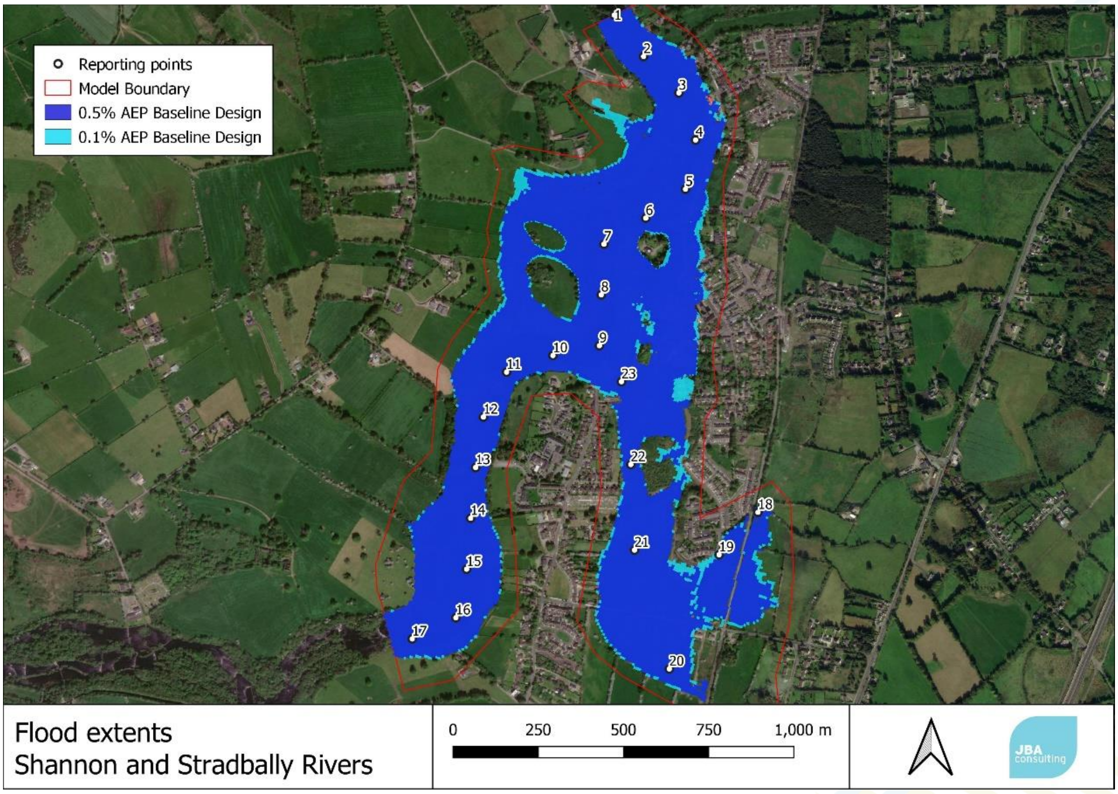


Figure B-8: Shannon and Stradbally Watercourses Flood Extents – 0.5% and 0.1% AEP Baseline Design Event (258 m^3/s headrace flow)

Figure B-9: Shannon and Stradbally Watercourses – Longitudinal Profile Chainage Points

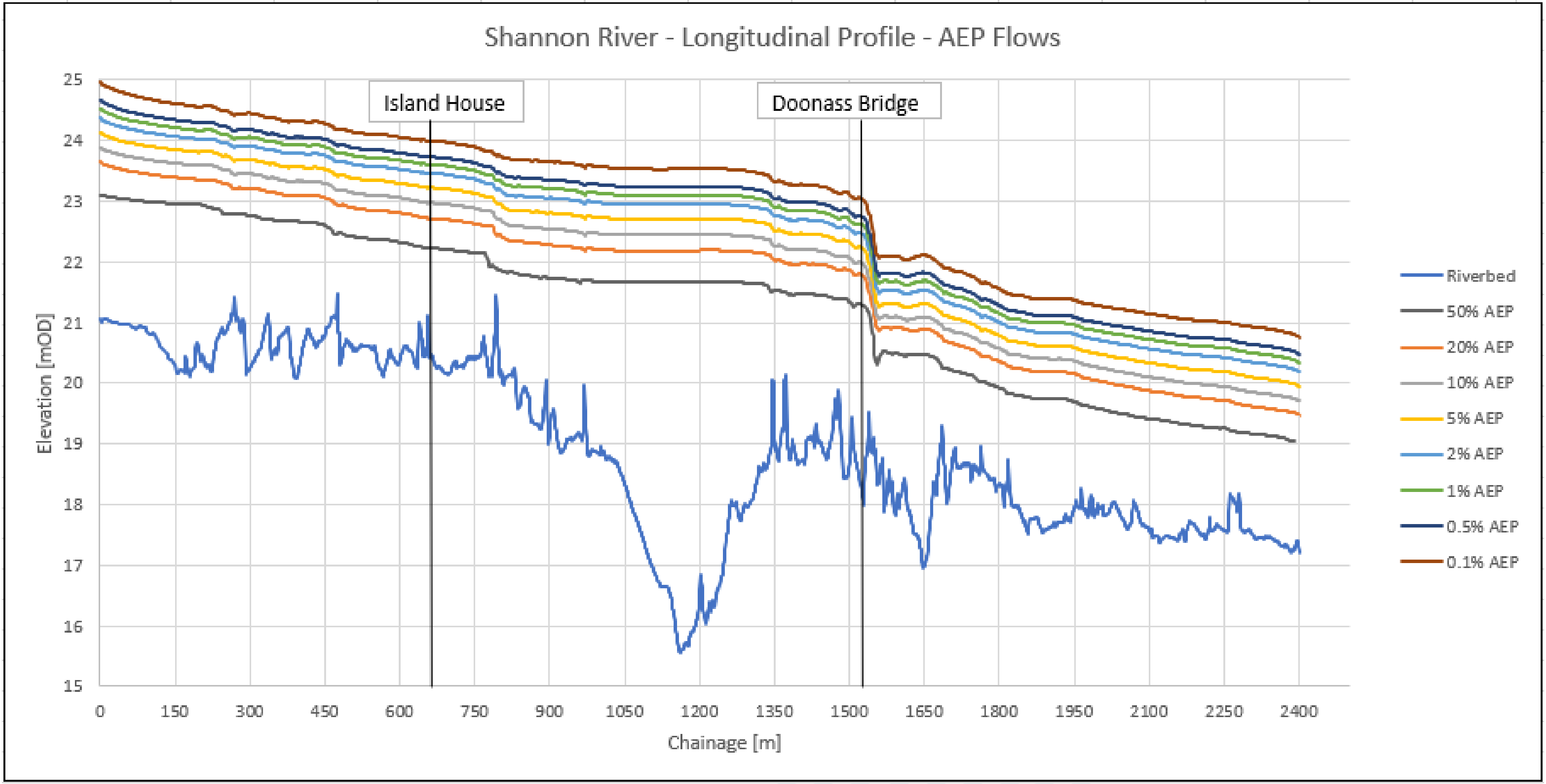


Figure B-10: Shannon River – Longitudinal Profile – "504" event (345 m^3/s headrace flow)

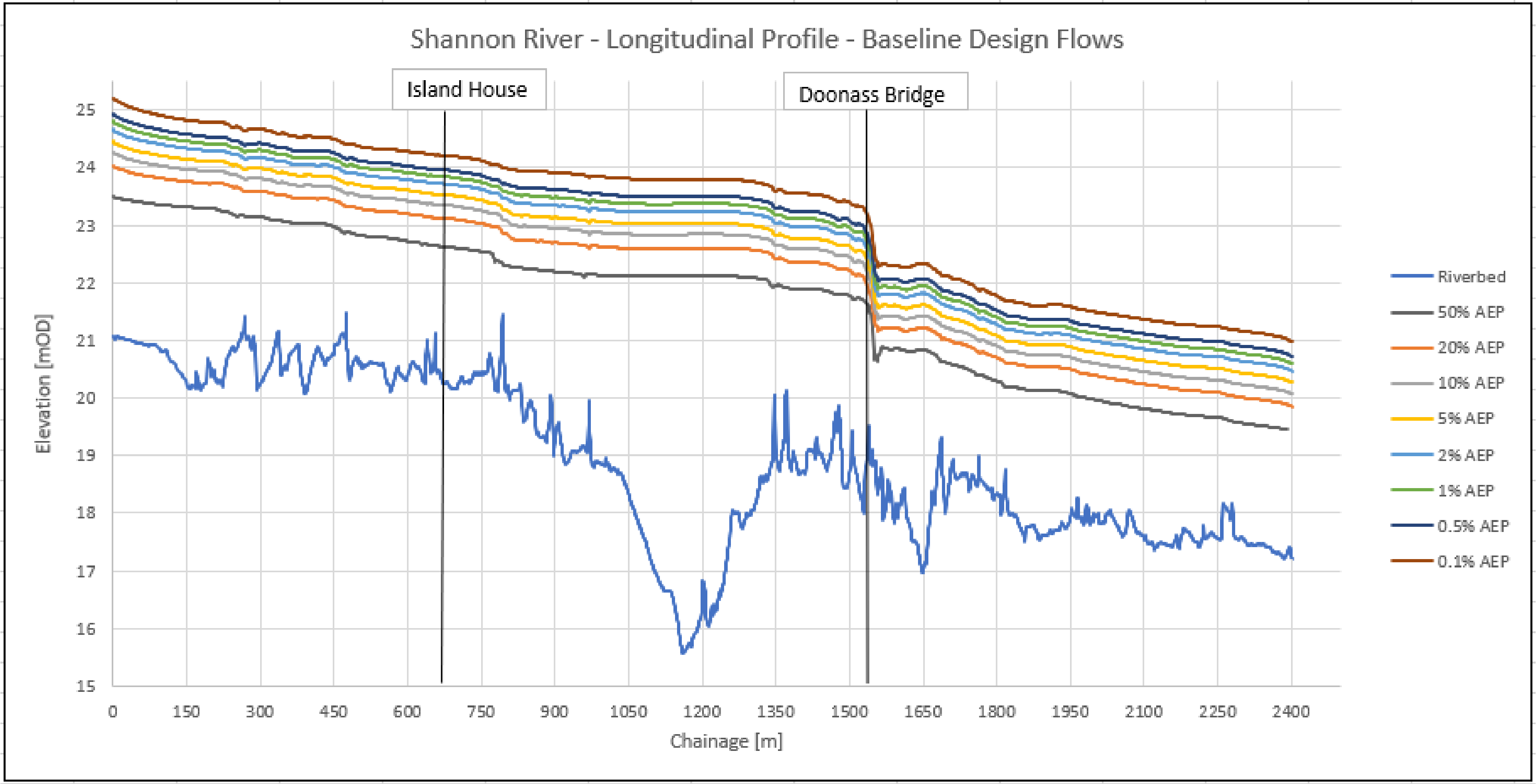


Figure B-11: Shannon River – Longitudinal Profile - Baseline Design Event (258 m³/s headrace flow)

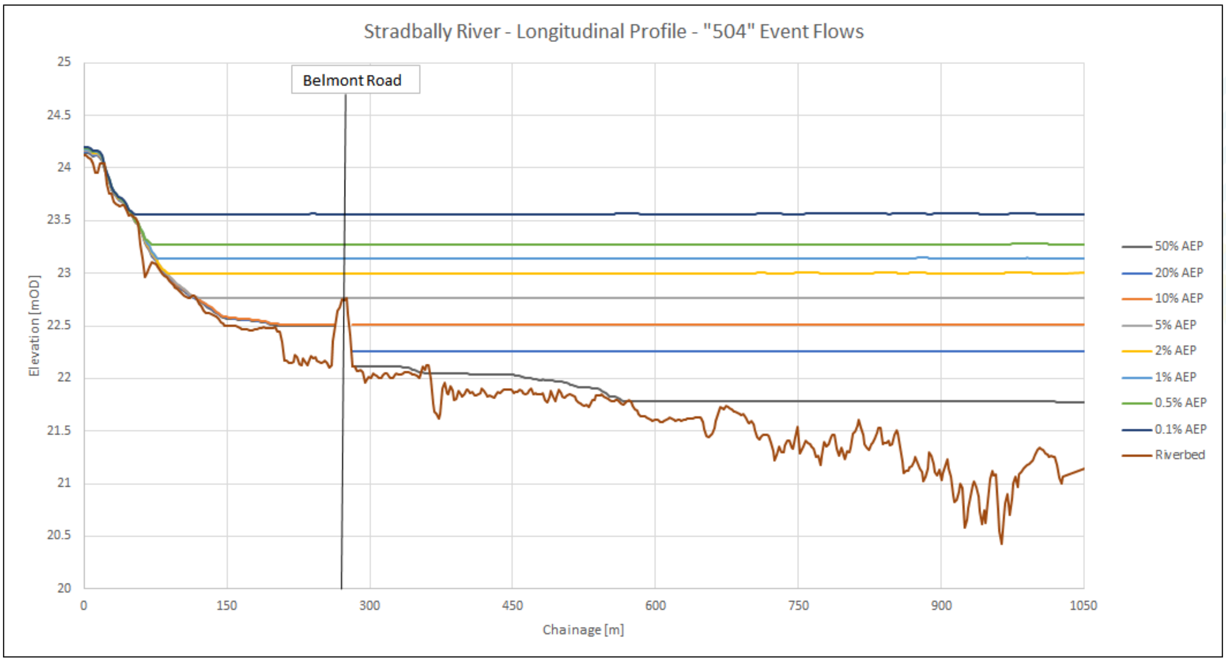


Figure B-13: Stradbally Stream – Longitudinal Profile "504" event (354 m³/s headrace flow)

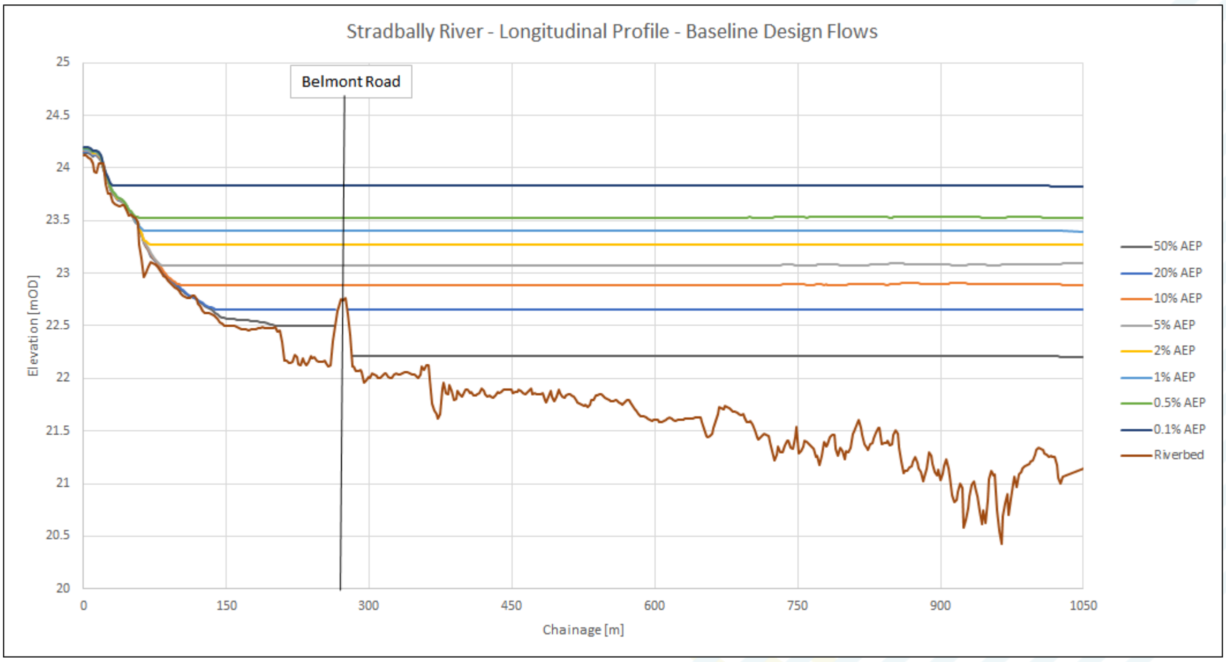


Figure B-14: Stradbally Stream – Longitudinal Profile – Baseline Design Event (258 m³/s headrace flow)

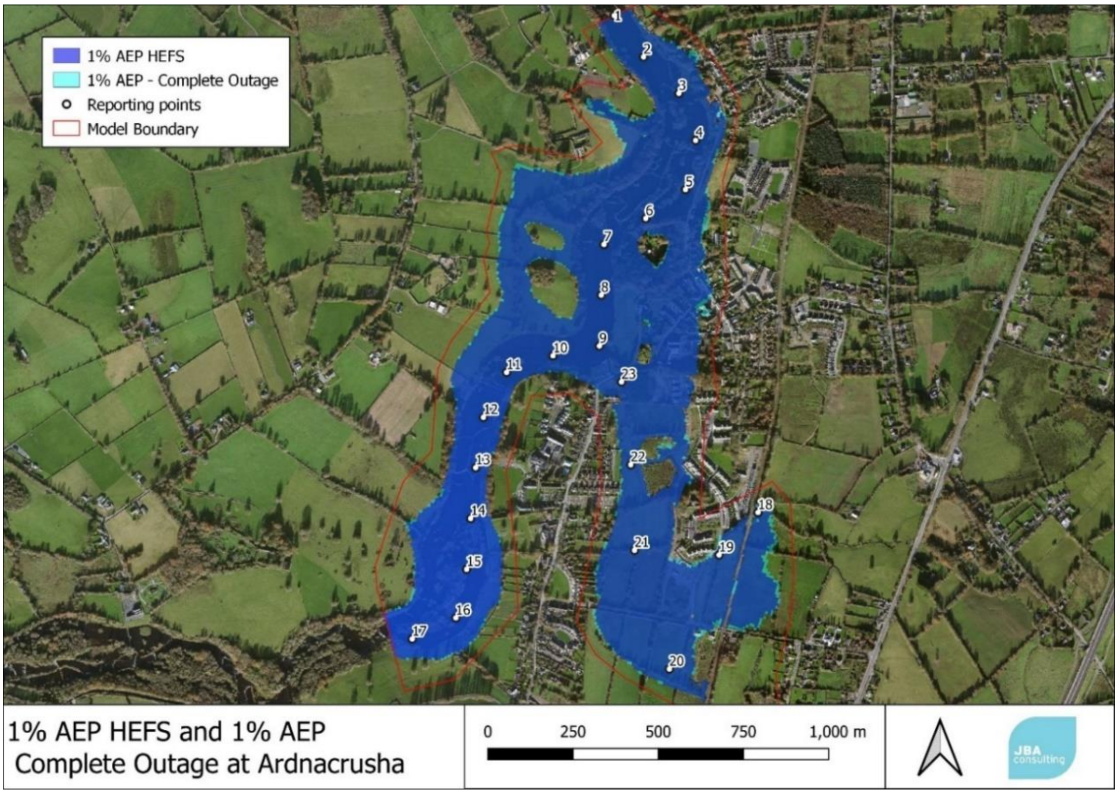


Figure B-15: 1% AEP HEFS for “504” event (standard operational conditions - 345 m^3/s headrace flow) and 1% Complete Outage at Ardnacrusha (zero headrace flow)

Appendix C – "504" Event and Baseline Design Event Flows

Table C1 – AEP flows "504" event (345 m^3/s headrace flow to Ardnacrusha)

%AEP	Flow over Parteen Weir (Shannon) [m3/s]	Kilmastulla River [m3/s]	Black River [m3/s]	Cedarwood Stream [m3/s]	Stradbally Stream (East) [m3/s]	Stradbally Stream (South) [m3/s]
50%	181.90	33.29	12.07	0.41	0.63	0.30
20%	281.30	33.29	12.07	0.56	0.80	0.38
10%	340.50	33.29	12.07	0.65	0.91	0.44
5%	393.50	33.29	12.07	0.76	1.03	0.49
2%	458.20	31.40	11.11	0.92	1.19	0.57
1%	504.40	28.71	9.82	1.07	1.31	0.63
0.5%	549.00	28.71	9.82	1.23	1.43	0.69
0.1%	648.50	28.71	9.82	1.66	1.74	0.84

Table C2 – Baseline Design Event (258 m^3/s headrace flow to Ardnacrusha)

%AEP	Flow over Parteen Weir (Shannon) Baseline Design Flows [m3/s]	Kilmastulla River [m3/s]	Black River [m3/s]	Cedarwood Stream [m3/s]	Stradbally Stream (East) [m3/s]	Stradbally Stream (South) [m3/s]
50%	268.90	33.29	12.07	0.41	0.63	0.30
20%	368.30	33.29	12.07	0.56	0.80	0.38
10%	427.50	33.29	12.07	0.65	0.91	0.44
5%	480.50	33.29	12.07	0.76	1.03	0.49
2%	545.20	31.40	11.11	0.92	1.19	0.57
1%	591.40	28.71	9.82	1.07	1.31	0.63
0.5%	636.00	28.71	9.82	1.23	1.43	0.69
0.1%	735.50	28.71	9.82	1.66	1.74	0.84

Appendix D – Cedarwood Stream

The Cedarwood Stream, located at the northern periphery of Castleconnell, differs substantially in scale and hydraulic characteristics from the Shannon River, necessitating a distinct approach for modelling its behaviour. In contrast to the Shannon's intricate hydraulic regime, shaped by its islands, pools and weirs, the Cedarwood Stream presents a more straightforward system that makes its way through the urban environment to outfall into the Shannon.

These particular characteristics of the Cedarwood Stream led to the construction of a separate 1D-2D FM-Tuflow model and hence why it is reported within this Appendix. The modelling of the stream was also phased at a later time in the programme after the Shannon modelling was complete and required new survey in 2022. A separate model was deemed appropriate for capturing the nuances of the watercourse's hydraulic behaviour and the potential impact of backwater flow from the Shannon River, which may surpass the bank heights of the Cedarwood Stream in its lower reaches. Further survey works were undertaken in 2024 following desilting and maintenance works on the stream upstream of the rail crossing. These necessitated an updating of the baseline model to capture the most up-to-date channel geometry. This was to ensure that the baseline flood extents presented are the most up-to-date.

The output from the main Shannon model, particularly the recorded water levels, served as the water level-time boundary for the Cedarwood model, ensuring a seamless integration between the two models.

The Cedarwood model, however, was excluded from the main Shannon River model. Instead, the stream's flows are incorporated directly into the Shannon model's 2D domain at its outlet, accurately depicting the interaction of these two watercourses.

D.1 – Baseline Data

D.1.1 – LiDAR

The Lidar data used in the model has a 2m resolution and a stated vertical accuracy of 200mm.

D.1.2 – Survey

The construction of the 1D model for the Cedarwood Stream relied on three distinct sets of topographic survey data. The initial survey, conducted by Murphy Surveys in 2012, provided coverage for the entire model extent, excluding the main river branch along the railway line (refer to Section D.2.1). To ensure the relevance of the 2012 data, it was sense-checked against both the 2022 and 2024 survey data. The 2012 data was cross-referenced against these newer surveys, which were collected as part of a phased approach to the Cedarwood Stream model, initiated later in the programme following the completion of the Shannon River model. This check confirmed that the 2012 data remained suitable for use in the downstream area of the Cedarwood Stream, where channel geometry has remained relatively stable.

As such, a subset of the 2012 data was utilised—specifically, 16 channel cross-sections and 6 structures at the downstream end of the Cedarwood Stream, covering the final 140m stretch before its confluence with the Shannon River. These particular sections showed minimal changes, making the older data reliable for this part of the model.

For the remaining model area, including the river branches along the railway line, newer survey data was required. A 2022 survey by McDonald Surveys was undertaken, which captured 35 cross-sections of the Cedarwood channel and 6 structures. This data provided an updated view of the Cedarwood Stream's hydraulic behaviour, including potential backwater effects from the Shannon River during high flow conditions.

Additionally, following maintenance and desilting works upstream of the railway line in 2023/2024, as well as the collapse and removal of the southern culvert wingwall, a further survey was conducted by MCDS in 2024. This data was incorporated into the model to ensure that the most up-to-date channel geometry was used, particularly in areas that were modified due to the desilting works. By updating the baseline model with the 2024 survey, the flood extents now reflect the current conditions of the Cedarwood Stream.

D.2 – Model Schematisation

D.2.1 – Study area characteristics

The modelled length of Cedarwood is 1090m and the total modelled watercourse floodplain area is approximately 0.16km2. The catchment is intersected by a railway line which provides a considerable constriction to flow and separates land use types. The catchment is predominantly rural upstream of the railway line and urbanised downstream of the rail line to the Shannon River. The width of the channel varies from 7m upstream to 2m downstream with a slope of approximately 0.65%.

The model extent is presented in Figure D0-1.

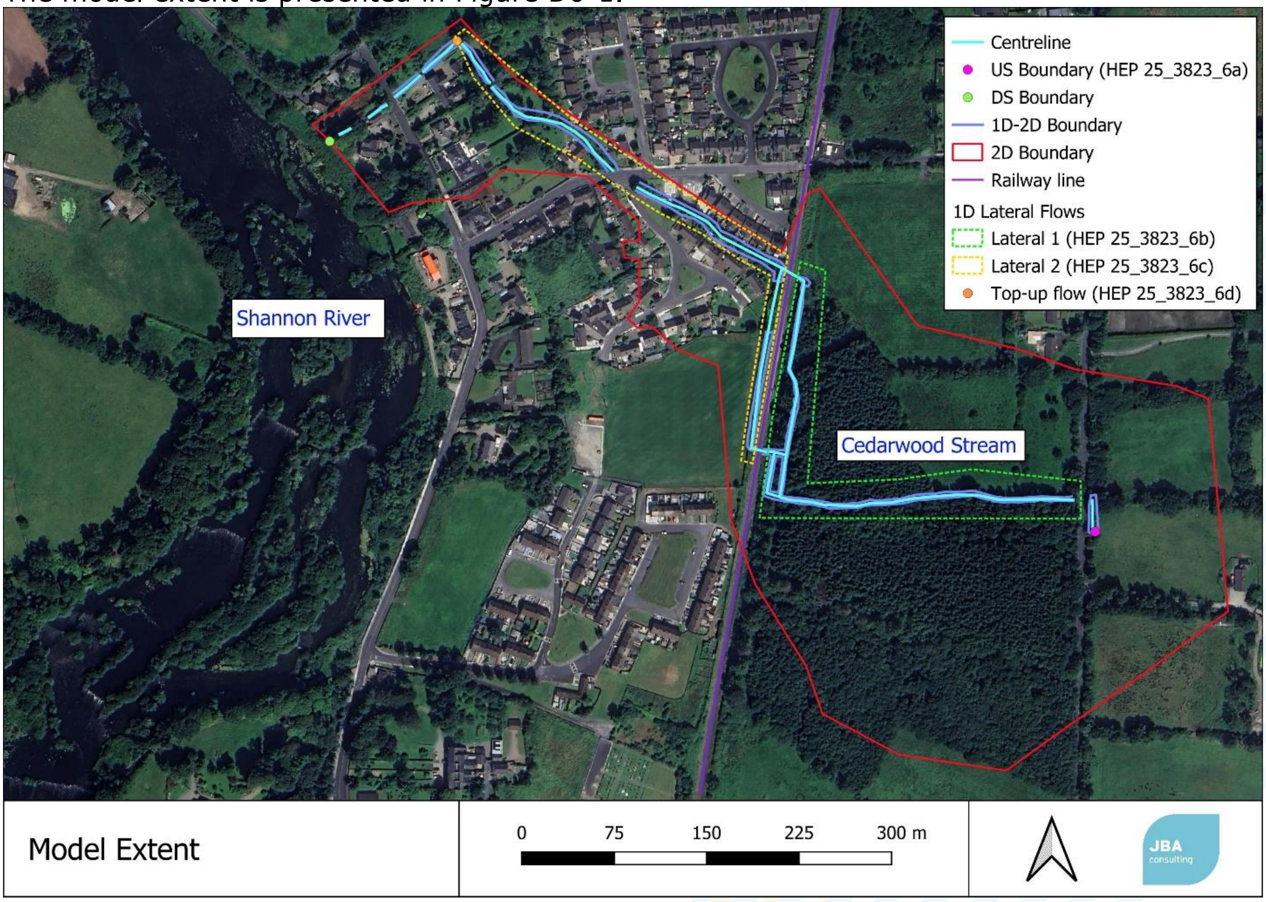


Figure D0-1: Model Extent

Based on both the 2022 and 2024 survey data, as well as site visits, it was discovered that the watercourse has two branches through the railway line , each with a culvert that crosses the railway line. Both branches and both culverts were included in the current FRS model. However, the CFRAM model only included the southern culvert, which has a higher invert level (by 0.6m) than the northern culvert. As such, the majority of the flow would spill through the northern culvert. This highlights a discrepancy between the CFRAM model and the actual watercourse conditions.

Additionally, the OSi map only shows one branch along the railway line, consistent with the CFRAM model.

D.2.2 – Software

A Flood Modeller-Tuflow model has been developed for this study because the narrow sections of the channel would not be well represented in 2D only, the 1D/2D model providing a better representation of the channel

D.2.3 – Grid size selection

A grid size of 2m has been used to represent the 2D domain in the model in order to accommodate the narrow sections of the channel where the width drops up to 2m. A 1m grid has been considered, but this would not provide an improved topography representation due to the grid cell size of the LiDAR (2m).

D.2.4 – Grid orientation

The model grid has been oriented from SE to NW in order to be aligned with the main direction of the watercourse downstream of the railway line, in the urban area, where the risk of flooding occurs.

D.3 – Model development

D.3.1 – Boundaries

The flow boundary conditions were integrated into the model in the form of two-point inflow hydrographs. These were situated at the upstream node of the model and also served to supplement the downstream region. In addition, two lateral inflow hydrographs were applied both upstream and downstream of the railway line. The maximum water levels are illustrated in Table D0-1, while Figure D0-2 presents the hydrographs related to the 1% AEP. For a visual reference to the locations of the flow boundaries, please refer to Figure D0-1.

Table D0-1: Cedarwood Flows [m^3/s]

AEP (%)	Cedarwood Upstream	Cedarwood Lateral 1	Cedarwood Lateral 2	Cedarwood North
	25_3823_6a	25_3823_6b	25_3823_6c	25_3823_6d
50% (2yr)	0.30	0.05	0.01	0.05
20% (5yr)	0.41	0.07	0.01	0.07
10% (10yr)	0.48	0.08	0.01	0.08
5% (20yr)	0.57	0.09	0.01	0.09
2% (50yr)	0.69	0.11	0.01	0.11
1% (100yr)	0.79	0.13	0.02	0.13
0.5% (200yr)	0.91	0.15	0.02	0.15
0.1% (1000yr)	1.22	0.21	0.03	0.20

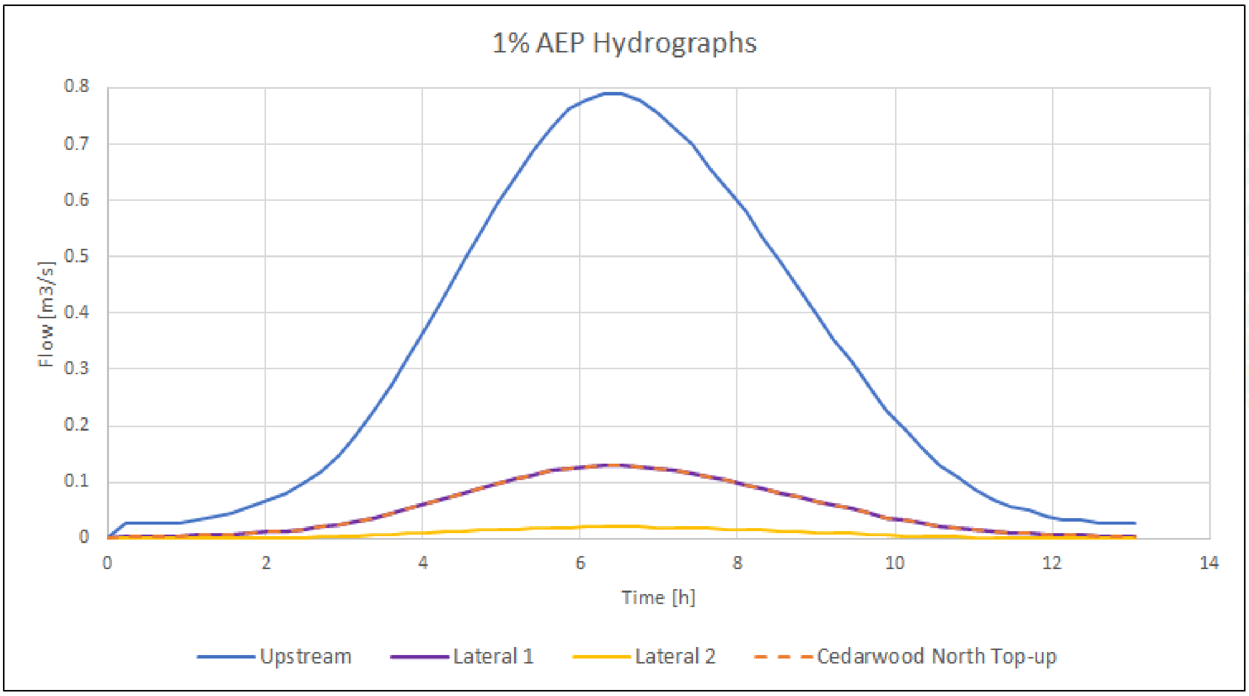


Figure D0-2: 1% AEP Hydrograph – Cedarwood Stream

The downstream boundary of the model is a fixed water level – time (HT) model boundary, using the peak water level extracted from the main Shannon model for the 1% AEP event (Ardnacrusha Power Station functions in standard operational conditions), peaking at 24.15 mOD. The Cedarwood Stream modelling assumes a peak-to-peak phasing with the Shannon River. Using the water level from the Shannon River during the 1% baseline design event was considered to potentially overestimate the flood risk on Cedarwood Stream. The '504' event was chosen to avoid an overestimation of the flood risk, providing a more balanced assessment of potential flood conditions on the Cedarwood Stream.

The link between the 1D and 2D domains within the Cedarwood model has been created using connection lines.

D.3.2 – Roughness coefficients

The Manning's values roughness applied to the 1D channel were 0.08, 0.045 and 0.15, as shown in the table below. Vegetation growth, especially in the summer is a significant feature of this watercourse.

The Manning's values were applied in accordance with photographic evidence along the stream, site visits in 2024 and 2022 and with reference to Chow 1959. The roughness coefficients were applied uniformly at cross sections (same values at bed and top of banks).

Reach	Manning's N value	Example cross section photograph
Upstream section (Upstream of Railway line)	0.045	 CED01022 (2024 photo)
	0.080	 CED01020 (2024 photo)

Upstream-Middle section (Cedarwood Grove)	0.055	 01CED01015 (2022 photo)
Downstream -middle section (80m Downstream of the Common's culvert)	0.15 (extremely vegetated channel)	 CED01011 (2022 photo)

Downstream section	0.045	 01CED00148 (2012 photo)

D.4 – Flood Behaviour

The modelling results show a key constriction along the Cedarwood channel is created by the railway line embankment, which delays the hydrograph peak and attenuates the flows. As a result, flow storage is naturally provided upstream of the railway line.

To provide a more detailed view, Figure D0-3 presents a cross-section of the railway embankment (A-A'), showing a water level of 28.69mOD upstream and 28.68mOD downstream of the embankment during a 1% AEP event. The embankment crest, however, stands significantly higher, at 30.56mOD, almost 2m above the peak flood level.

Taking into account the natural topography, it is important to note that the ground levels upstream are lower than those downstream. This means that areas upstream are naturally more susceptible to flooding.

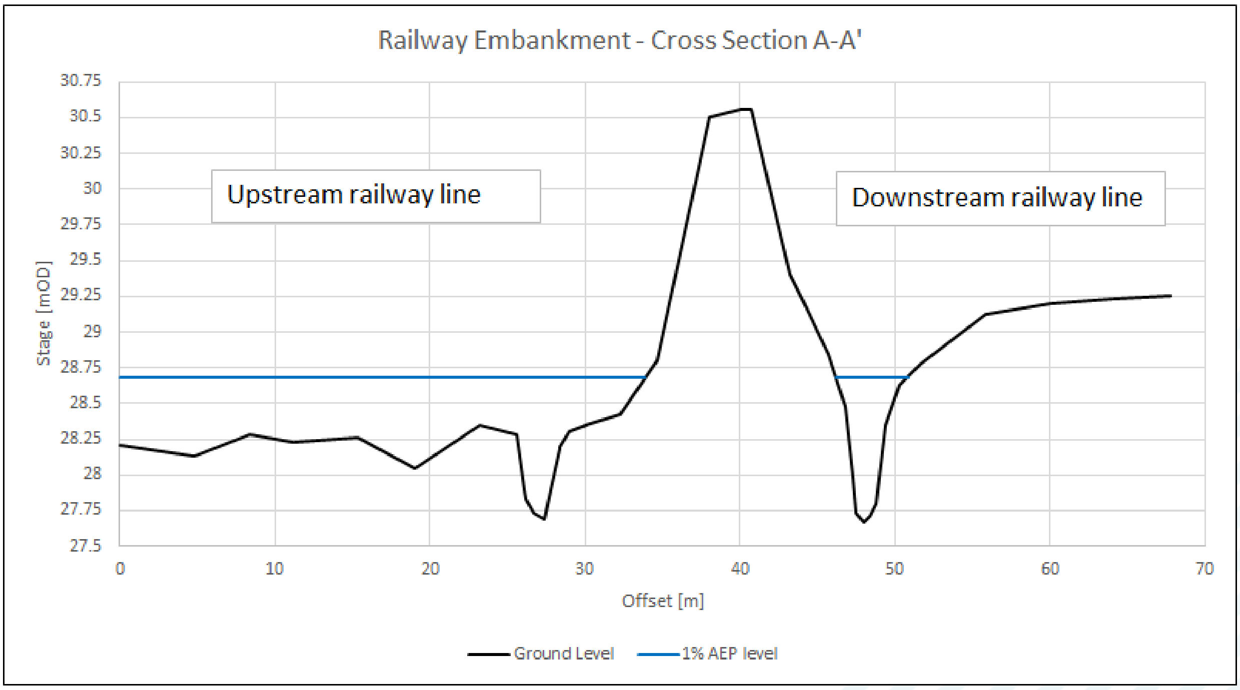


Figure D0-3: Railway Embankment – Cross Section A-A'

Figure D0-4 presents the difference between the hydrograph upstream and downstream of the railway line (nodes CED01024 and CED01016). The hydrograph peak is delayed approximately 2.5 hours and the peak flow is attenuated from 0.79 m^3/s to 0.66 m^3/s as a consequence of the railway constriction.

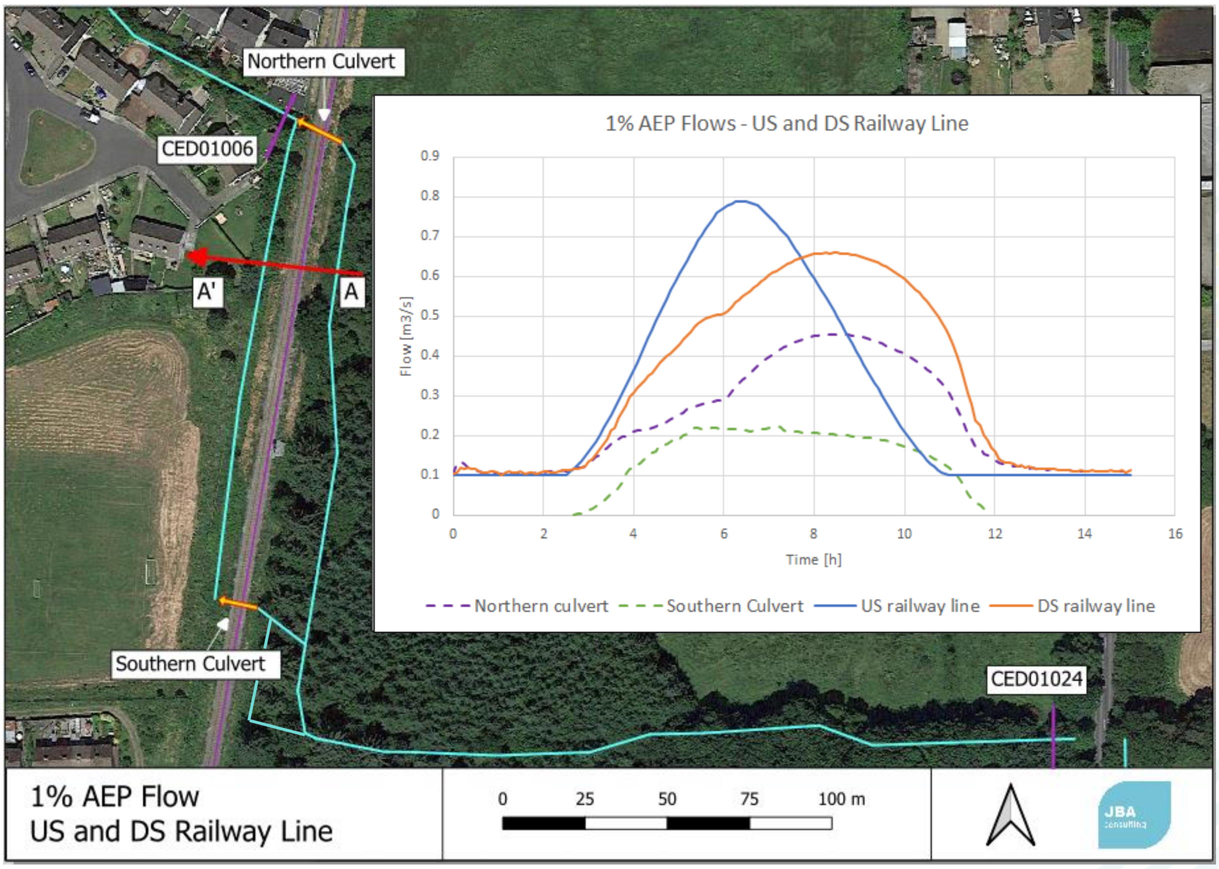


Figure D0-4: 1% AEP Hydrographs upstream and downstream of the railway line

Figure D0-5 illustrates the extent of the 1% AEP flood upstream of the railway line. Flooding in this area occurs by overtopping both the left and right banks and filling of the forestry lands. The outflow flow from this area is conveyed by the two culverts under the railway line, as displayed in Figure D0-4. To note the CFRAM model incorporated only 1 culvert (southern culvert). The majority of flow is predicted to pass through the northern culvert, which has a lower invert level (0.6m lower than the southern culvert).

The properties along Cedarwood Grove are not indicated as flooded in the 1% and 0.1% AEP event runs, unlike the predictions made by the original CFRAM model. This discrepancy arose from a modelling error in the CFRAM model. Specifically, the Manning's n roughness value for the Common's Road culvert was mistakenly inputted as 0.2 instead of the accurate value of 0.02, thereby inaccurately increasing the water levels upstream of the culvert. There has also been an improved understanding of the hydrology within the catchment and the current models have a lower flow distribution along the main reach of the Cedarwood Stream.

Figure D0-6 presents the cross section CED01016, showing the modelled flood water levels for the 1% and 0.1% AEP events. The water level margin to top of bank level at the lower left bank is 190mm and 80mm respectively. The minimum freeboard to the property threshold level is 370mm during the 1% AEP event (minimum floor level is 28.92mOD). The freeboards at this location are discussed further in Section D.7.2.

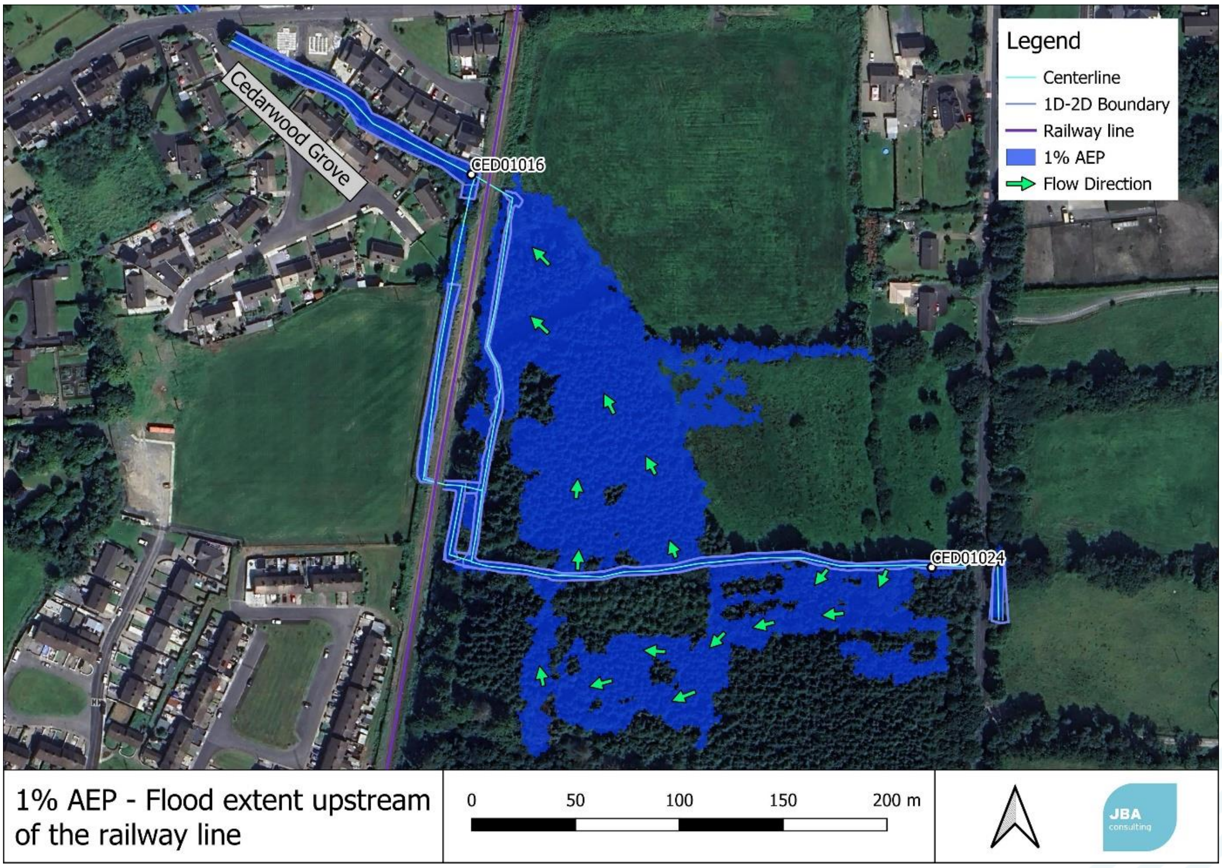


Figure D0-5: 1% AEP flood extent Cedarwood Grove and upstream of the railway line

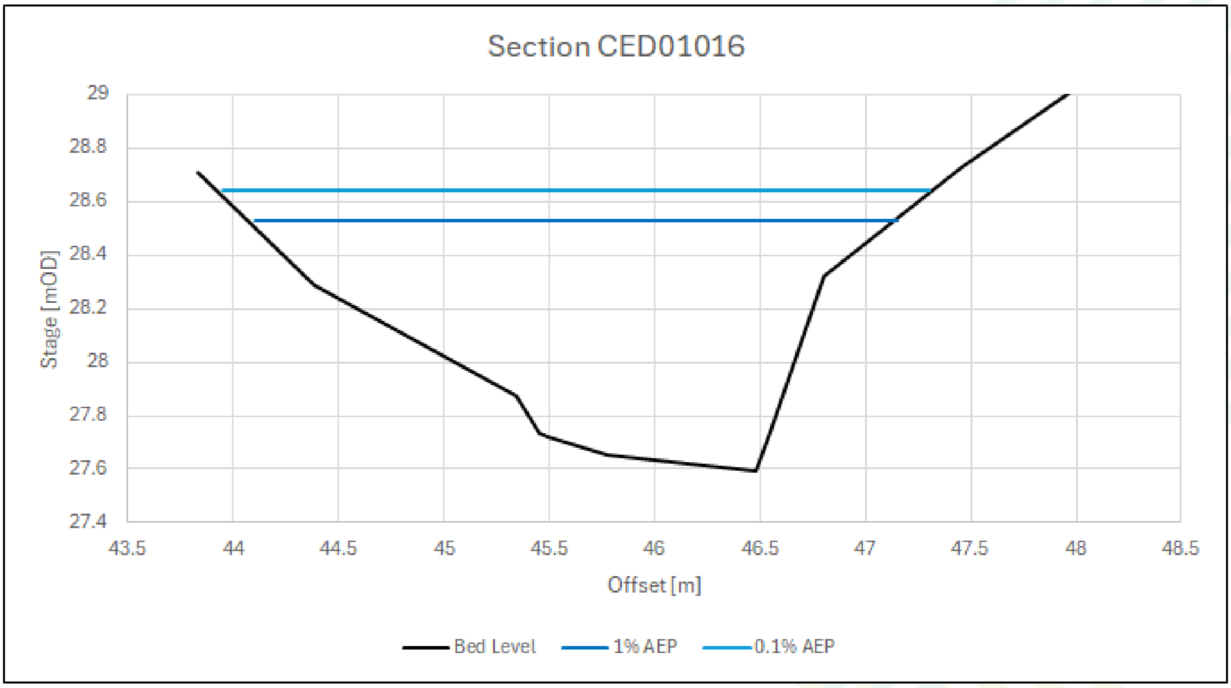


Figure D0-6: 1% and 0.1% AEP water levels – Section CED01016 at Cedarwood Grove

Figure D0-7 depicts the extent of flooding downstream. In the scenario of a 1% AEP event, overtopping is observed on the left bank due to a flow constriction caused by the culvert identified as CED01004 (refer to Figure D0-7). The culvert invert level, which is 400mm above the bed level as represented in the longitudinal section in Figure D0-10, elevates the water levels upstream. This results in overtopping upstream of the bridge, specifically at section CED01007. During the 0.1% AEP, the left bank overtopping results in a flow path towards the Shannon confluence (see Section B-B').

The steep slope of 3% on Cedarwood Stream, upstream of the confluence with the Shannon (as depicted in Figure D0-10), means that the flooding of the Cedarwood floodplains is not influenced or impacted by the River Shannon.

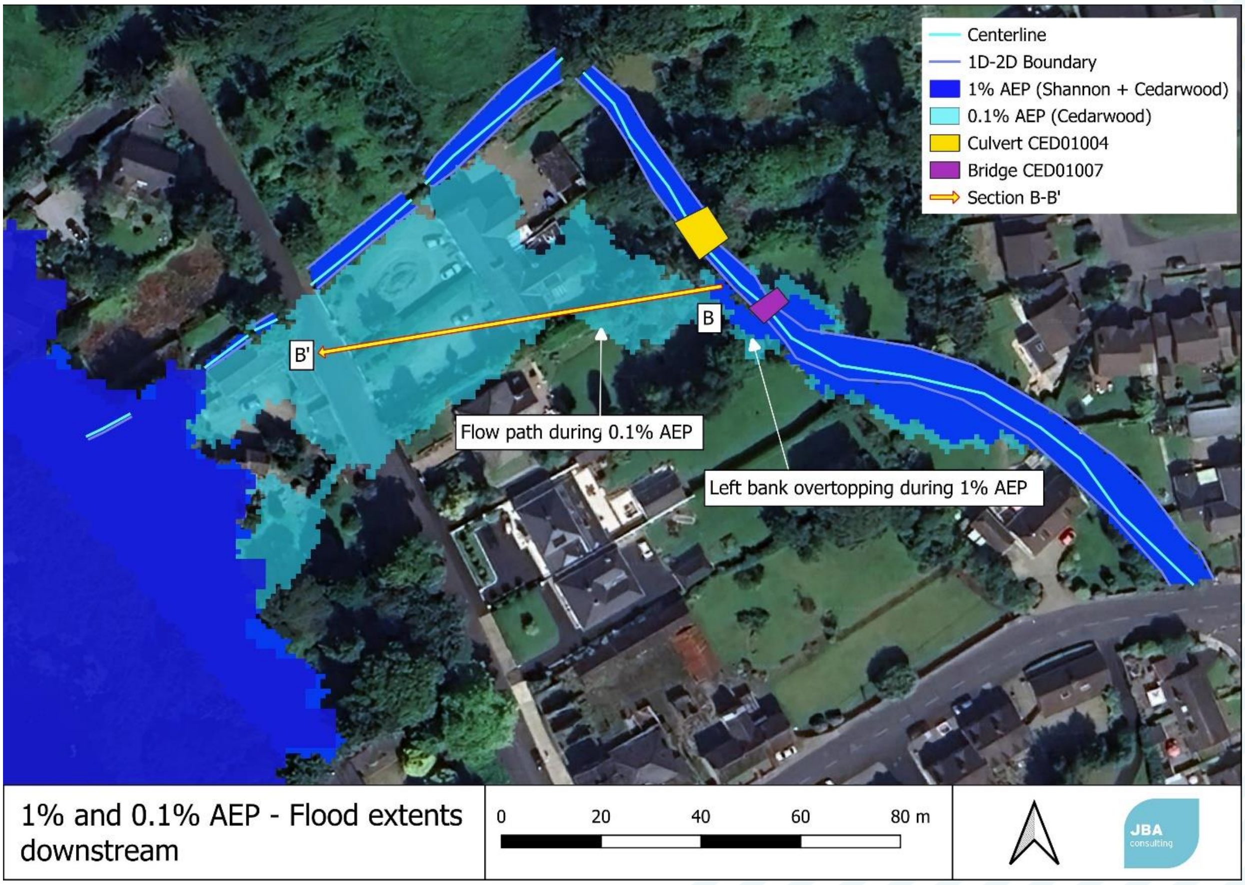


Figure D0-7: 1% AEP flood extent - downstream end of Cedarwood Stream

(Note: no flood depth filtering has been applied to this extent map)

A longitudinal section along the flow path (identified as B-B') is depicted in Figure D0-8. This figure demonstrates the flow path that is formed during a 0.1% AEP event, with the maximum depth reaching 100mm. Notably, the 1% AEP water level on the left bank stands at 28.06mOD, only 20mm lower than the 28.08mOD threshold level that would result in overtopping downstream, thereby enabling a flow path during a 1% AEP event as well. Thus, the model shows high sensitivity to water levels in this area, with a minor difference of only 20mm determining whether a flow path forms (and consequently impacting properties) or not during the 1% AEP event.

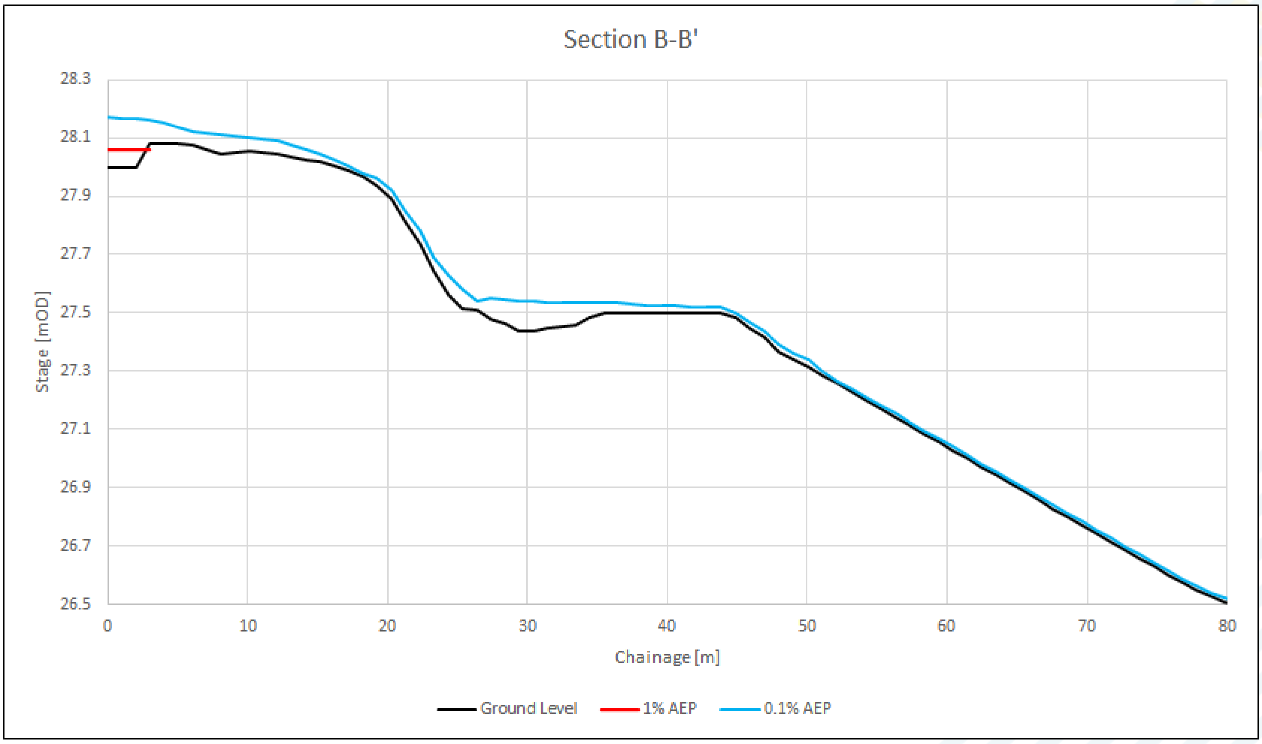


Figure D0-8: Section B-B' – 1% and 0.1% AEP levels

The modelled flows and water levels at reporting locations are presented in Table D0-2 for the 1% AEP event.

Refer to Section D.6 – Model Results for the results of the full range of return periods.

Table D0-2: Cedarwood Stream – Flows and Water Levels 1% AEP

Node label	1% AEP	
	Water Level [mOD]	Total Flow [m³/s]
CED01024	29.07	0.79
CED01015	28.50	0.66
CED01013	28.39	0.66
CED01007	28.07	0.67
01CED00077	25.36	0.76

The 1% AEP flood extent is presented in Figure D0-9 and the longitudinal profile in Figure D0-10.

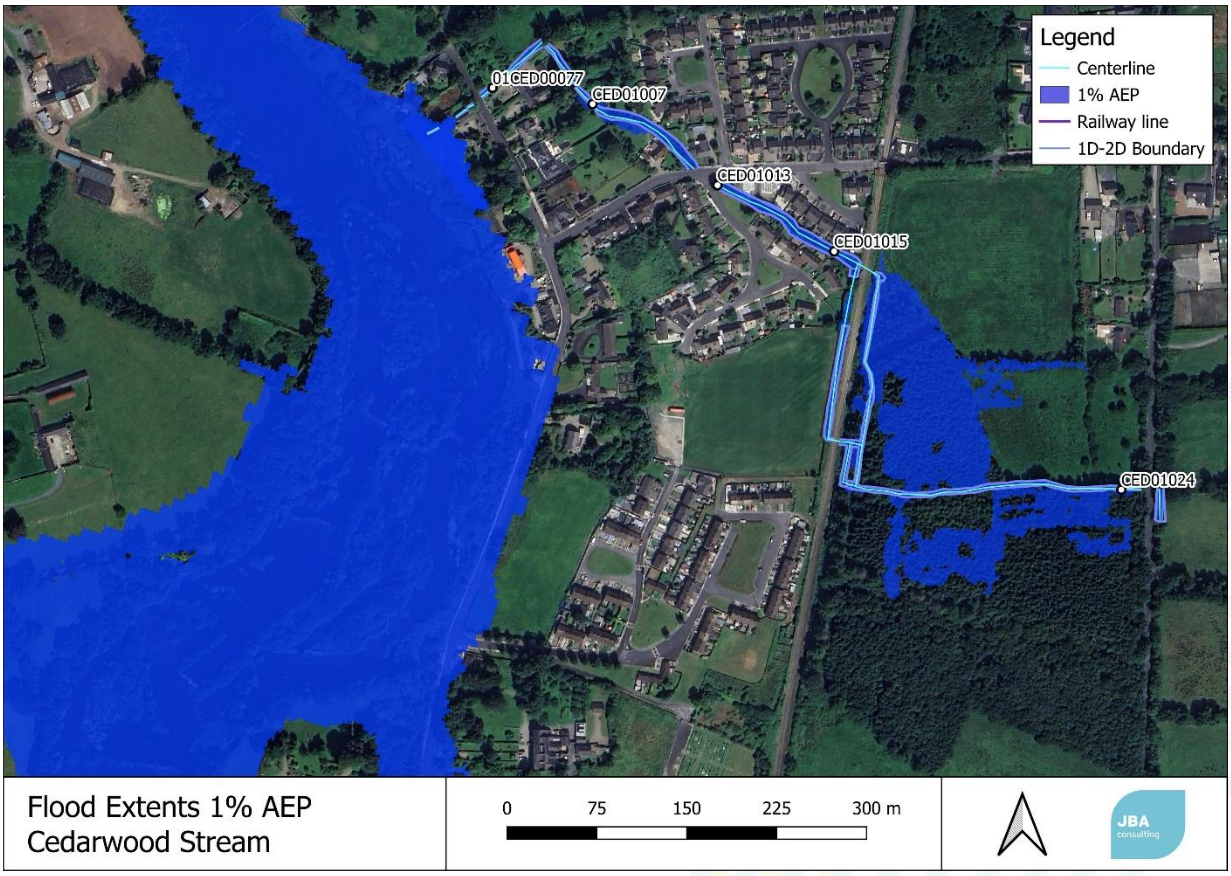


Figure D0-9: Cedarwood Stream – 1% AEP Flood Extent

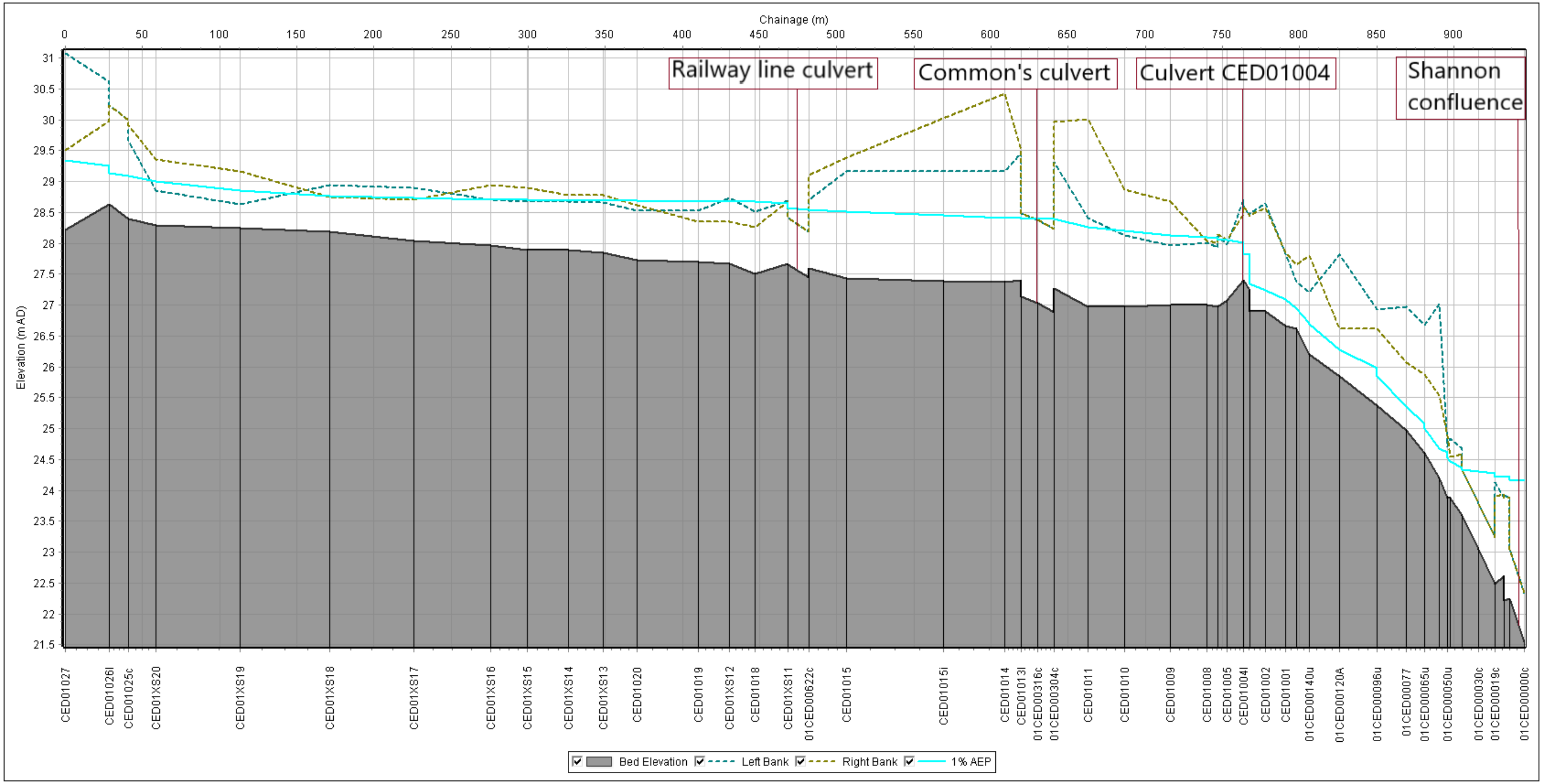


Figure D0-10: Cedarwood Stream - Peak 1% Water Level – Longitudinal Section

D.5 – Model Sensitivity Testing

D.5.1 – High and low Manning's n values

The Manning's n surface roughness applied to the model (see Section D3.2) was increased and decreased by 20%, as presented in Table D0-3 below. Vegetation and debris is a significant factor in the management of the Cedarwood Stream.

Table D0-3: Manning's n values used in the sensitivity tests

Material	Baseline values	High n (+20%)	Low n (-20%)
Riverbed	0.045	0.054	0.036
	0.080	0.096	0.064
	0.150	0.180	0.120

The increase in water level is a maximum of 50mm when a higher roughness value is applied and, conversely, the water level decreases by only approximately 50mm when the lower roughness coefficients are applied to the model during the 1% AEP event. As such, the model sensitivity to roughness variation is considered low.

The longitudinal section with the peak water levels for the high and low Manning's n values is displayed in Figure D0-11.

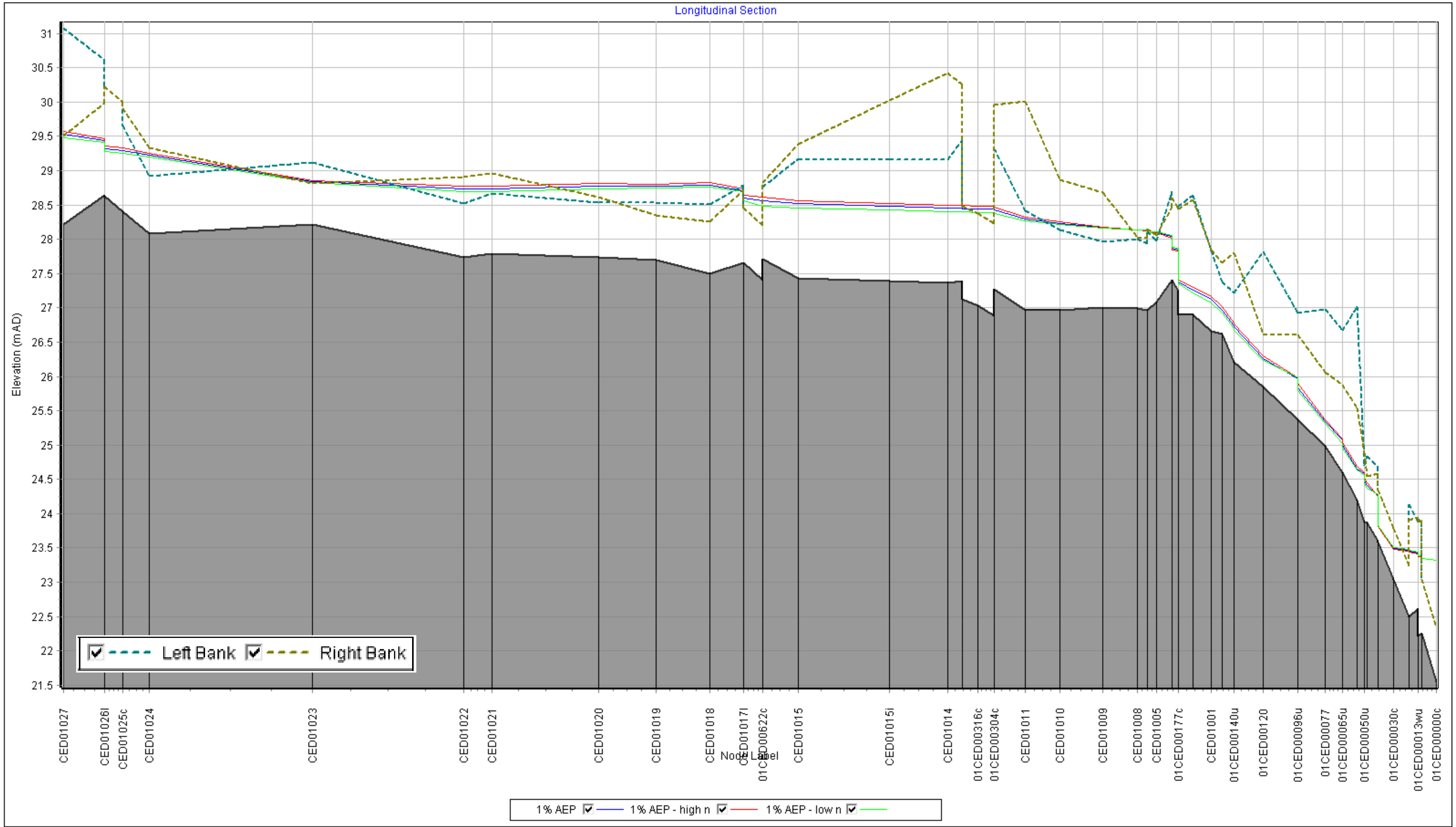


Figure D0-11: Cedarwood Stream - Peak 1% Water Level – Longitudinal Section high & low Manning's n

D.5.2 – Culvert Blockage

A series of blockage scenarios were run in the model in order to estimate a 67% blockage at the culverts under the railway line, at the Common's Road culvert and at culvert CED01004 during the 1% AEP event. Significant debris is currently accumulated in the channel and heavy vegetation growth is prevalent.

In the eventuality of a partial blockage at the northern railway line culvert (750mm diameter), the water level increases by 110mm upstream of the railway line and decreases by 100mm downstream. As previously mentioned, the flows are attenuated upstream of the railway line, with the two culverts serving as the sole connection between the upstream floodplain and the urbanized area downstream. Therefore, any reduction in the culvert capacity would lead to a decrease in the flow downstream. Conversely, any increase in culvert size would result in an increase in flow and an elevated flood risk to the properties downstream of the railway line.

The southern rail culvert (0.87m width x 1.4m height) blockage behaves similarly to the northern culvert blockage, but with a lesser impact (only 20mm increase in water levels upstream) since the majority of the flow is conveyed through the northern culvert. The southern rail culvert was originally depicted as 0.7m wide and 1m high in the baseline model. Following the updated survey collected in 2024, it was noted that its dimensions increased to 0.87m wide and 1.4m high (from hard bed to culvert soffit). This is attributed to channel clearance as well as the removal of a collapsed wingwall at this location.

The results of the blockage test at Common's Road culvert (1.35m diameter) indicate that the water level downstream decreases by 40mm. However, upstream, the water level increases by 170mm, which would create a flood risk at Cedarwood Grove.

The results of the blockage run on culvert CED01004 (1.2m diameter) show an increase in water level by 220mm, which results in overtopping the banks and forming a flow path towards the confluence with Shannon, impacting several properties, as displayed in Figure D0-15.

The longitudinal sections with the peak water levels for the blockage scenarios at structures are displayed in Figure D0-12, Figure D0-13 and Figure D0-14.

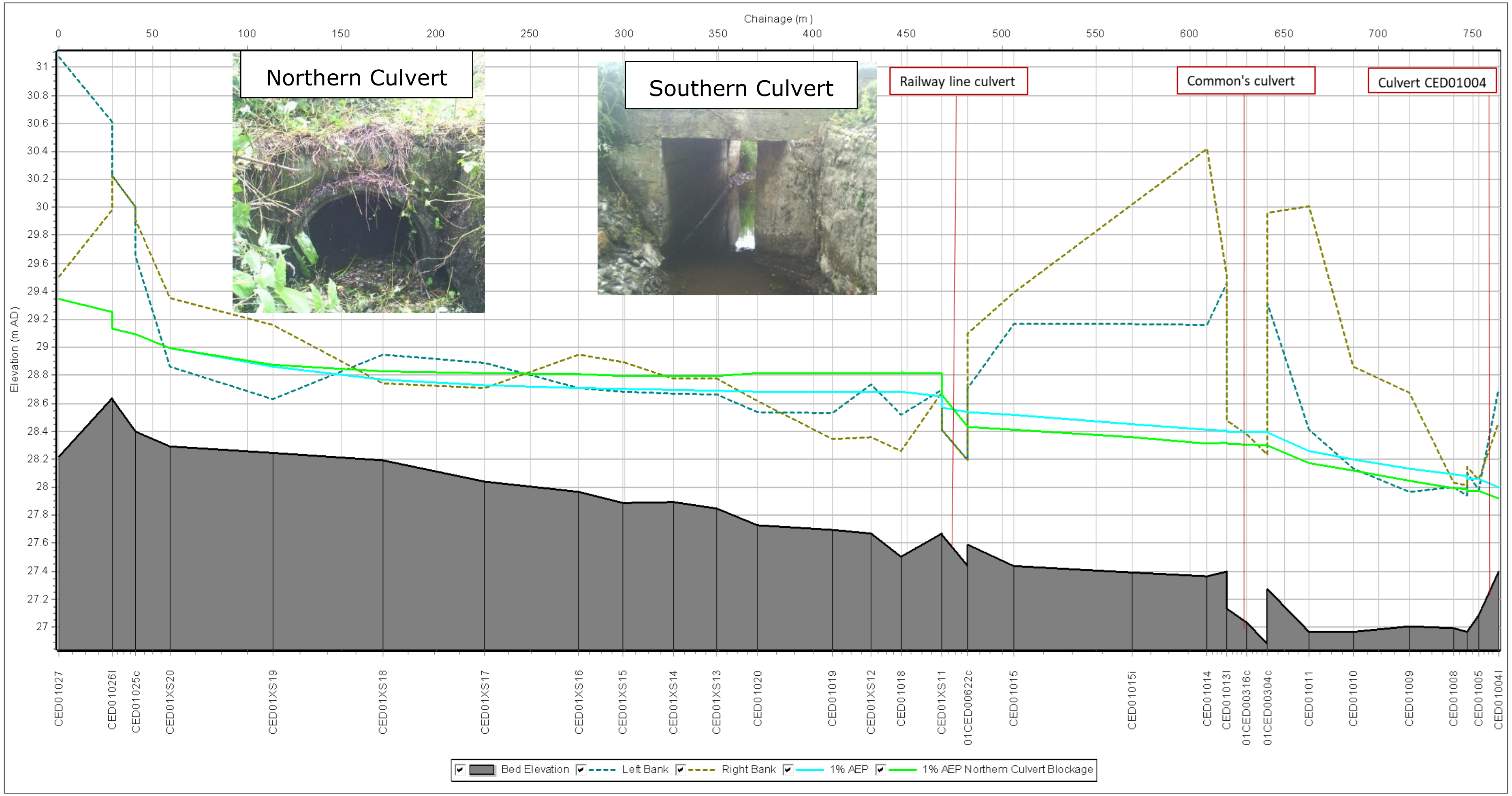


Figure D0-12: 1% AEP Longitudinal Section - Railway line culvert blockage

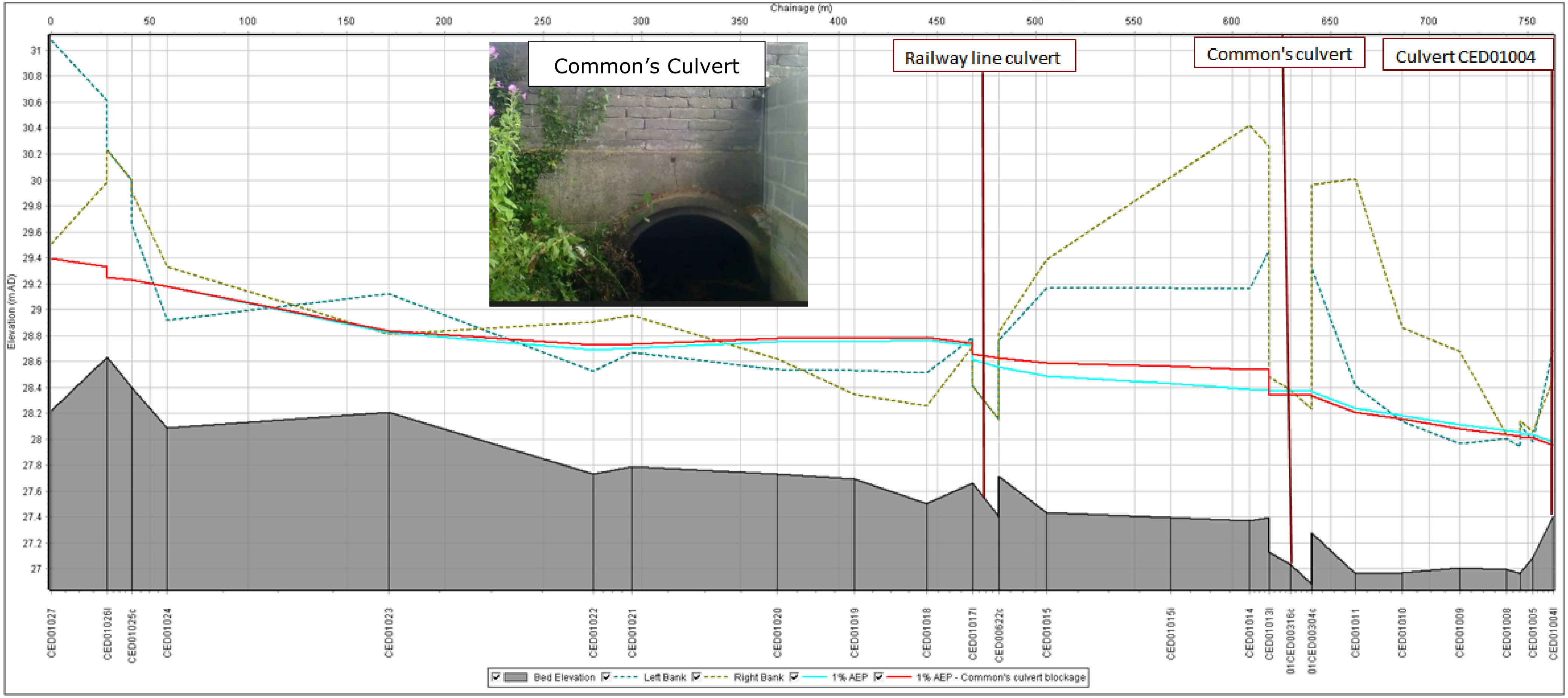


Figure D0-13: 1% AEP Longitudinal Section – Common's Road culvert blockage

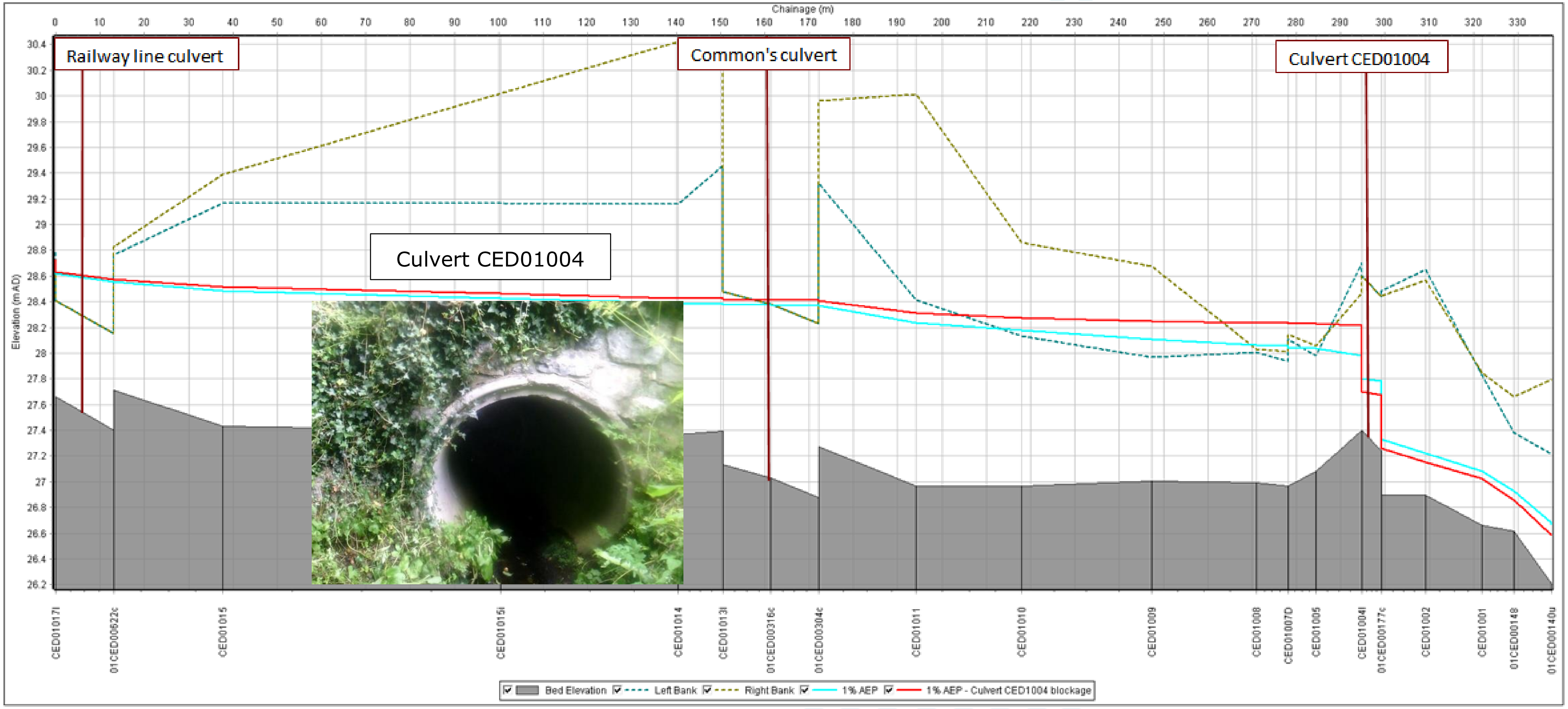


Figure D0-14: 1% AEP Longitudinal Section – Culvert CED1004 Blockage

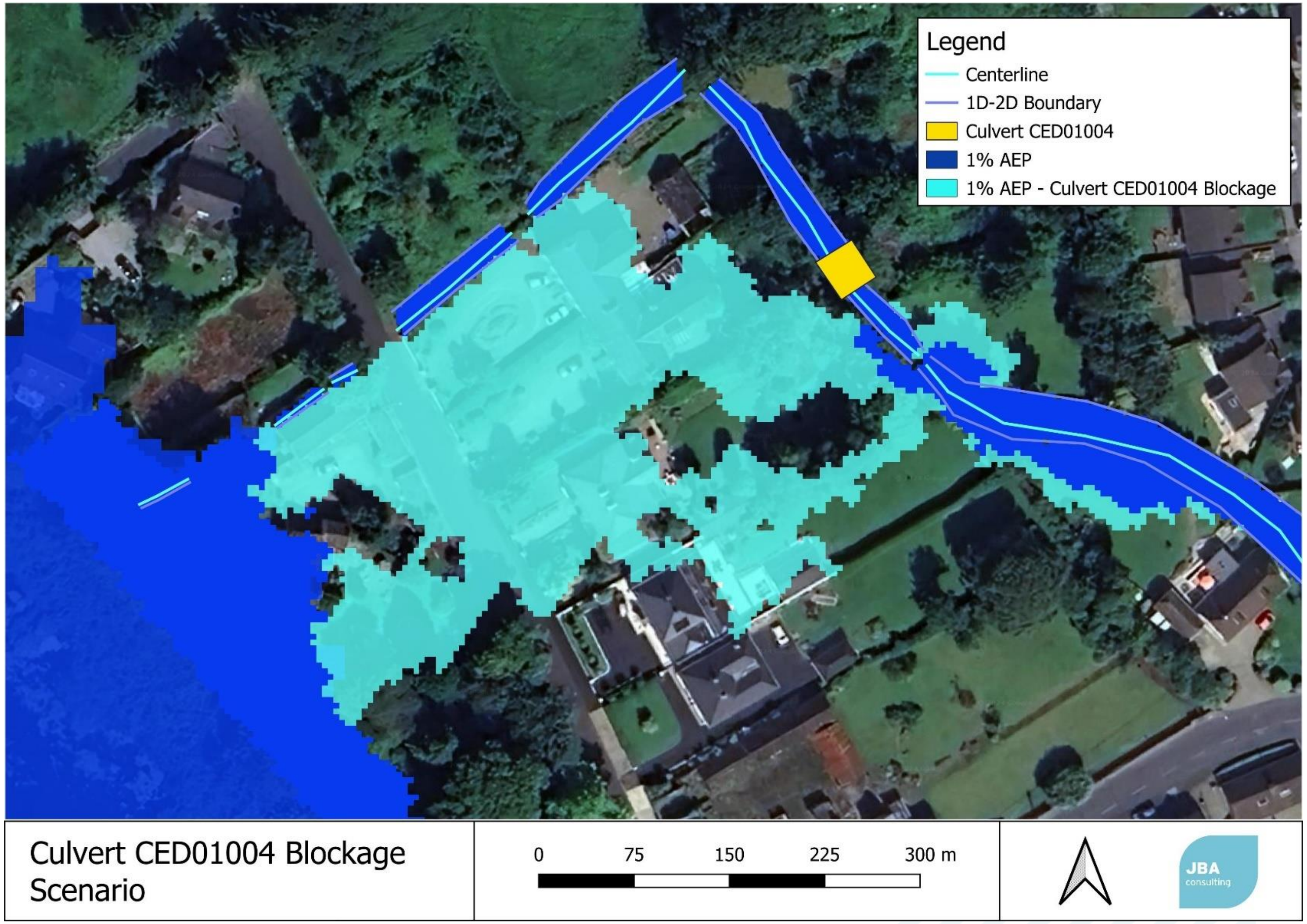


Figure D0-15: 1% AEP – Culvert CED01004 Blockage Scenario

D.5.3 – Flow Sensitivity and Climate Change

To assess the model's sensitivity to flow, the flows were increased by 20% and 30%, corresponding to the MRFS and HEFS climate change scenarios, respectively.

Water levels upstream of the railway line rise by 90mm under the MRFS scenario and by 130mm under the HEFS scenario. Downstream of the railway line, the increase ranges from 30 to 50mm for MRFS and from 40 to 80mm for the HEFS scenario.

The flood extents for the MRFS and HEFS scenarios are presented in Figure D0-16.

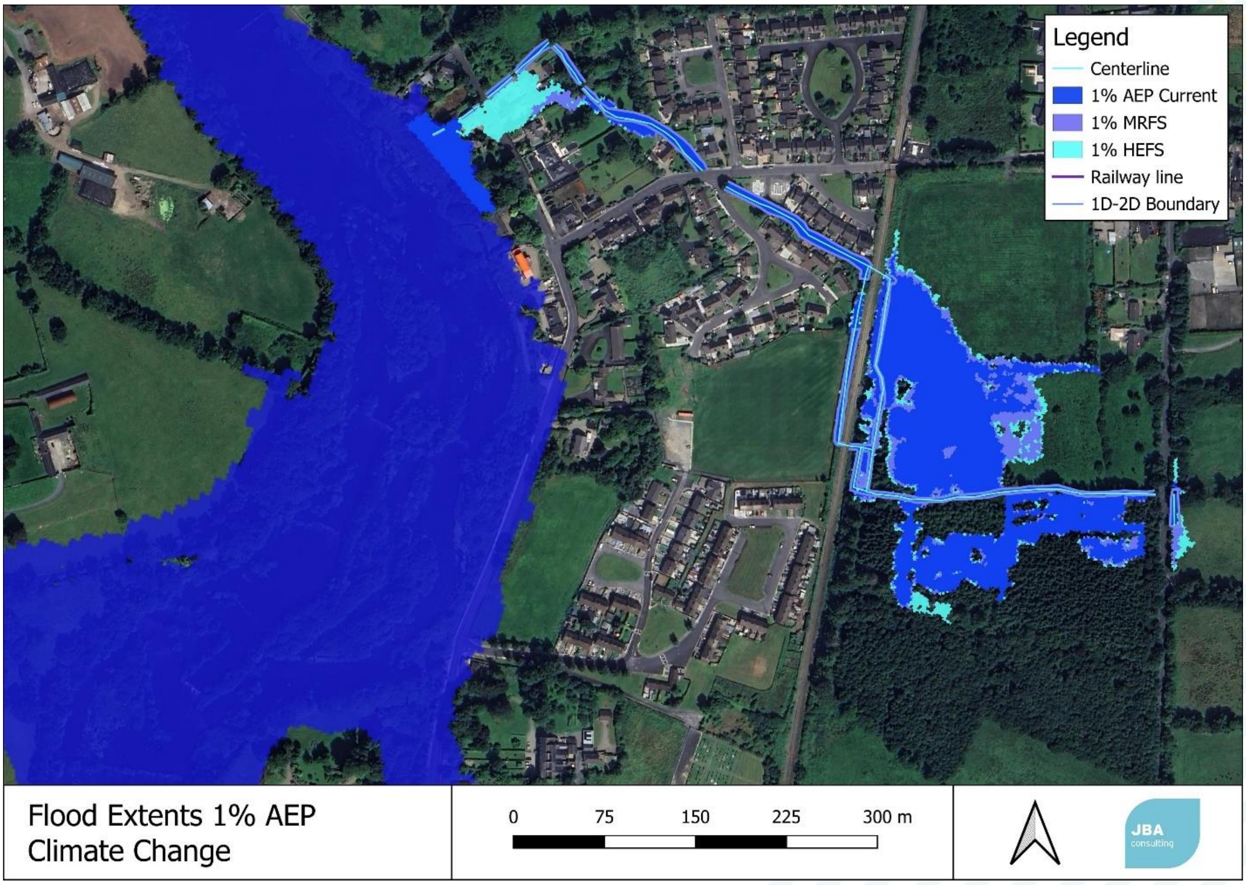


Figure D0-16: Climate Change

D.5.4 – Sandbags

During the 2024 site visit, sandbags were observed upstream of the railway line on the channel branch leading to the northern culvert, acting as a temporary flow constriction. These sandbags were not included in the design simulations, as they are expected to be dislodged during a significant flood event.

A sensitivity test was conducted with the sandbags in place, which indicated that flood levels would increase by 30mm in the forestry area and by 20mm downstream of the railway line. As such, a flow constriction within the forestry area would increase the water level both upstream and downstream of the railway line.

Figure D0-17 illustrates the location of the sandbags and the flood extents for the 1% AEP scenario with the sandbags included in the model.

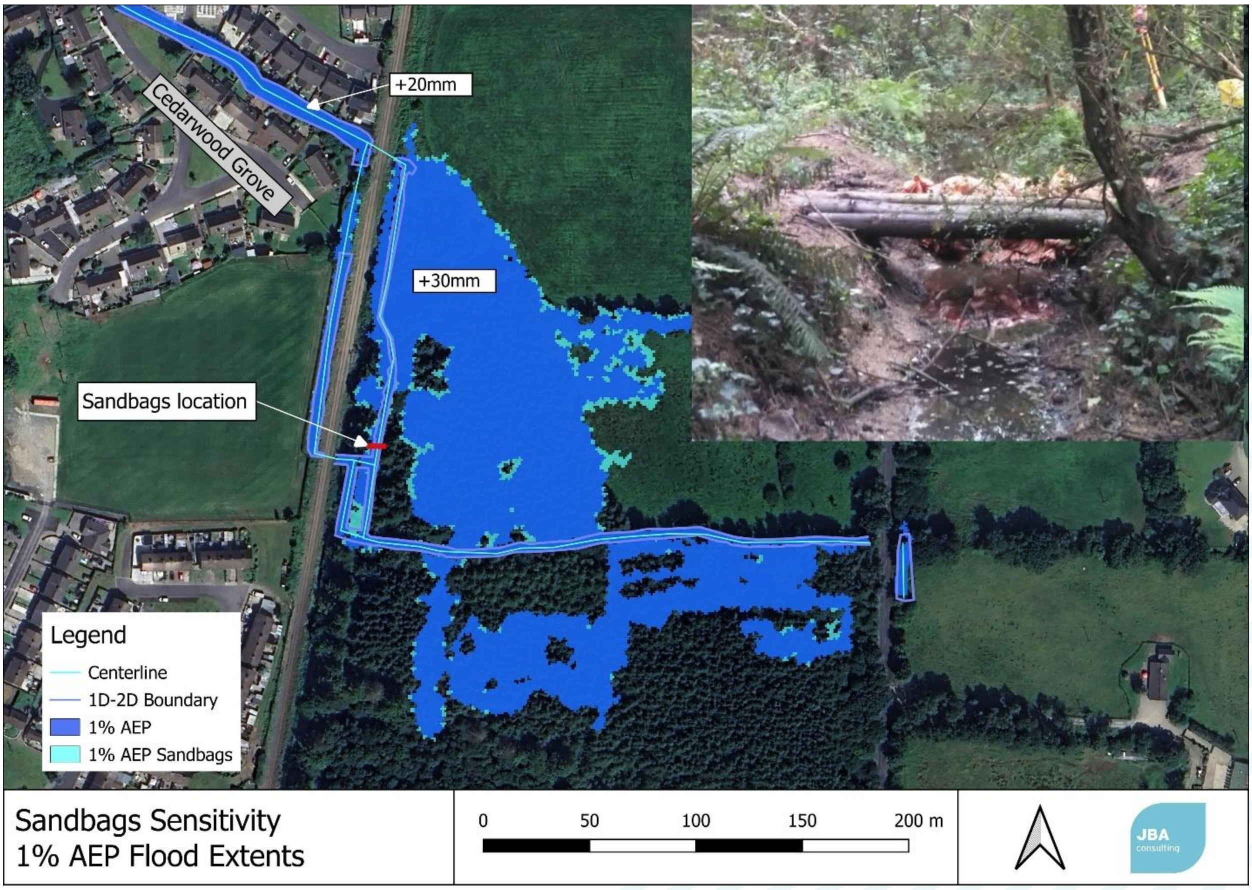


Figure D0-17: Sanbags Sensitivity Test

D.5.5 – Culvert Daylight

A test was conducted to assess the impact of daylighting the culvert running along the railway line, as shown in Figure D0-18. The results indicate that, if the culvert were daylighted, water levels would rise by 30mm downstream of the culvert and decrease by 30mm upstream.

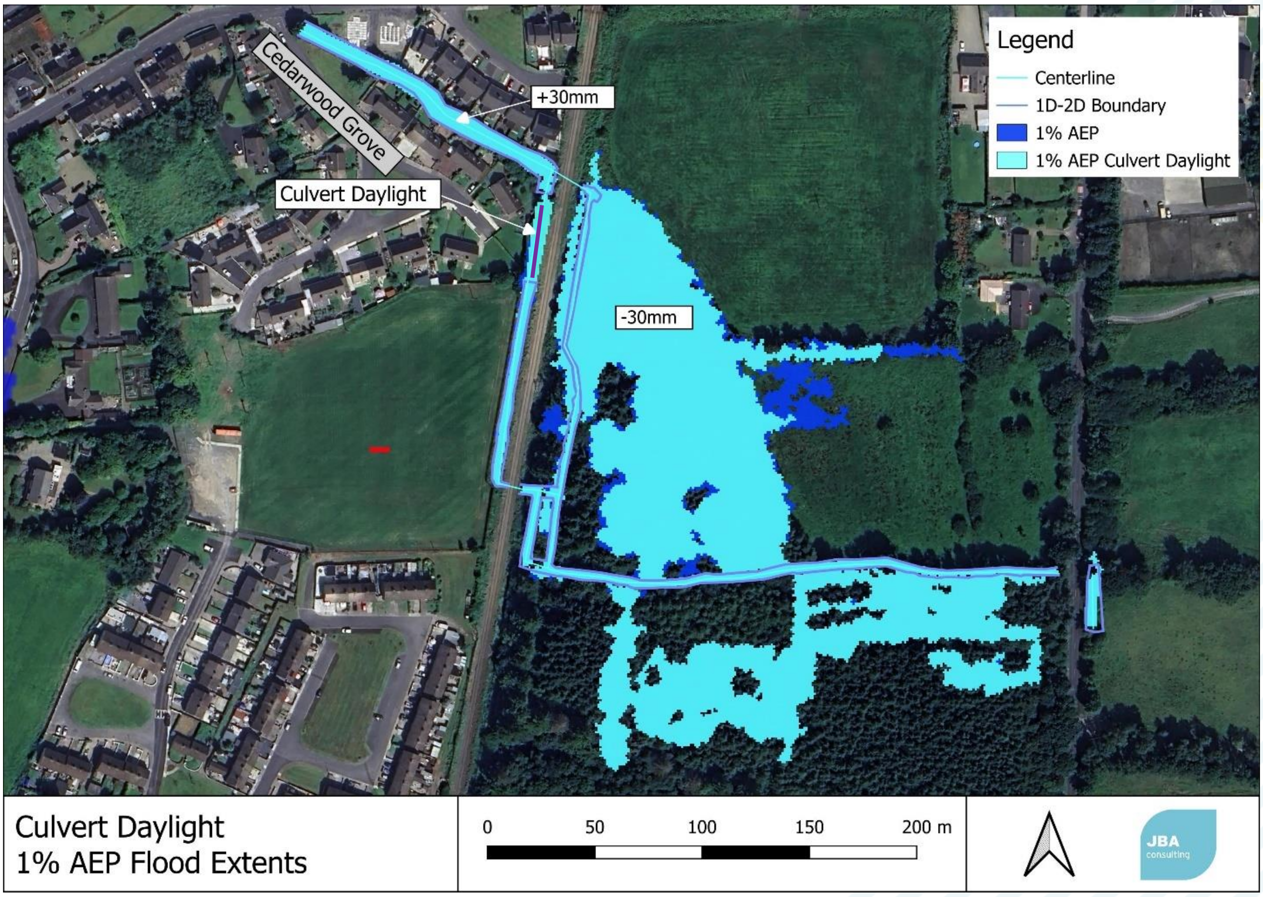


Figure D0-18: Culvert Daylight

D.5.6 – River Branches Sensitivity

A simulation was conducted to raise the bank levels in the upstream half of the forestry, forcing all flow through the southern culvert, as shown in Figure D0-19. The results indicate a 270mm increase in water levels, which would impact properties at Cedarwood Grove.

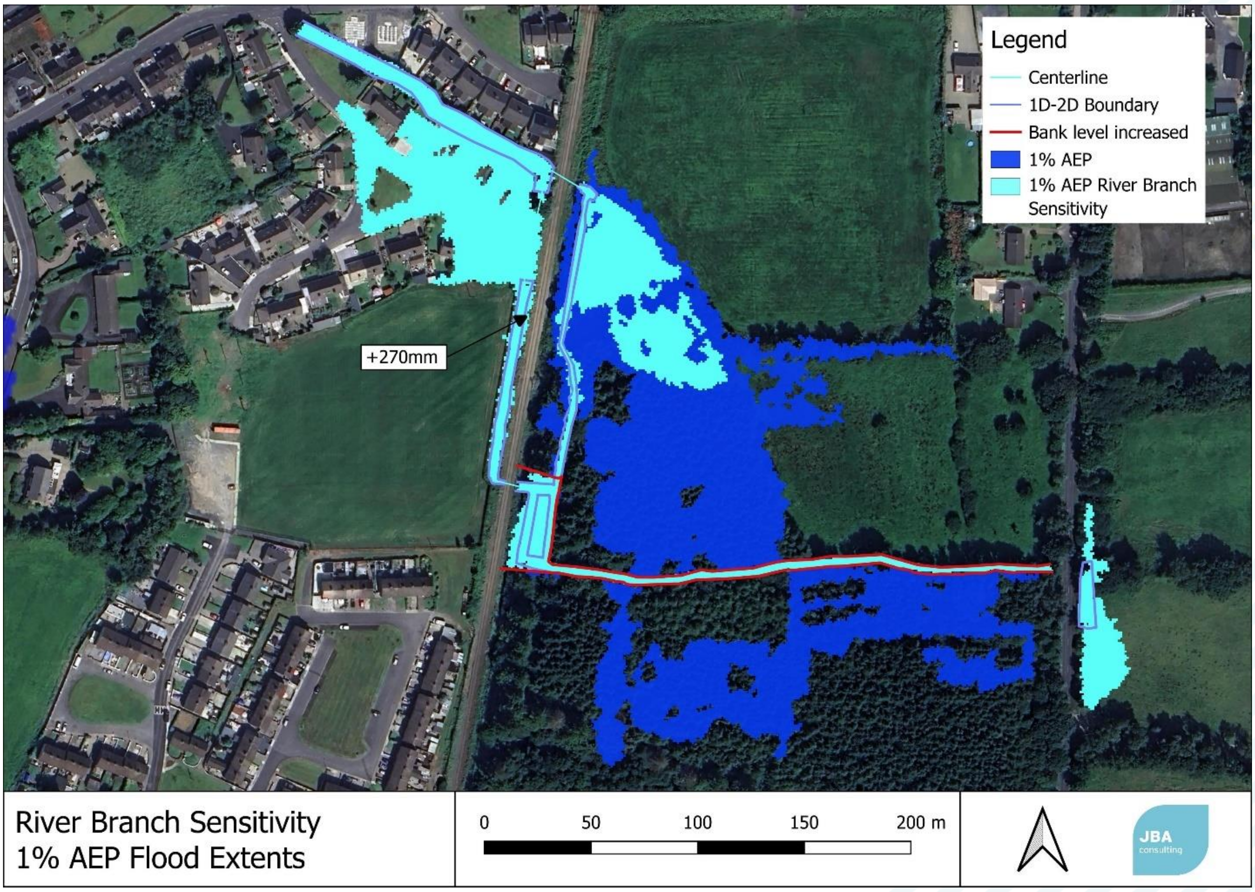


Figure D0-19:River Branches Sensitivity

D.5.7 – Southern Culvert Wingwall

As previously discussed, the wingwall of the southern culvert collapsed and was removed from the channel, increasing the culvert width from 0.7m to 0.87m. To assess the impact of this change, the model was run with the original 0.7m width. The results show a 10mm increase in water level upstream of the culvert and a 10mm decrease downstream. This indicates that the change in width is not highly sensitive to water levels. The sensitivity primarily lies in the southern culvert's elevated invert level, which remains 600mm higher than the northern culvert, even after the recent desilting works undertaken in 2023/2024.

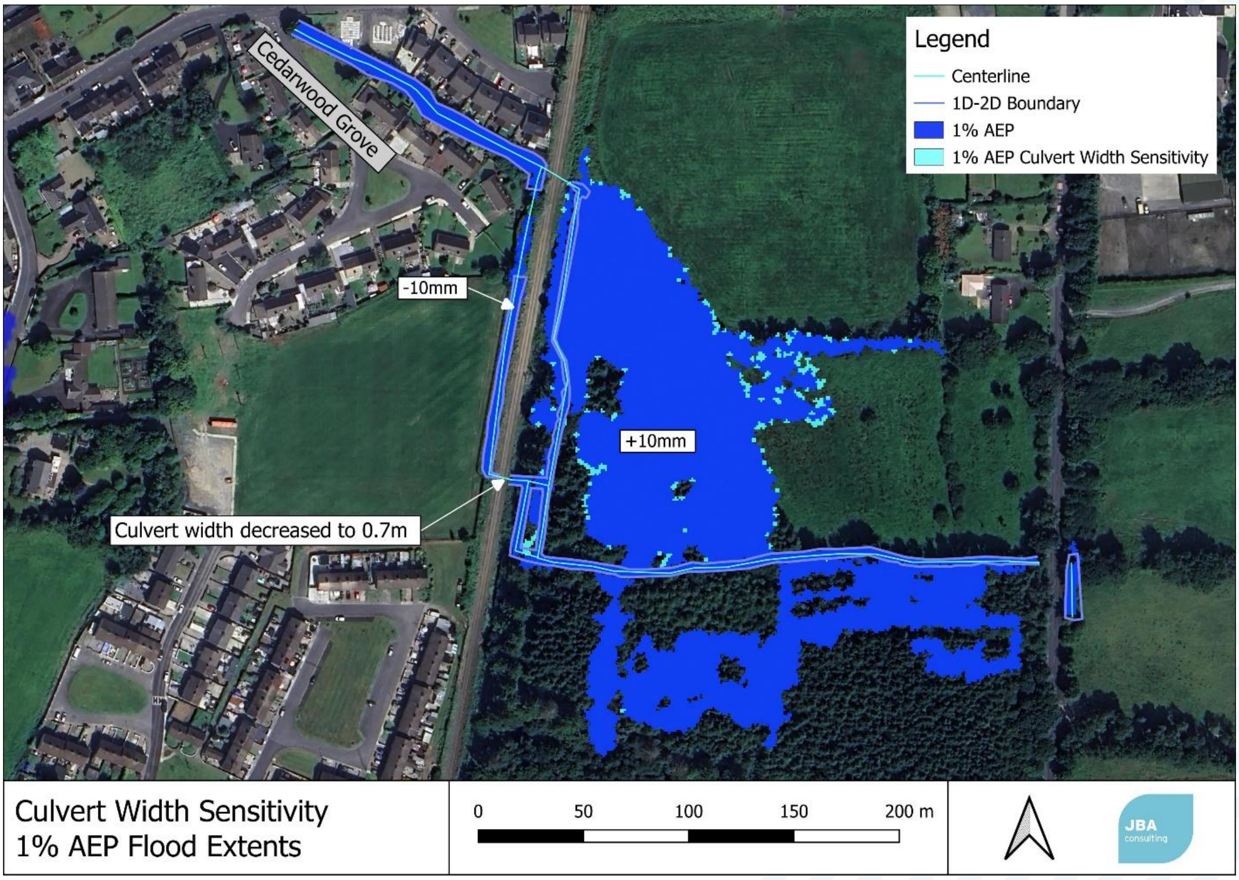


Figure D0-20: Southern Culvert Width Sensitivity

D.6 – Model Results

This section presents the peak water levels, peak flows, and flood extents for eight distinct return periods: 50%, 20%, 10%, 5%, 2%, 1%, 0.5%, and 0.1% AEP. It's important to highlight that areas with flood depths of less than 20mm have been excluded from the delineation of the flood extents for the flood maps provided in Figure D0-21 and Figure D0-22.

Table D0-4: Cedarwood Stream – Peak Water Levels [mOD]

Node label	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
CED01024	28.93	29.00	29.02	29.04	28.99	29.07	29.09	29.12
CED01015	28.23	28.33	28.36	28.40	28.45	28.50	28.55	28.61
CED01013	28.15	28.24	28.27	28.30	28.35	28.39	28.43	28.54
CED01007	27.85	27.92	27.95	27.99	28.03	28.07	28.11	28.18
01CED00077	25.24	25.28	25.30	29.32	25.34	25.36	25.39	25.44

Table D0-5: Cedarwood Stream – Peak Flows [m^3/s

Node label	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.1% AEP
CED01024	0.30	0.41	0.48	0.57	0.69	0.79	0.89	1.22
CED01015	0.33	0.43	0.47	0.52	0.59	0.66	0.73	0.84
CED01013	0.33	0.43	0.47	0.53	0.60	0.66	0.73	0.84
CED01007	0.34	0.44	0.48	0.53	0.60	0.67	0.74	0.81
01CED00077	0.38	0.50	0.55	0.60	0.68	0.76	0.84	0.98

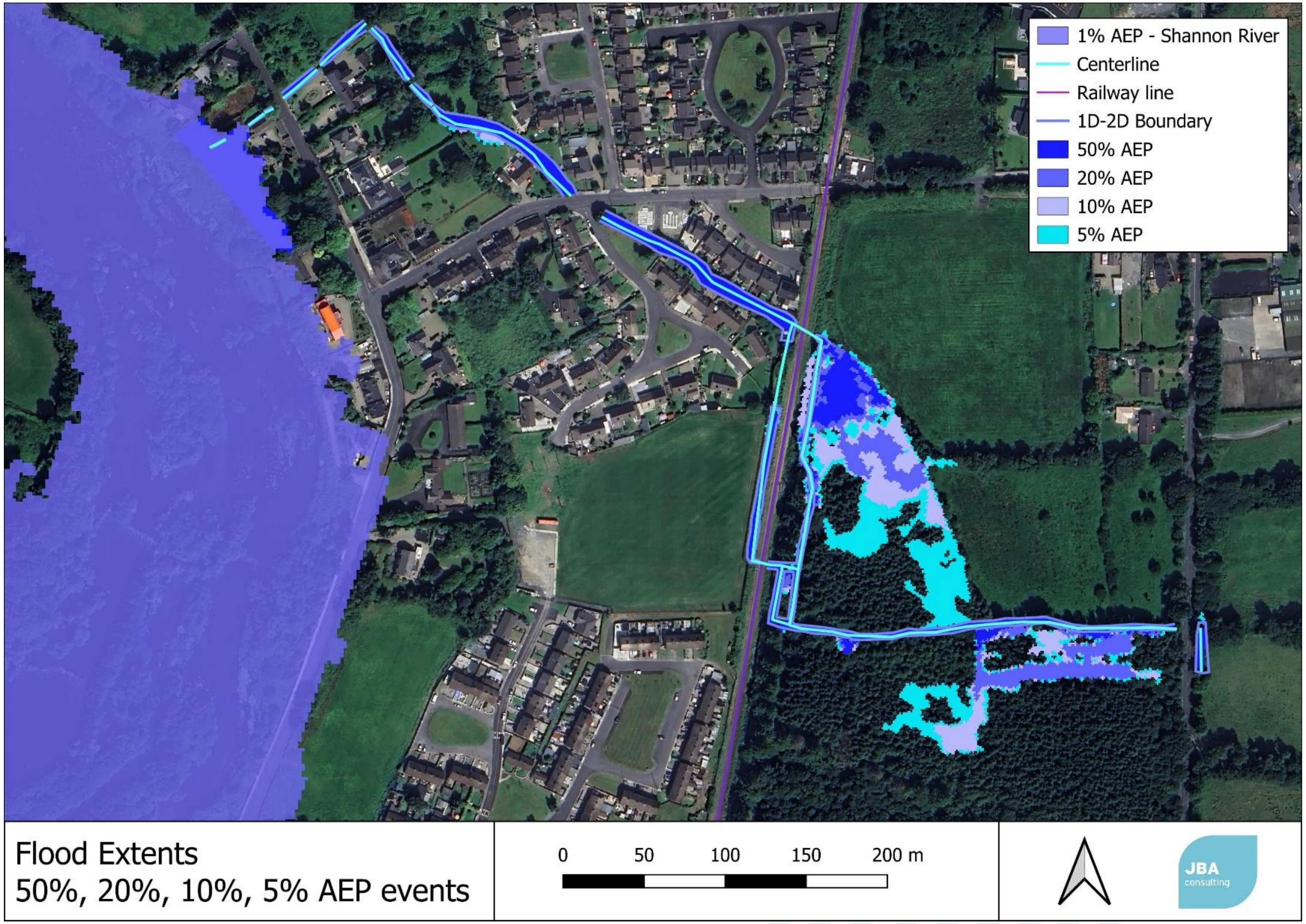


Figure D0-21: 50%, 20% 10%, 5% AEP Flood Extents

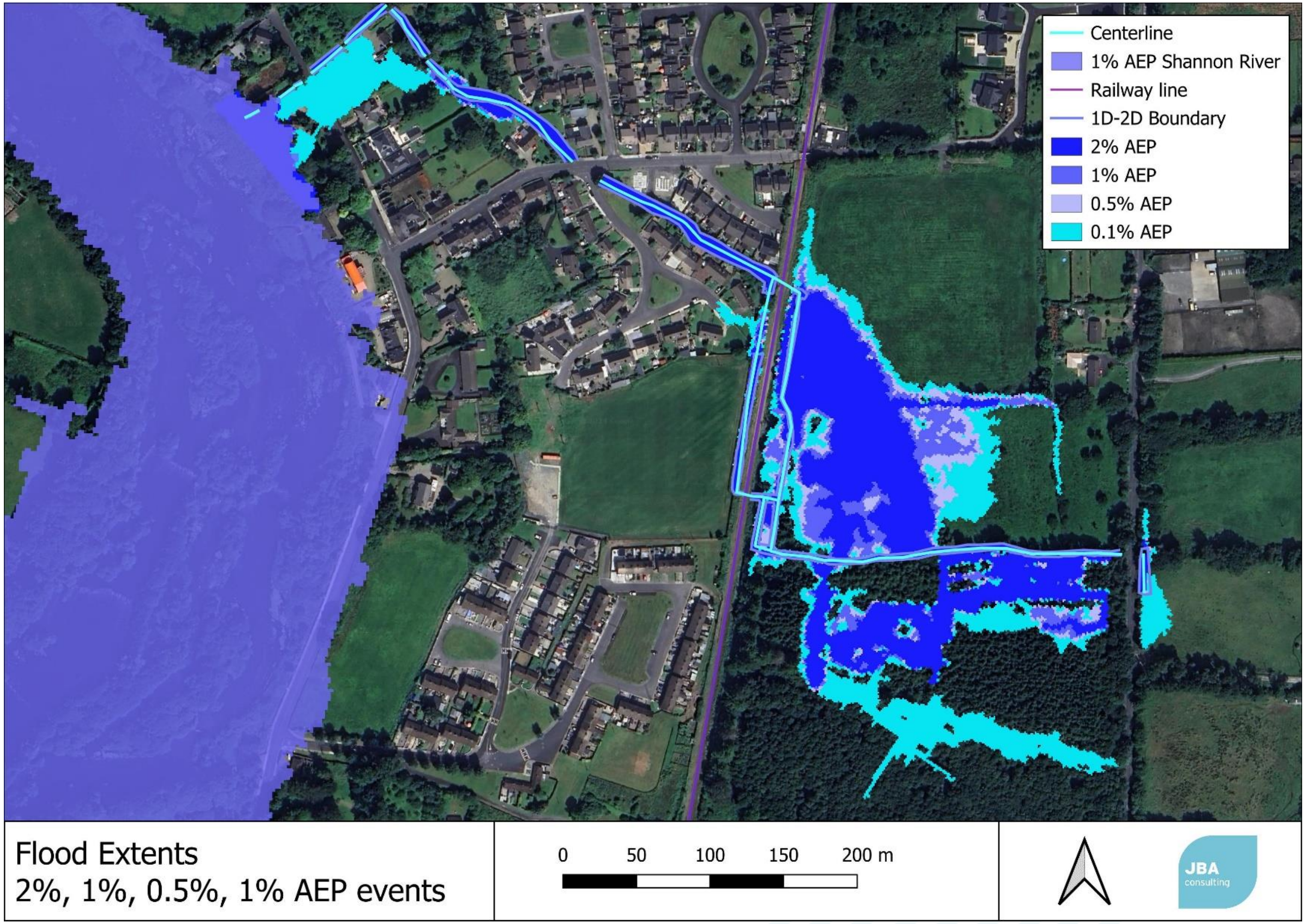


Figure D0-22: 2%, 1% 0.5%, 0.1% AEP Flood Extents

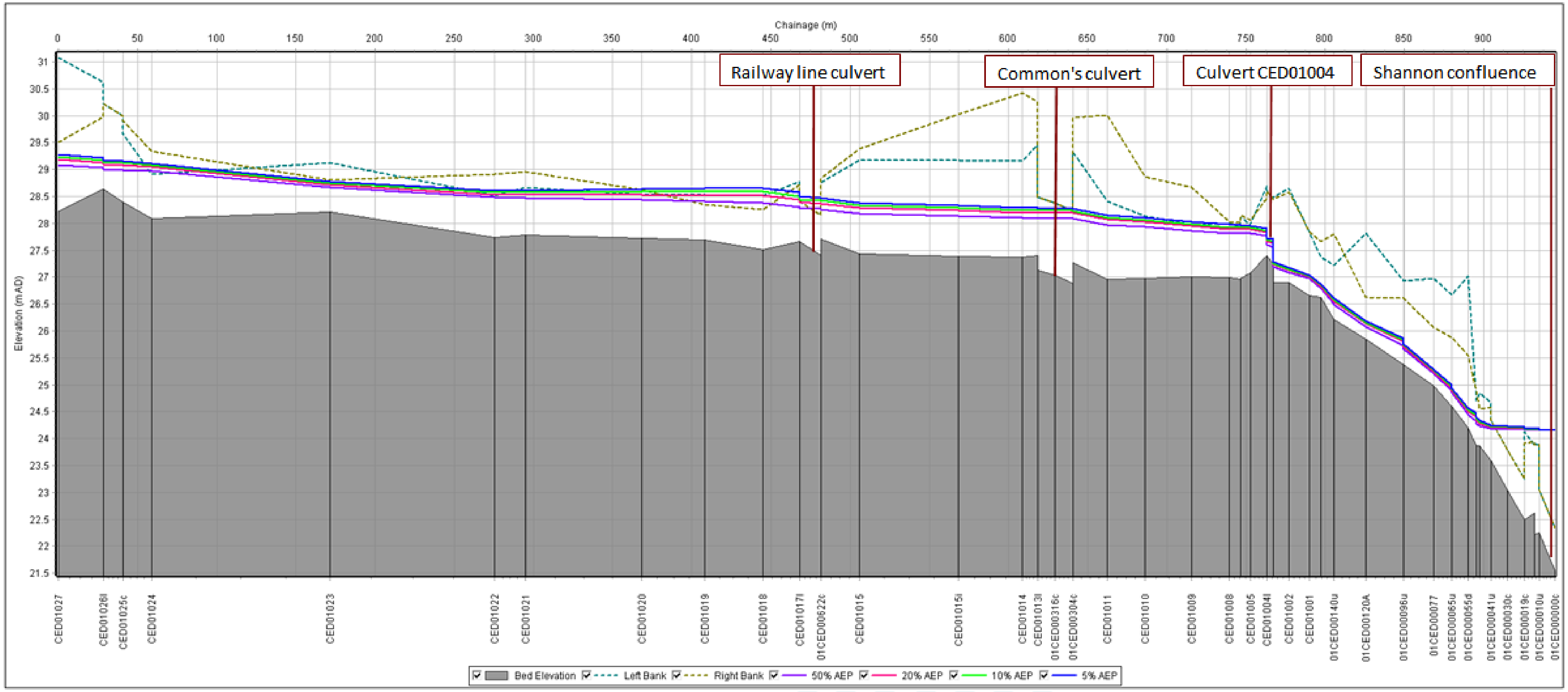


Figure D0-23: Water Levels 5%, 10%, 20% and 50% AEP Events – Longitudinal Section

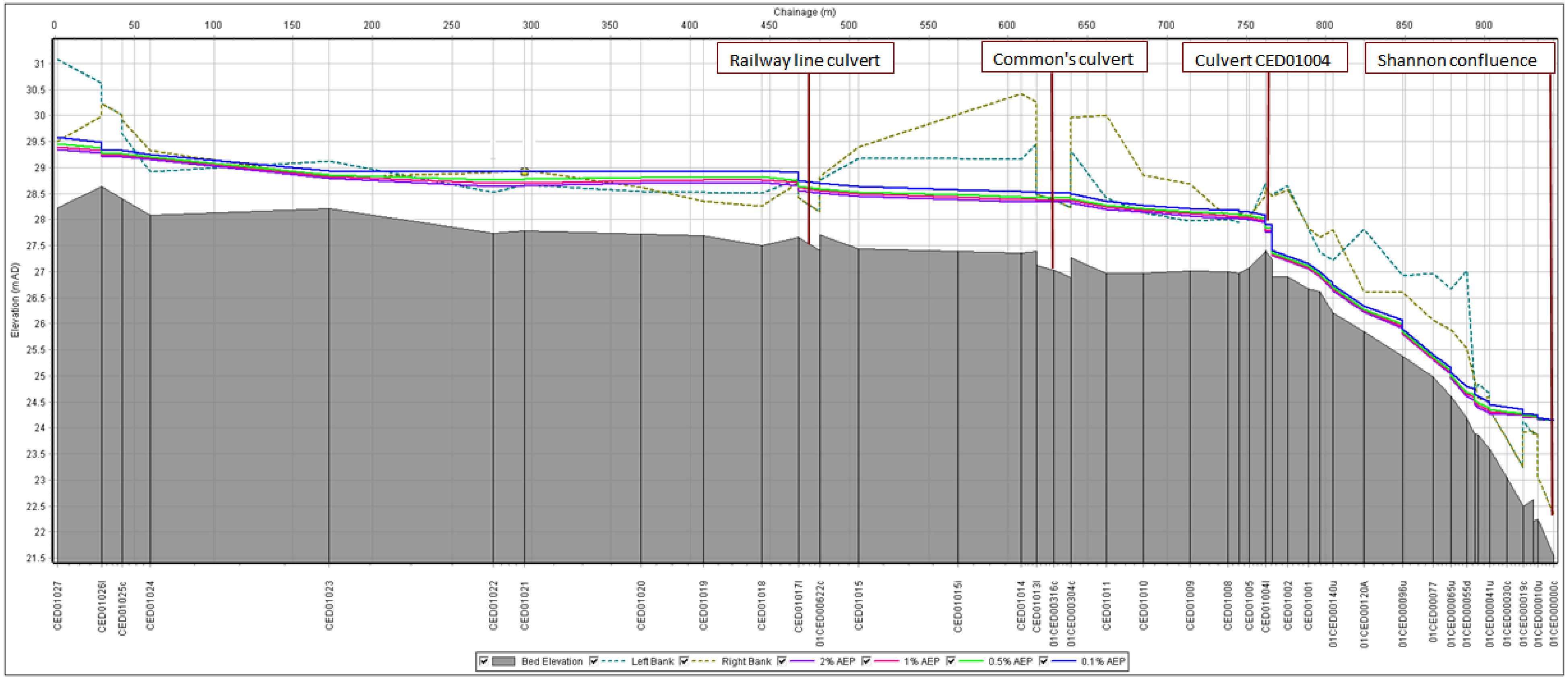


Figure D0-24: Water Levels 0.1%, 0.5%, 1% and 2% AEP Events – Longitudinal Section

D.7 – Measure Testing

The Cedarwood Stream is an urban watercourse, which has a number of crossings which cause a restriction in the hydraulic conveyance of flood waters. The stream benefits from natural attenuation in lands upstream of the railway. Any flood risk management measures will be focused on management of the vegetation along the watercourse and improvements in the hydraulic capacity of a number of culverts.

Freeboard is a term used to represent a factor of safety, between predicted flood level and either the top of the defence or the finished floor level or threshold of the property. The freeboard reflects the uncertainties in the water level computations which can be indicated by using the sensitivity analysis. The water levels were found to be generally insensitive to the standard uncertainties and assumptions made in the modelling and therefore a minimum standard freeboard of 300mm has been adopted. The provision of an adequate freeboard is used to assess the performance of measures and these are tested in the following sections.

Larger culverts under the railway line were considered as this was a significant control on flows downstream. However, this would exacerbate the flood risk downstream and potentially impact properties which would be contrary to the aims of the flood relief scheme and require a greater extent of flood works.

Additionally, it is important to emphasize that the predicted increase in water levels upstream of the railway line due to the railway line culverts is only 100mm during the 1% AEP event. Hence, even a water level decrease of 100mm (as a result of bigger culverts capacity), which would be similar to the 5% AEP event levels, would still result in extensive flooding of the upstream forestry areas. Refer to Figure D0-21 for the 5% AEP flood extents.

D.7.1 –Private Culvert – CED01004

Figure D0-10 (longitudinal section) illustrates that culvert CED01004, creates a flow constriction that results in the left bank overtopping during a 1% AEP event. This is a result of the narrow channel entrance to the culvert, its limited diameter and importantly the elevated nature of its invert.

To evaluate the impact of the culvert on water levels upstream, two model runs were conducted. Firstly, the culvert (based on its current size) was lowered by 400mm to align with the upstream and downstream bed levels. Secondly, the culvert was removed entirely for testing purposes, channel regraded and the narrow channel upstream widened.

Figure D0-25 depicts the flood extents resulting from the “culvert lowered” scenario, where the culvert was lowered to align with the bed level upstream and downstream. The results show that there is no overtopping of the left bank, thus indicating that this solution is effective in mitigating flood risks in the area.

The 1% AEP flood levels decrease from 27.98mOD to 27.55mOD right upstream of the culvert location.

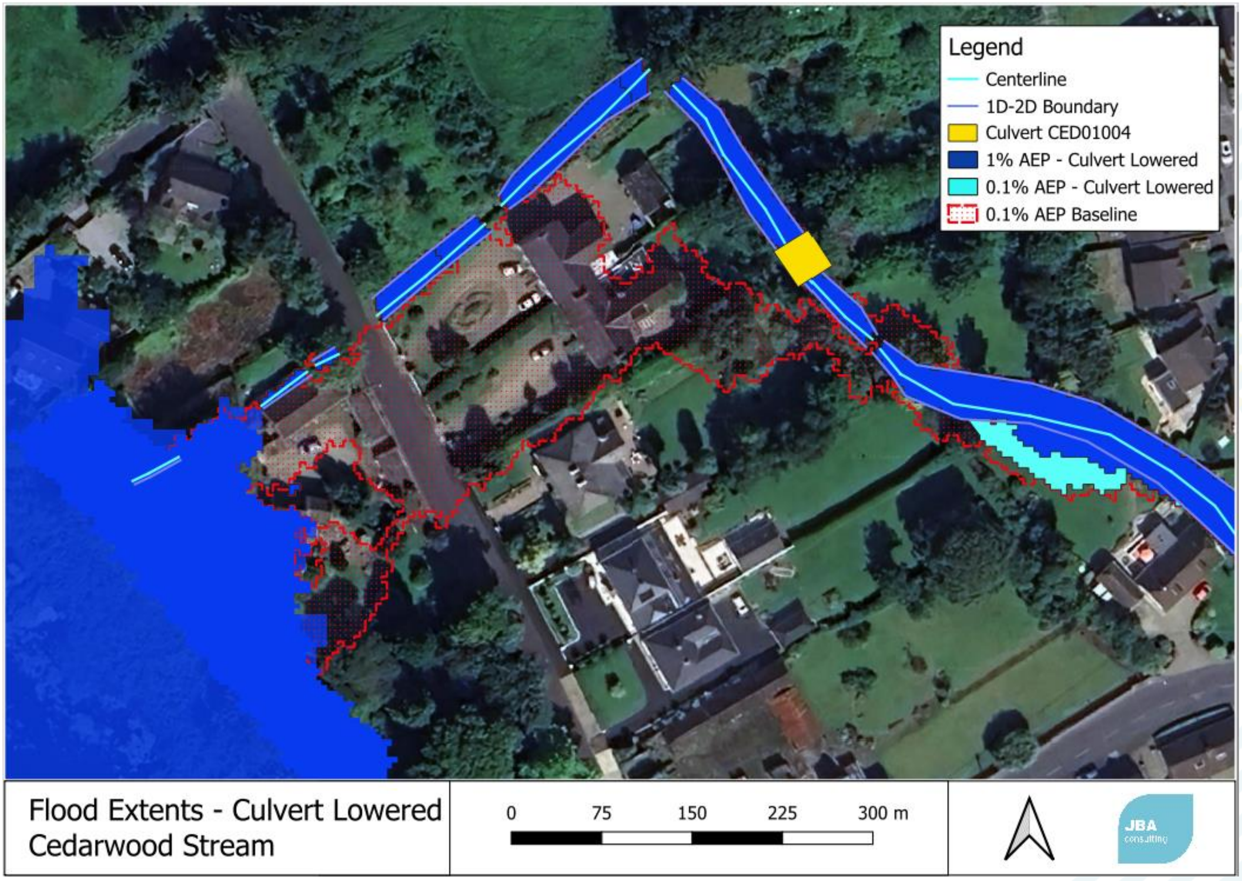


Figure D0-25: 1% and 0.1% AEP Flood Extents – "Culvert Lowered" Scenario

Based on the model results, the water levels immediately upstream of the culvert decreased by 420mm and 660mm when the culvert was lowered and removed, respectively. Notably, the 240mm difference between the two scenarios suggests that the culvert and the channel immediately upstream are under dimensioned. The longitudinal section presented in Figure D0-26 depicts the water level changes for both model runs, with the minimum 300mm freeboard being provided to the banks for the regrading option.

As discussed previously, the model shows high sensitivity in water levels at this location. A minor difference of just 20mm can dictate whether flooding occurs downstream, towards the confluence with the Shannon River, thereby affecting several properties due to left bank overtopping. Furthermore, the blockage test resulted in a significant rise in water levels, which had consequential impacts on properties located downstream.

Considering these sensitivities, it is proposed the installation of an expanded-width box culvert in the reshaped channel section, to improve water flow and reduce the risk of overtopping. It should be noted that the width and cover is limited at this location so a full Section 50 design approach may not be appropriate.

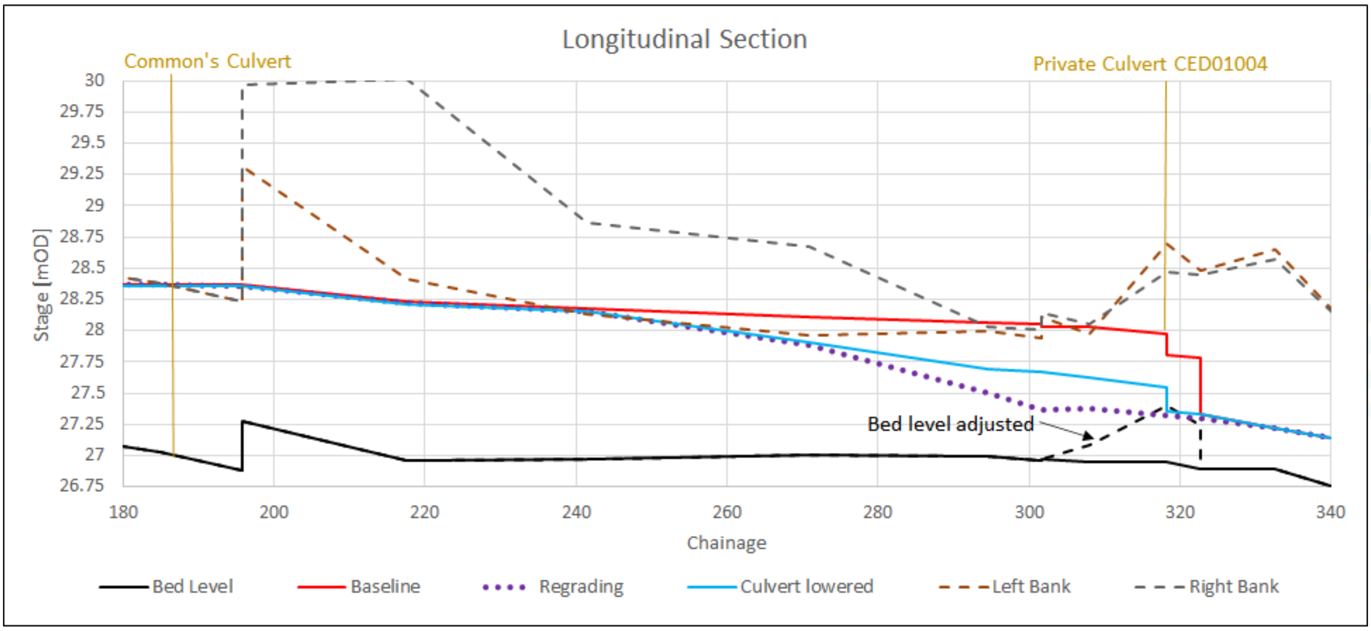


Figure D0-26: Longitudinal Section – Culvert Testing

D.7.2 – Cedarwood Grove

During the 1% AEP event, it was observed that the left bank of the river along Cedarwood Grove had a low freeboard. This was confirmed by model data abstracted at four reporting points (shown in Figure D0-27), with a minimum freeboard of 370mm predicted at the upstream end. To address this issue, several model runs were conducted to increase the freeboard in this area.

The following scenarios were tested:

- Regrading option (private culvert CED01004 replaced by a wider box culvert, with lower invert level and channel widening at the culvert entrance);
- channel maintenance on a 80m reach downstream of Common's Road culvert;
- culvert regrading combined with channel maintenance upstream.

The channel maintenance scenario was represented by a reduction in Manning's n downstream of Common's culvert from 0.15 in the baseline scenario to 0.04, to reflect the effects of the maintenance, while the Manning's n along the Cedarwood Grove was reduced from 0.055 to 0.045, which can be achieved by local tree and bush removal in the area.

It is important to note that ongoing channel maintenance, rather than a one-time removal of vegetation, would be required to sustain these reduced Manning's n values and maintain flow efficiency over time.

Figure D0-27 – Cedarwood Grove Reporting Points and Proposed Channel Maintenance Area

Table D0-6 illustrates the results of the various model runs aimed at increasing the freeboard along the left bank of the river in Cedarwood Grove during a 1% AEP event.

The channel maintenance scenario shows a freeboard within the 450mm-1200mm range. In the regrading scenario, the freeboard ranged between 400mm and 1080mm, with only 30mm improvement from the baseline scenario.

The combined scenario of regrading and channel maintenance showed the highest increase in freeboard, ranging from 490mm to 1260mm. These findings suggest that a combination of culvert CED01004 replacement to a wider box culvert, with lower invert level, channel widening at the culvert entrance and channel maintenance upstream and downstream of Common's Road culvert is an effective solution for increasing the freeboard and mitigating flood risks in the area.

Table D0-6: Peak Water Levels and Freeboard (Fb) at properties in Cedarwood Grove [mOD/m]

Rep Point	Floor Level	Baseline	Fb	Regrading Option	Fb	Channel Maintenance	Fb	Regrading Option + Plus Channel Maintenance	Fb
1	28.92	28.55	0.37	28.53	0.39	28.44	0.48	28.40	0.52
2	28.98	28.50	0.48	28.49	0.49	28.36	0.62	28.32	0.66
3	29.34	28.48	0.86	28.47	0.87	28.32	1.02	28.27	1.07
4	29.50	28.45	1.05	28.44	1.06	28.29	1.21	28.21	1.29

The peak water levels resulting from the four scenarios analysed can be visualized Figure D0-28. This figure provides a graphical representation of the model runs conducted to increase the freeboard along the left bank of the river in Cedarwood Grove during a 1% AEP event. The scenarios include regrading, channel maintenance, regrading combined with channel maintenance and the baseline scenario. The graph allows for a direct comparison of the peak water levels resulting from each scenario and can be used to inform decisions regarding the most effective approach for mitigating flood risks in the area.

Whilst it has been noted that a blockage scenario at Common's Road increases water levels at Cedarwood Grove, the maintenance works planned would reduce this risk significantly, velocities are low to convey debris in this upper reach to the culvert and hence no allowance has been included in the design for culvert blockage. No specific debris control measures such as screens are proposed.

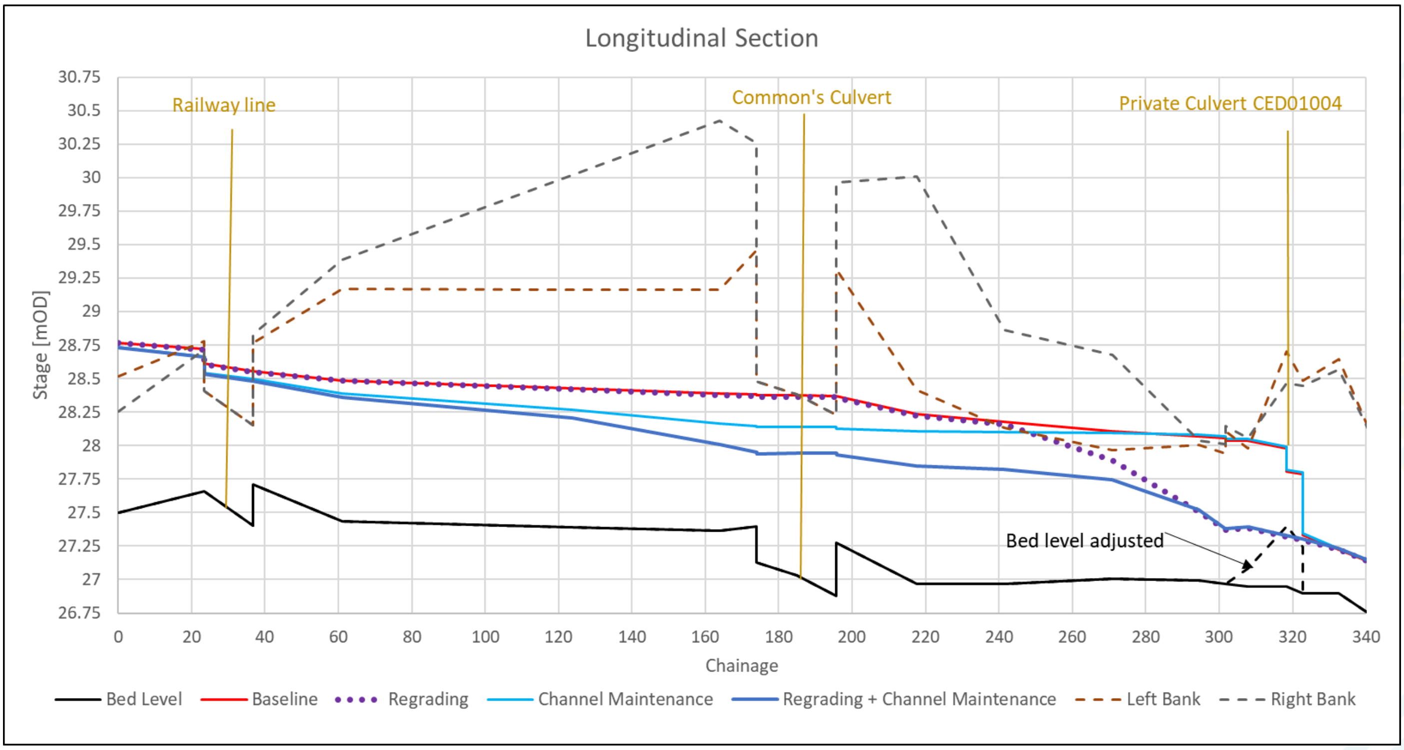


Figure D0-28 –Longitudinal Section - 1% AEP Peak Water Levels

D.7.3 – Cedarwood Stream - Downstream

This section presents the results for the downstream end of Cedarwood Stream, specifically at its confluence with the Shannon River. As stated previously, the model's downstream boundary uses a fixed water level-time (HT) model boundary, which makes use of the peak water level extracted from the primary Shannon model during the 1% AEP event. This occurs while the Ardnacrusha Power Station operates under standard conditions. The effect of water level variation on the Shannon River was also evaluated by testing a lower water level, corresponding to a 10% AEP event (shown as Low Shannon).

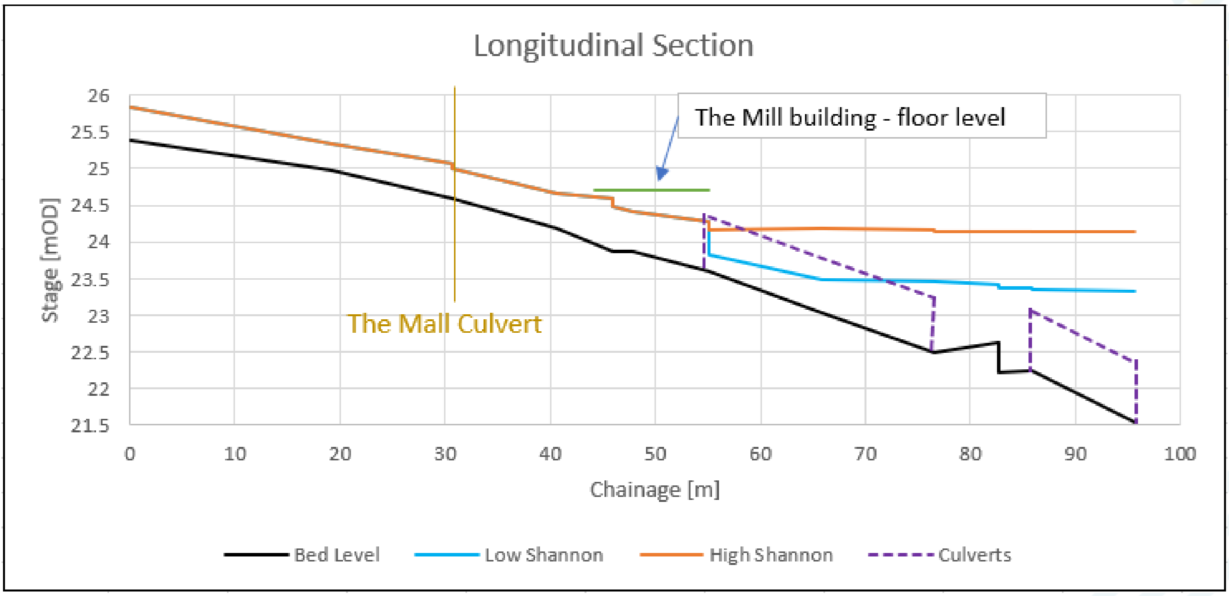


Figure D0-29: Longitudinal Section - Cedarwood Stream Downstream

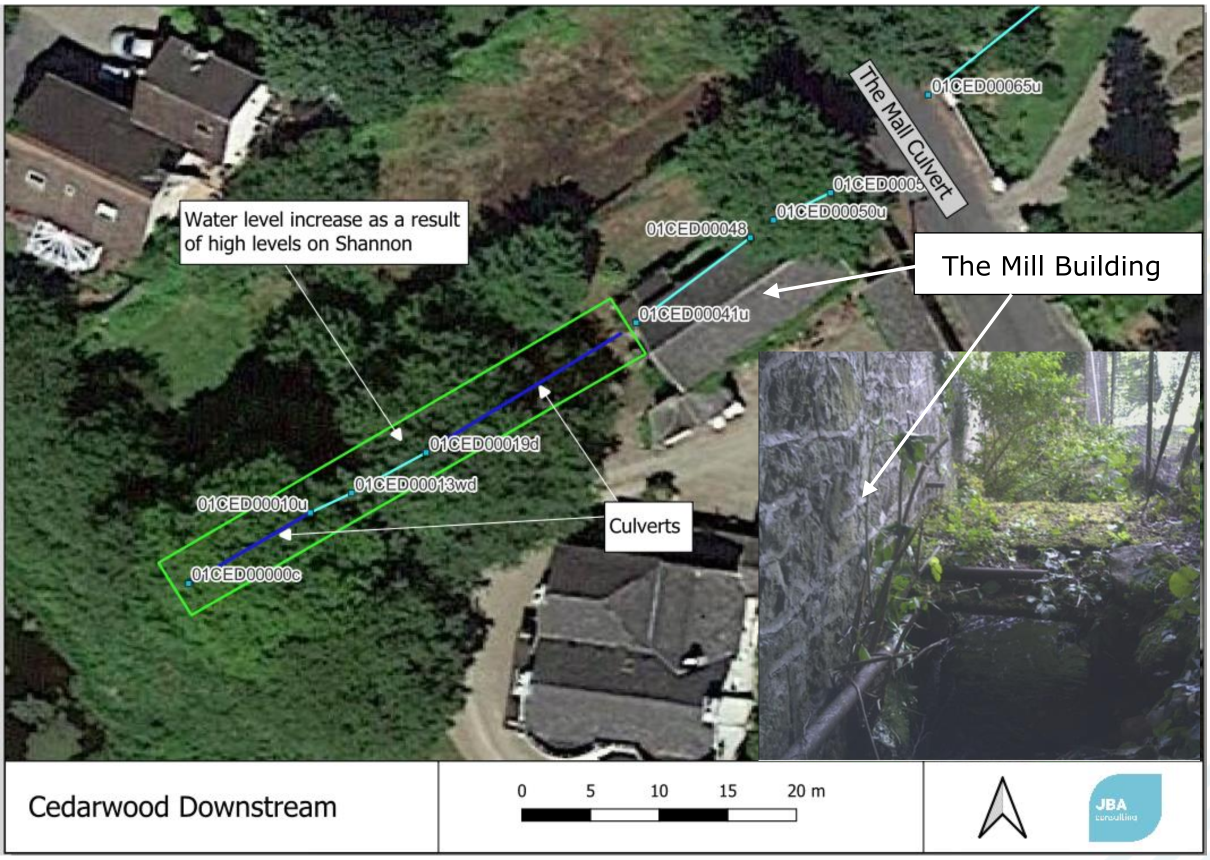


Figure D0-30: Cedarwood Downstream

The longitudinal section and plan view show that the 1% AEP water levels on the Shannon River impacts on a reach of 40 metres when compared with the 10% AEP water levels on Shannon. Within this area, there is one section of open channel between the culverts measuring 10 meters and two culverts that account for the remaining 30 meters.

The floor level of the Mill building is 24.71mOD and the building is not impacted by the variations of water level on the Shannon River during the 1% AEP event on the Cedarwood Stream, as displayed on the Longitudinal Section in Figure D0-29. No works are required at the Mill Building, except external lime mortar rendering of the external wall that forms the left channel side (the right-hand bank is not at risk of flooding, with minimum 400mm freeboard available for the 1% AEP event on Cedarwood). The outfall into the Shannon is proposed to remain open, without a flap valve.

The existing pipework from the open section to the Mill building is pressurized by approximately 1m, therefore it is recommended to investigate the joints during the design phase and remediation undertaken if necessary.

D.8 - Recommended works

The recommended works for reducing the flood risk and improving the flow conditions on Cedarwood include:

- Lowering the inlet level of culvert CED01004 to align with the channel downstream.
- Increasing the size of the culvert CED01004 and width of channel entrance
- Clear out trees, bushes and silt upstream from CED01004 to Common's Road, with a focus on established trees and bushes
- Clearing out trees, bushes and silt from the railway line culvert to Common's Road.
- Establishing a narrow dry weather flow channel and creating low berms along the entire length of the maintained channel to encourage self-cleaning conditions
- Creating silt deposition zone just downstream of Common's Road bridge that can be easily trapped and removed. It is suggested that the trees on the left-hand bank are removed and the channel widened.
- Introduction of new culvert in Grange House, diverting the stream around the existing open channel section. This intervention is to limit impacts on the cultural/heritage aspects of Grange House, and are not required hydraulically to increase or control conveyance. It has been included in the testing regime.
- Conducting inspections and annual vegetation and silt removal

The implementation of these works will improve drainage of the Cedarwood Stream and increase velocities to allow flushing of sediment through the slack gradient reach, as shown in the graphs below for the 50% AEP event. This will reduce the potential for vegetation growth and serve as a deterrent to fly-tipping, thereby reducing the risk of blockages

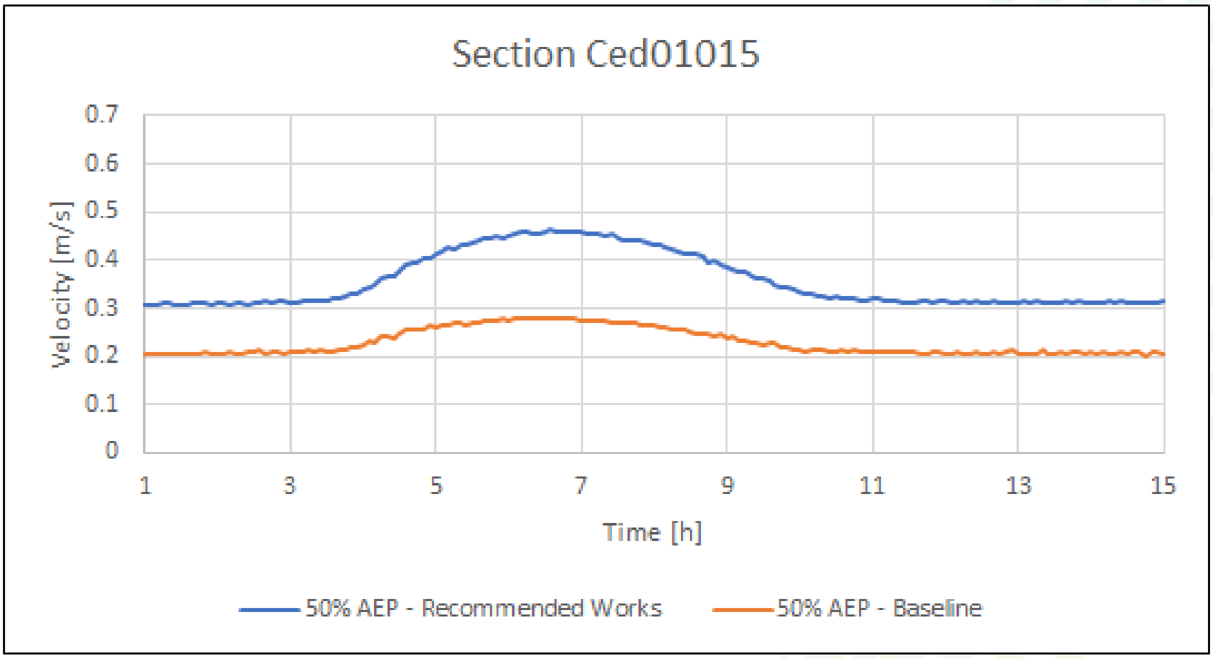


Figure D0-31: 50% AEP Velocities – Section Ced01015 (Downstream of railway line)

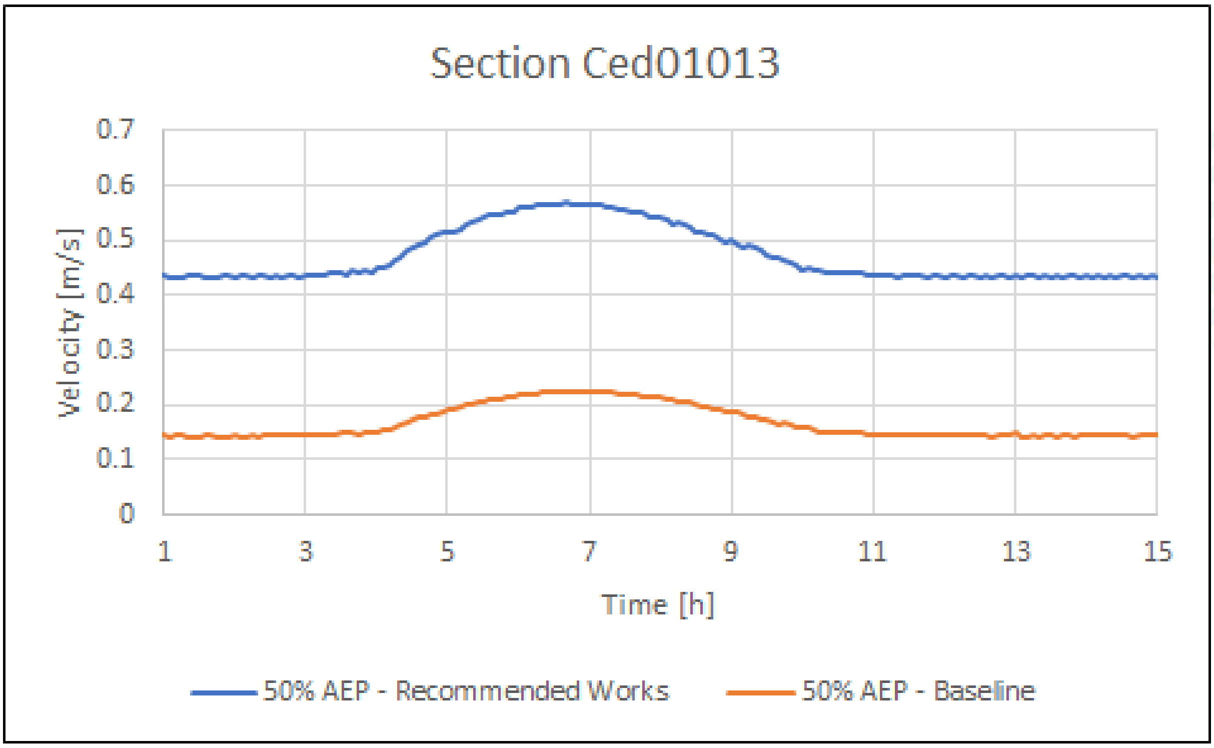


Figure D0-32: 50% AEP Velocities – Section Ced01013 (Cedarwood Grove)

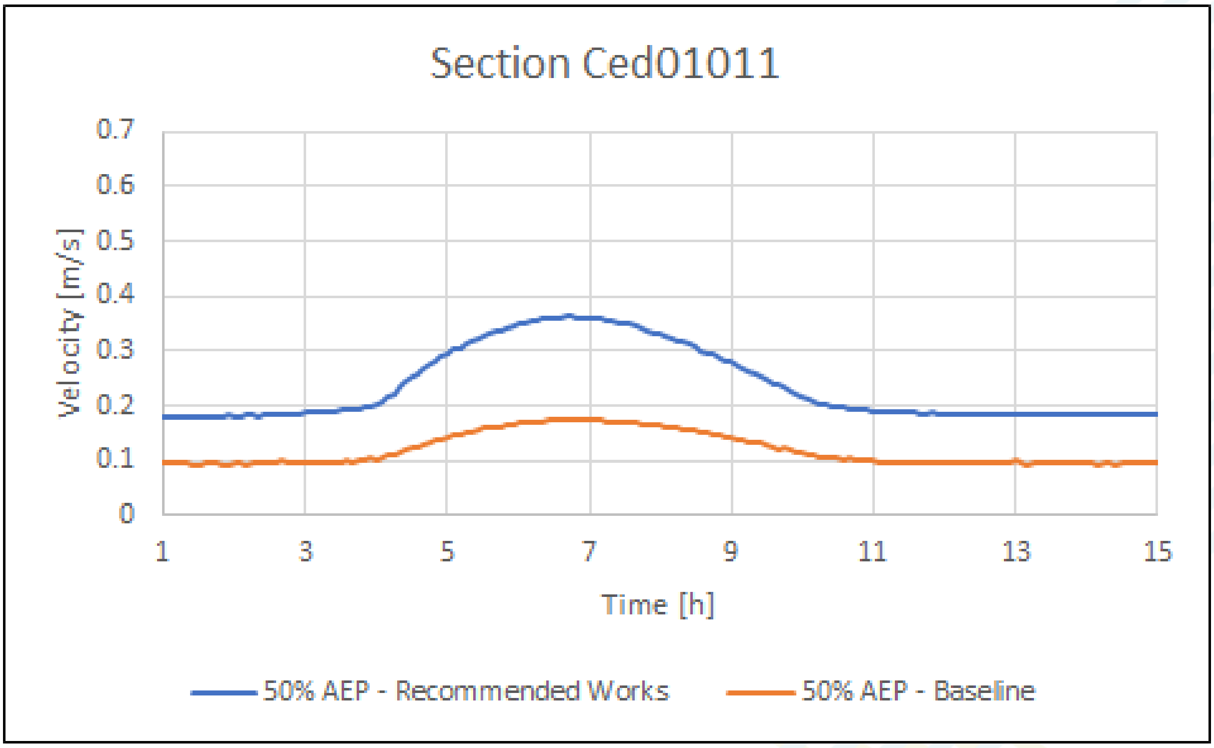


Figure D0-33: 50% AEP Velocities – Section Ced01011 (Downstream of Common's Road)

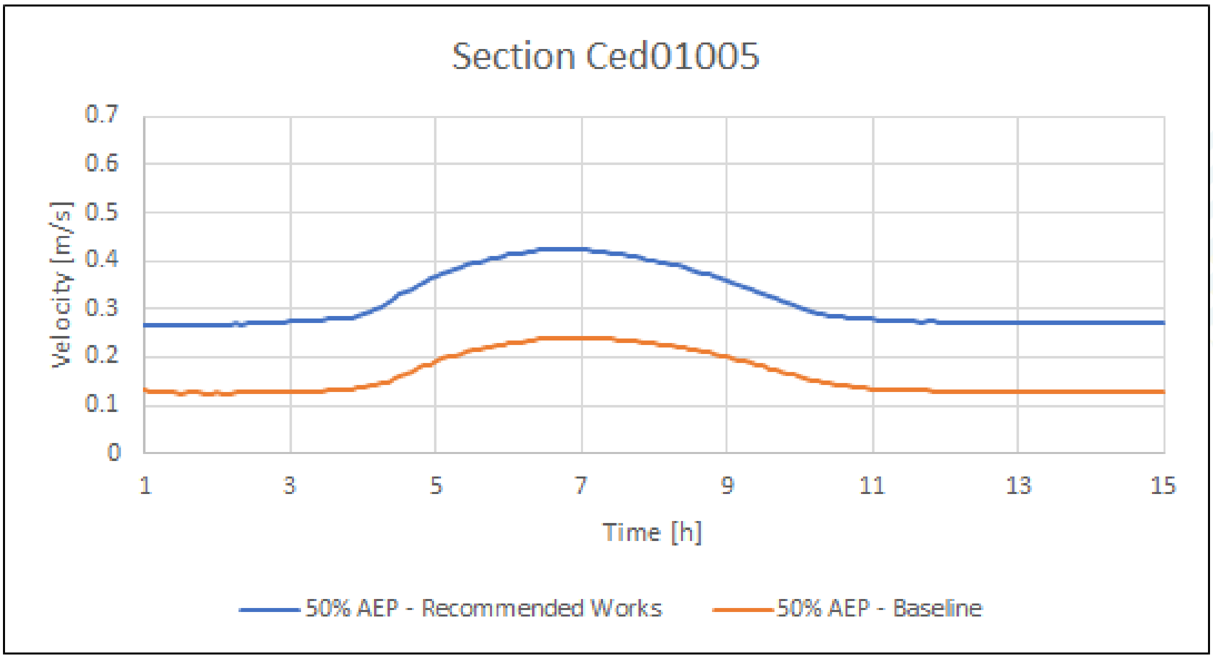


Figure D0-34: 50% AEP Velocities – Section Ced01011 (Upstream of Private Culvert CED01004)

D.9 - Summary

The Cedarwood Stream, distinct from the Shannon River in scale and hydraulic behaviour, was modelled using a separate 1D-2D FM-Tuflow model. The construction of this model was phased after the completion of the Shannon River modelling and required an updated survey in 2022. This model captured the stream's specific hydraulic behaviour and potential impact of backwater flow from the Shannon River. Although excluded from the main Shannon model, the stream's flows were integrated directly into the Shannon model at its outlet to depict the interaction of the two watercourses accurately.

LiDAR data with a 2m resolution and 200mm vertical accuracy was used in the modelling. The 1D model relied on topographic survey data from two different surveys: Murphy Surveys (2012) and McDonald Surveys (2022). The former provided full extent coverage but only 16 channel cross-sections and 6 structures from this survey data were used, specifically for the downstream end of the model, along a distance of 140m. The latter survey offered comprehensive data on the remaining study area, including on both river branches along the railway line and was utilized to model the majority of the stream.

In addition to the 2022 survey, a further survey was conducted by McDonald Surveys in 2024, following maintenance and desilting works carried out upstream of the railway line. The 2024 survey was essential to update the baseline model and accurately reflect the modified channel geometry, particularly where the maintenance works had altered the stream. These updates were necessary to ensure the most current conditions of the Cedarwood Stream were captured.

The catchment is bisected by a railway line which causes a significant flow constriction. Upon examination of the new survey data and site visits, it was revealed that the watercourse had two branches along the railway line, each with culverts crossing the line. The CFRAM model only included the southern culvert, while the FRS model included both culverts. Vegetation growth, notably during the summer, was identified as a significant feature of this watercourse.

The downstream boundary of the model was set at a fixed water level, using the peak water level from the main Shannon model. The model assumed a peak-to-peak phasing with the Shannon, with Manning's values roughness applied to the 1D channel.

The railway constriction delays the hydrograph peak and attenuates the peak flow. Flooding upstream occurs by overtopping both banks and filling the forestry lands. The majority of the flow was predicted by the model to pass through the northern culvert, which is lower than the southern one.

In a 1% AEP event scenario, left bank overtopping was observed downstream of Common’s Road culvert, due to a flow constriction caused by the CED01004 culvert due to its narrow channel entrance, limited diameter, and elevated invert, resulting in a flow path towards the Shannon confluence. The model indicates high sensitivity to water levels in this area, with a 20mm difference determining the formation of a flow path during the 1% AEP event. A blockage test on culvert CED01004 also resulted in a significant rise in water levels, impacting downstream properties.

Two model runs were conducted to evaluate the culvert's impact on upstream water levels: one where the culvert was lowered by 400mm to align with the upstream and downstream bed levels, and another where the culvert was entirely removed and the channel regraded and widened. The former scenario was effective in mitigating flood risks, preventing left bank overtopping and reducing the flood levels. Therefore, to mitigate these risks, an expanded-width box culvert installation is proposed in the reshaped channel section to improve water flow and provide adequate freeboard.

Different scenarios, including regrading, channel maintenance, and a combination of both, were tested to increase the freeboard along the Cedarwood Grove. The most effective solution for flood risk mitigation in this reach was found to be the combination of regrading and channel maintenance, which significantly increased the freeboard.

It should be noted that, due to the desilting and maintenance works undertaken upstream of the railway line in 2023/2024, there has been an increase in flood risk when compared to previous conditions, particularly downstream of the railway line. The desilting and vegetation clearance have improved flow capacity upstream and through the railway line culverts, leading to increased flow rates that raise water levels downstream. While these changes enhance the stream’s capacity to convey water, they may exacerbate the flood risk in certain areas, particularly around existing properties, if further interventions are carried out upstream without proper flood mitigation measures in place.

In the 2024 update of the model, an in-stream sandbag structure was observed to be in place upstream of the railway crossing. This was analysed as part of the sensitivity analysis and presented an increase of 20mm in water levels in the Cedarwood Stream downstream of the railway crossing. However, this is not a permanent structure and as such was not deemed necessary to be included in the baseline condition, as it is unlikely that it would remain in place during a flood event.

The recommended works along Cedarwood Stream include lowering the inlet level of culvert CED01004, increasing the culvert's size and width of the entrance channel, clearing out trees, bushes and silt from specific areas, and conducting regular vegetation and silt removal. The implementation of these works will improve the Cedarwood Stream's drainage, increase velocities, allow for sediment flushing through the slack gradient reach, reduce potential for vegetation growth, and deter fly-tipping into the stream. No debris management screens are recommended as long as an effective maintenance regime is implemented after the scheme.

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