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Cork County Council

Midleton Flood Relief Scheme

Hydraulics Report

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Job number 252803-00

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Abbreviations

AEP	Annual Exceedance Probability
CCC	Cork County Council
CCTV	Closed-circuit television
CFRAM	Catchment Flood Risk Assessment and Management
DTM	Digital terrain model
EPA	Environmental Protection Agency
FFL	Finished Floor Level
FSE	Factorial Standard Error
FSR	Flood Studies Report
LiDAR	Light Detection and Ranging
NRR	Northern Relief Road
OD	Ordinance Datum
OPW	Office of Public Works

1. Introduction

1.1 Context

Arup has been commissioned by Cork County Council (CCC) to develop a Flood Relief Scheme (FRS) for Midleton. The overall scheme will consist of flood alleviation measures that defend against fluvial, tidal, pluvial and groundwater flooding sources of flooding.

There are five stages to the project:

- Stage I – Development of a number of flood defence options and the identification of a preferred Scheme.
- Stage II – Public exhibition.
- Stage III – Detailed design, confirmation and tender.
- Stage IV – Construction.
- Stage V – Handover of works.

This Hydraulics report is produced as part of Stage I of the project and details the hydraulic analysis undertaken for Midleton for the existing scenario. Hydraulic modelling undertaken as part of the optioneering phase of the project is detailed in the Midleton FRS Options report.

1.2 Scope

The purpose of this report is to detail the hydraulic analysis carried out as part of Stage I of the project for the existing (present day) scenario. The scope of this element of work is to:

- Review the hydraulic modelling undertaken as part of the Lee CFRAM Study;
- Develop a dynamic1D/2D hydraulic model of all the relevant watercourses and associated floodplains in Midleton;
- Calibrate and/or validate the hydraulic model against historic flood events (December 2015, April 2018, December 2018 and October 2023);
- Simulate a range of combined fluvial/tidal design flood events for the current scenario for both fluvially dominant and tidally dominant scenarios. The Annual Exceedance Probability (AEP) events to be considered are: 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1%. Ground water flood risk and Pluvial flood risk are also to be assessed as part of the project;
- Produce flood maps for Midleton which integrate fluvial, tidal and pluvial flooding for both the existing and climate change scenarios;
- Calculate flood depths at every property within the study area for a range of return period events for use in the economic damages assessment;

1.3 Study Areas

Figure 1-1 presents an overview of the catchment area upstream of Midleton. The scheme area is presented in Figure 1-2. As can be seen from Figure 1-2, the scheme area includes all of Midleton, Ballinacurra and the area in the vicinity of the Water Rock (also known as the Oatencake) Stream.

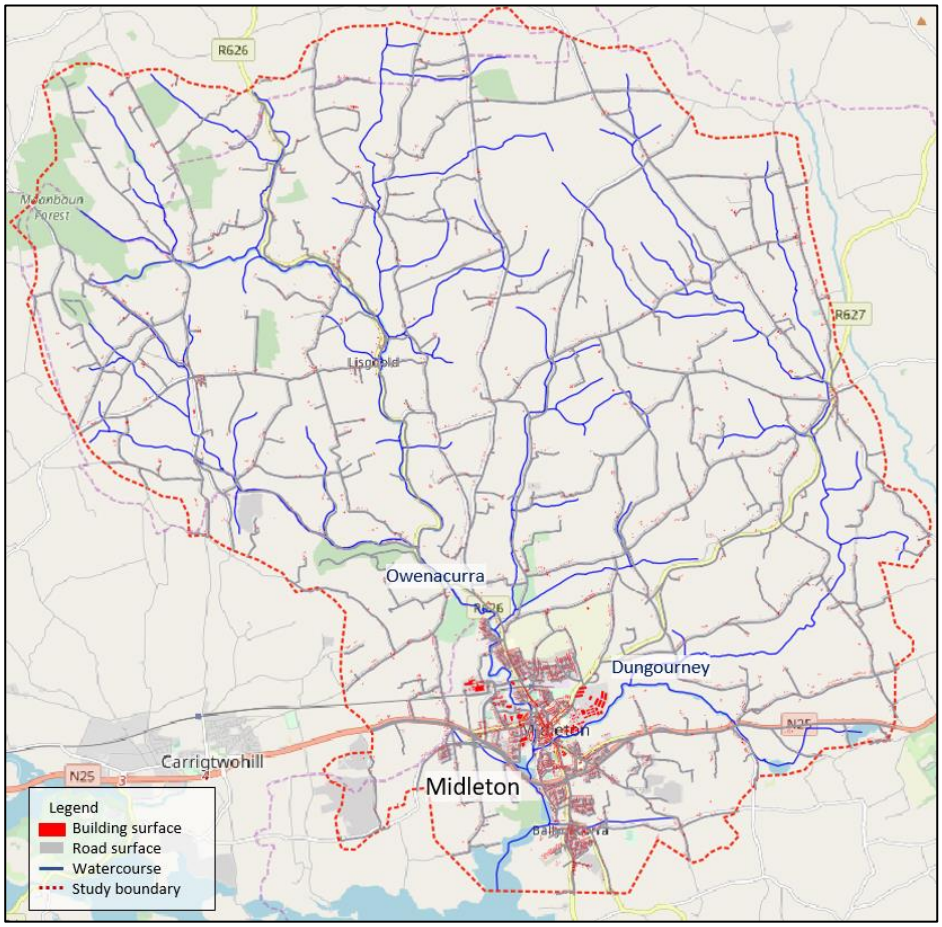


Figure 1-1 Study Area (© Open Street Map)

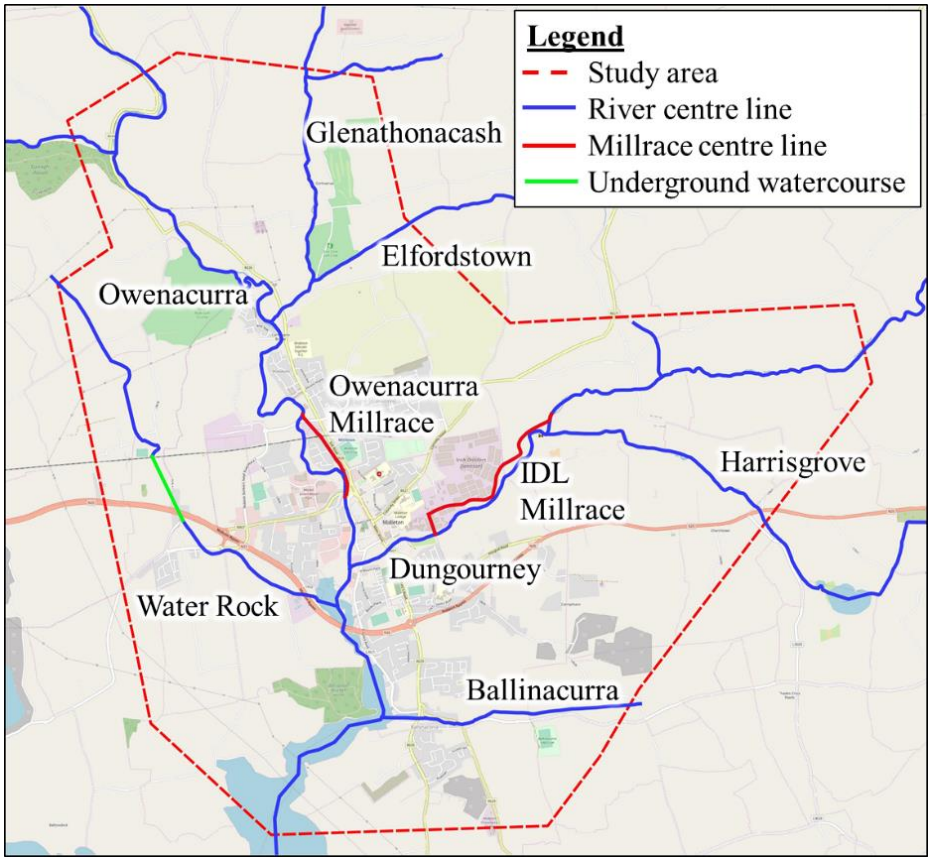


Figure 1-2 Scheme Area (© Open Street Map)

The watercourses considered as part of the study are listed in Table 1-1. The coordinates of the upstream and downstream extent of each watercourse within the study area is also provided in this table. Figure 1-2 indicates the centre lines of the watercourses.

Table 1-1 Primary Watercourses

Watercourse	Upstream extent (ING)	Downstream extent (ING)	EPA River ID
Owenacurra	186435, 76118	187974, 71749	19O03
Dungourney	192474, 74986	187967, 73112	19D07
Glenathonacash	187611, 77260	187278, 75287	19G66
Elfordstown	188204, 75975	187640, 75652	19E02
Harrisgrove	191699, 73472	189699, 74291	19H02
Ballinacurra	190523, 71922	188540, 71794	19W17
Water Rock (Also knowns as Oatencake)	185584, 75434	187602, 72879	19O08

1.4 Standard of Protection of the Scheme

The required Standard of Protection (SoP) of the Scheme as stated in the Project Brief is to "prevent flooding during flood events with a 1% (for fluvial floods) and 0.5% (for tidal / coastal floods) annual exceedance probability (AEP)". While the brief does not explicitly refer to freeboard requirements, Arup has confirmed with CCC/OPW that the target SoP is to include an appropriate allowance for freeboard which will be determined as part of the Optioneering.

The SoP against both pluvial flooding and groundwater flooding is not explicitly specified in the brief. For pluvial flooding, Arup has therefore adopted the same SoP as for fluvial flooding i.e. the 1% AEP SoP plus freeboard. It is difficult to specify a SoP for groundwater flooding due to a lack of historic data on groundwater events in the town as well as the inherent uncertainty over the behaviour of groundwater in different flood events. Given that the groundwater component of the December 2015 flood event was extreme, this event has been adopted as a proxy for the groundwater design event i.e. the groundwater SoP of the scheme includes all events up to and equivalent in magnitude to the December 2015 groundwater flood event.

The project brief states that alternative Standards of Protection should be considered as part of the project where they would "provide greater benefits relative to cost, a more socially acceptable scheme or for other pertinent reasons". Alternative standards of protection are considered as part of the Optioneering for the scheme and are detailed in the Options report.

1.5 Overview of the report

Section 1 presents an overview of the Midleton Flood Relief Scheme Project and outlines the objectives of the hydraulic modelling element of the study.

Section 2 describes the various datasets collected as part of the study. The development of the various elements of the model is described in Section 3 of the report while Section 4 presents the calibration of the model.

Section 5 of the report presents the finding of the Sensitivity Analysis runs.

Section 6 presents the October 2023 Storm Babet flood event model calibration for the Owenacurra and Dungourney reaches.

Section 7 outlines how both the hydrological estimation and hydrogeological assessment have been incorporated into the hydraulic modelling. For further information on the hydrological estimation and hydrogeological assessment conducted please refer to the Midleton Flood Relief Scheme Hydrology Report[1]

An overview of the results design runs is presented in Section 8. This section needs to be read in parallel with inspecting the various flood maps and hydraulic modelling results which are presented in the Appendices.

Section 0 presents the projected increase in flood risk associated with climate change and discusses the mechanisms of flooding in each area associated with climate change scenarios.

The methodology used to derive both the pluvial and groundwater flood maps are presented in Section 10.

The overall conclusions of the hydraulic modelling are presented in Section 11.

2. Data Collection

This chapter details the various datasets used in the development and running of the Midleton 1D/2D hydraulic model.

2.1 Mapping

A suite of maps of varying resolutions (1:1000, 1:5000 and 1:50,000) have been used in the construction of the hydraulic models and in the presentation of model results. These maps have been provided under licence from Ordnance Survey Ireland (OSi).

The OSi NTF dataset has also been used to define the outline of existing buildings in the 2D Tuflow grid and also for correctly identifying different surface types in the floodplain.

2.2 River Survey Data

The 1D elements of the hydraulic models have used channel and structure cross sectional survey data from a number of different surveys as follows:

- Lee CFRAM Study survey data (2007);
- Midleton FRS Infill and validation survey (2017/18);
- Detailed culvert survey (2018).

The data from the three surveys is sufficient to develop an accurate 1D model without the need to utilise interpolated cross sections. Each of the three surveys are described in detail below.

2.2.1 Lee CFRAMS survey data

A detailed channel and structure survey of the Lee Catchment was undertaken by Maltby Land Surveys Ltd as part of the Lee CFRAMS between February and June 2007. Approximately 250km of river channel were surveyed which included the Owenacurra river and Dungourney stream. As both of these watercourses were classified as Urban Area Watercourses (UAW's) under the Lee CFRAM Study, cross sections were surveyed at approximately 100m intervals along the channel and at all structures that were deemed to be of hydraulic significance. The cross sections extended for approximately 20m into the floodplain on either side of the channel.

[1] Midleton Flood Relief Scheme Hydrology Report (2025)- https://www.floodinfo.ie/frs/en/midleton/project-info/project-documents/

2.2.2 Midleton FRS Infill and validation survey

As part of the review and quality assurance of the Midleton Lee CFRAMS hydraulic model, Arup identified a number of areas where additional cross section survey data would improve the performance and accuracy of the Midleton FRS model. All of these areas were subsequently surveyed as part of OPW's Infill and Validation Survey Management Contract. In addition, watercourses in the study area that were omitted from the Lee CFRAMS and hence had no available data, were also surveyed as part of the infill survey.

Murphy Surveys undertook the infill and validation surveys in May 2017 and January 2018.

Spot levels along the banks of the Owenacurra and Dungourney rivers were also collected as part of the survey in order to accurately define the bank levels along the key lengths of the reach. This data allows the level at which water spills from the 1D to the 2D elements to be correctly represented in the model.

Figure 2-1 presents the 1D cross sectional data used in the model for the centre of Midleton. The cross sections in red are from the Infill and Validation Survey while the cross sections in blue are from the survey undertaken as part of the Lee CFRAM Study.

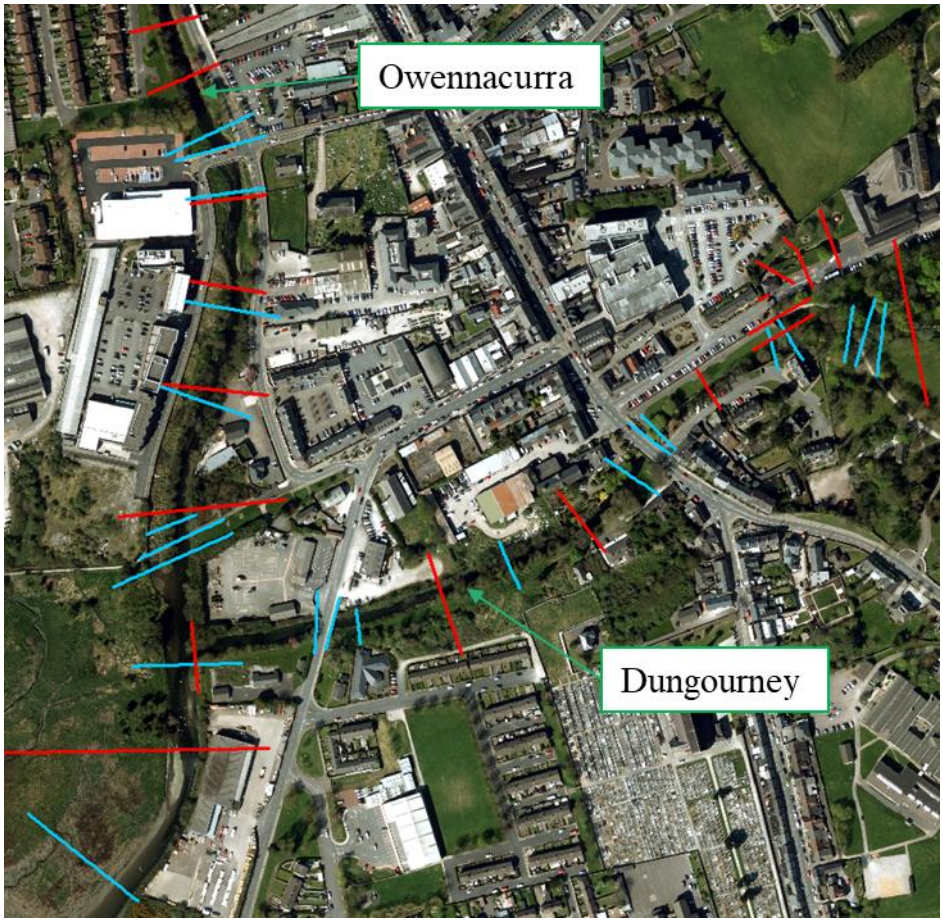


Figure 2-1 Midleton FRS Hydraulic model cross sections

2.2.3 Detailed culvert survey

A detailed CCTV and geometric survey of the structures and offtakes along two millraces in Midleton was undertaken by Amelio surveys in March 2018. The data allows for the following water courses to be included in the 1D/2D hydraulic model:

- Owenacurra Millrace;
- IDL Millrace (offtake channel from the Dungourney);

The alignment of the millraces is presented in Figure 1-2 while Figure 2-2 presents a close-up view of the Owenacurra Millrace alignment. Appendix C provides detailed data sheets for all the structures along the millraces.

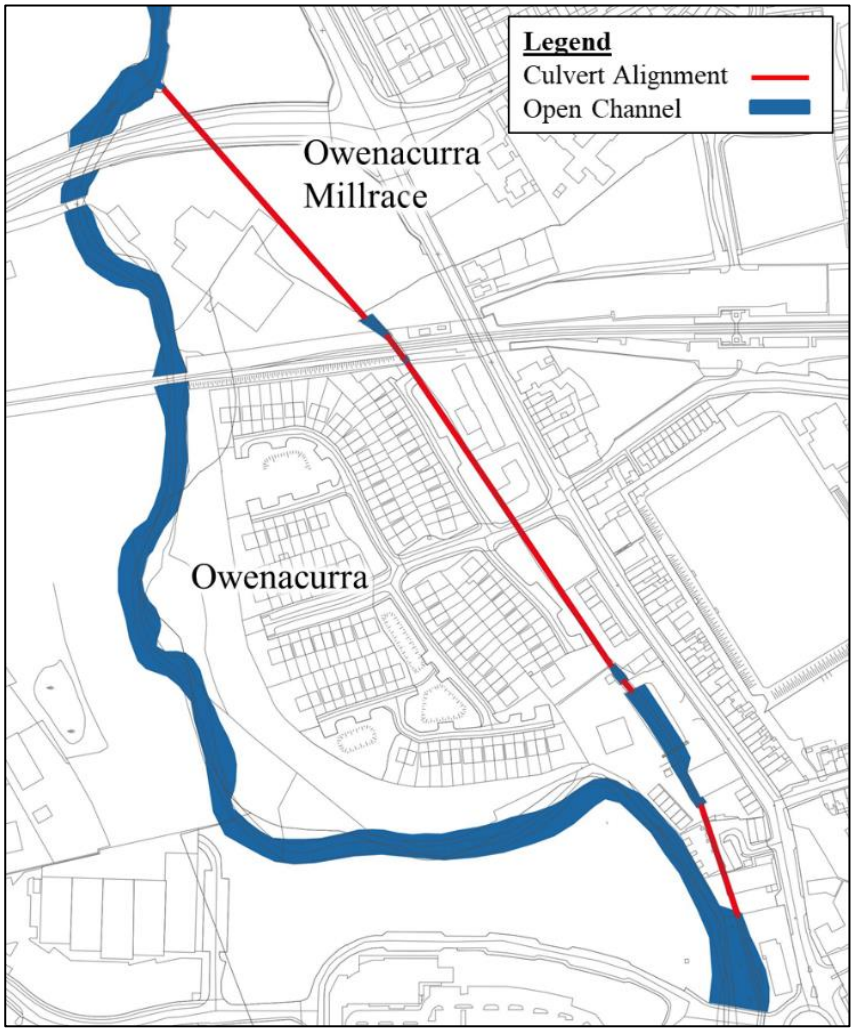


Figure 2-2 Owenacurra Millrace alignment

2.3 Digital Terrain Model

The Digital Terrain Model (DTM) is a bare earth representation of the floodplain topography in which all the buildings and vegetation have been removed. It is used in the model to define the ground elevations of the 2D model grid and represents a critical aspect of the model.

The DTM used in the study was surveyed by BlueSky International in April 2017. The specification of the dataset is provided as:

- ING65 co-ordinates
- OSGM15 Geoid model

Figure 2-3 presents a snapshot of the Midleton Lidar dataset superimposed over aerial imagery.

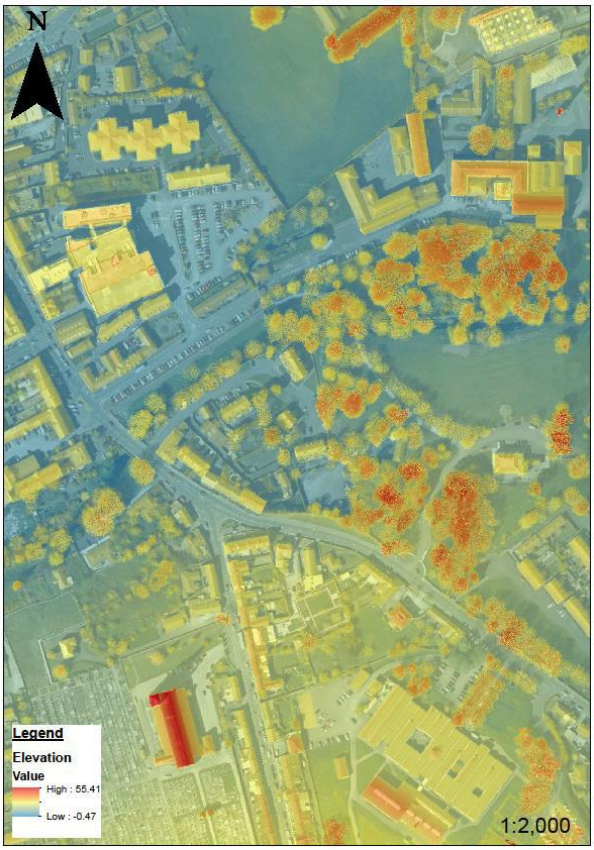


Figure 2-3 Sample Lidar data from the Midleton dataset

2.4 2015 Flood event data collection

Midleton was badly flooded in December 2015. A significant amount of anecdotal data on the event was collected both during the event and after the event by various emergency response personal, Cork County Council Staff, as well by members of the public. Additionally, Arup staff collected post event data as part of a previous commission to undertake a detailed analysis of the event[2].

Various data is therefore available from the event, including;

- Anecdotal information on the mechanisms of flooding;
- extent of inundated areas;
- peak water levels and the time of peak water at a number of locations;
- Number of inundated properties;

All the data collected on the event was used to calibrate the 1D/2D hydraulic model as presented in Section 4 of this report.

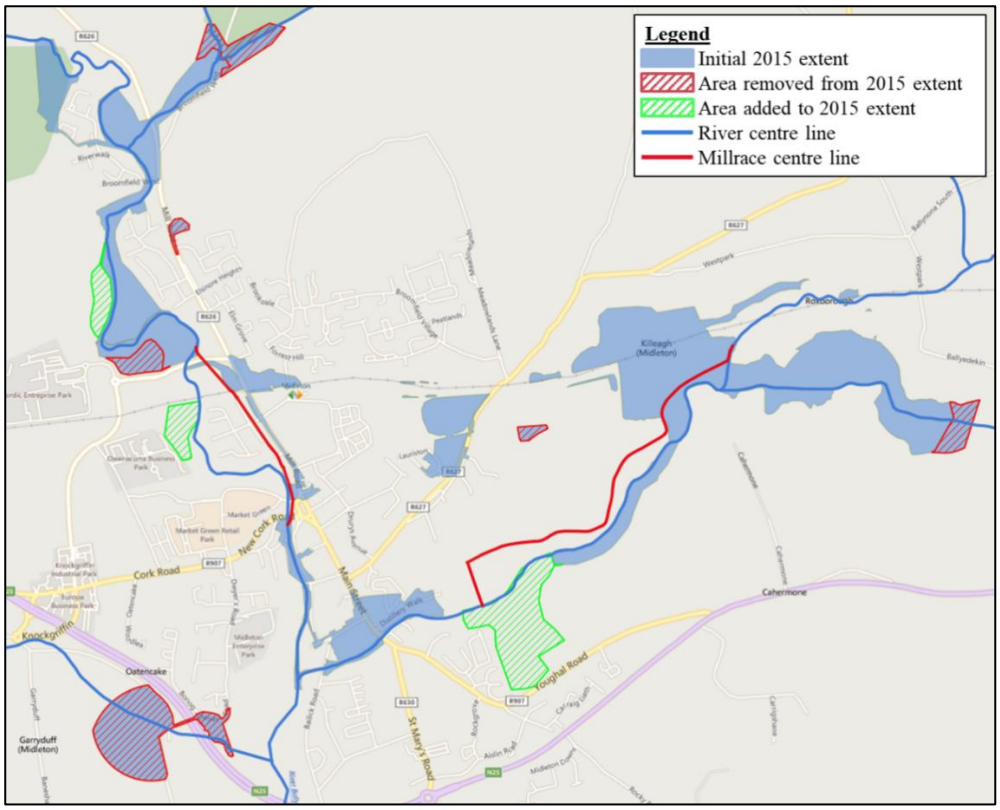
2.4.1 2015 Estimated flood extent

Arup was provided with an estimate of the maximum extent of the December 2015 event by Cork County Council. The extent was based on various datasets, including;

- Observed flood extents and mechanisms of flooding;
- Post flood-event surveys;
- Drone footage of the event (after the peak had passed);

[2]Arup, Midleton Flooding Dec 2015/Jan 2016, Flood Risk Review Report. June 2016.

When compared against the 2017 Lidar dataset, it is evident that there are a number of areas within the estimated flood extent that could not have been inundated due to the existing topography.
There were also a number of areas immediately adjacent to the Owenacurra River that are known to have been inundated during the event but were not recorded as having been flooded. We have therefore made a number of minor edits to the estimated extent in order to correct these anomalies. The results of the modifications are presented in Figure 2-4.


Legend
Initial 2015 extent
Area removed from 2015 extent
Area added to 2015 extent
River centre line
Millrace centre line
Cork Road
New Cork Rd
Main Street
Toughal Road
St Mary's Road
Caherlag Lane
Caherlag
Caherlag
Caherlag
Oatencake
Garryduff (Midleton)
Killeagh (Midleton)
Ballyvodock
Ballyvodock
Ballyvodock

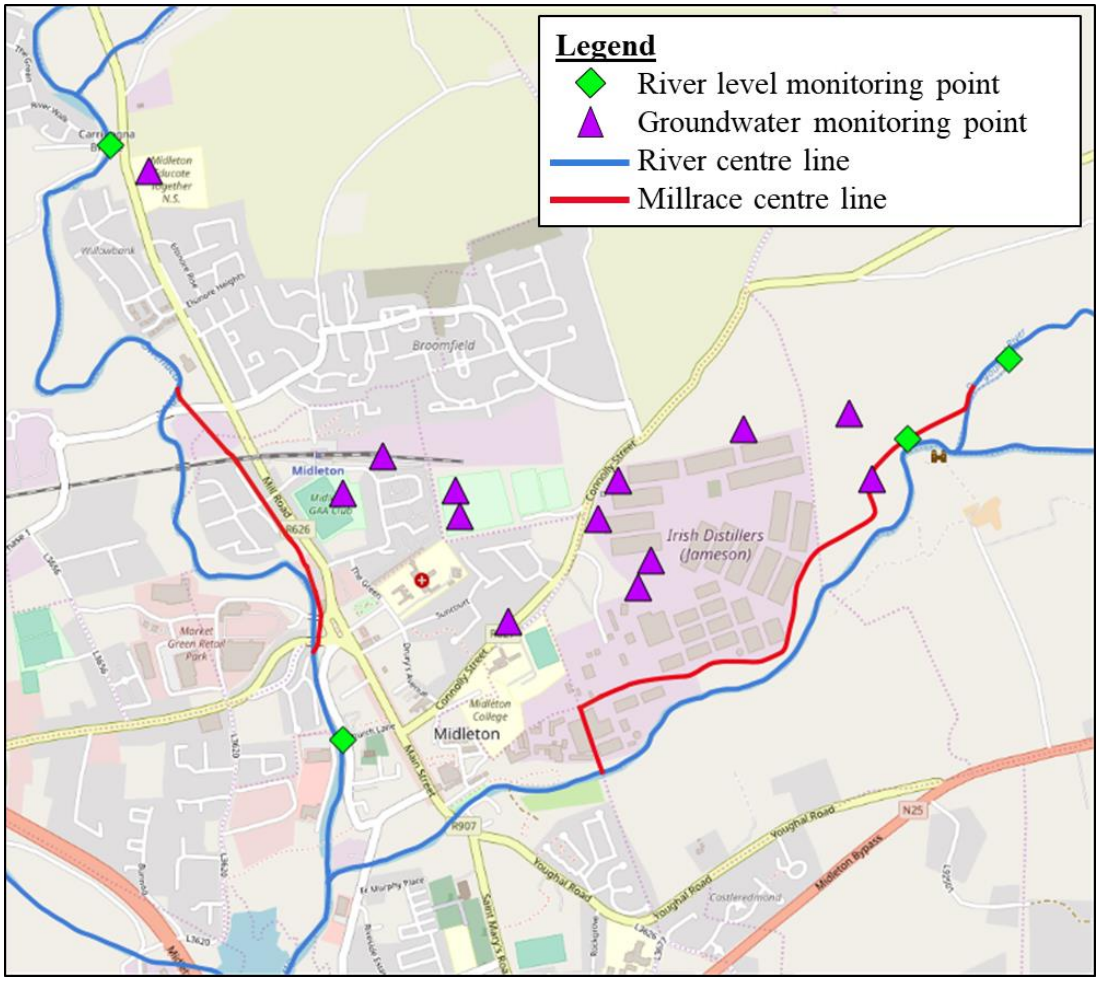


Figure 2-5 Locations of groundwater and river monitoring points (© Open Street Map)

The river monitoring data is presented in Figure 2-6. It can be seen that the data collection commenced in January 2018 and ended in January 2019. These locations were selected as they were accessible, suitable to understanding the groundwater and surface water interaction and also a suitable location for hydraulic 1D2D model calibration. There is however a gap in the Moore's Bridge data due to the unauthorised removal of the gauge from the Owenacurra by a third party. Sufficient data has however been collected at this location to allow for the model to be calibrated. Additional collection of data at this location is therefore not deemed necessary.

There is also a minor gap in the data from the gauge at Lidl. This water level data has been used to calibrate the Midleton Hydraulic Model as detailed in Section 4 of the report.

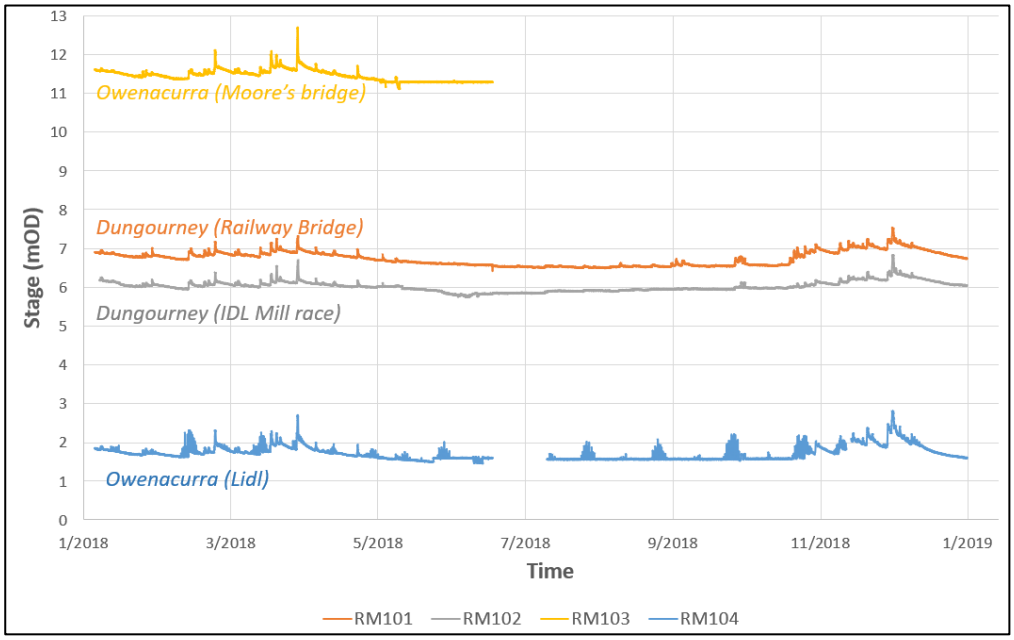


Figure 2-6 River monitoring data

2.6 2023 Flood event data collection

As part of the Midleton FRS project and as per Section 2.13 of the project brief, Arup prepared a post-flood event review report that considered the impact of Storm Babet (18th of October 2023). The scope of this report was as follows;

- Site walkovers to affected areas in the immediate aftermath of the event in order to gather data and anecdotal information on the impact of the flooding.
- Gathering of photographs and video footage of the event from various social media streams;
- Extensive liaison with CCC in order to ensure that all the data and information gathered by CCC has been assessed and considered in the report. This data includes aerial imagery of the event taken by the Guileen Coast Guard;
- Review of the event hydrology and recorded rainfall data;
- Review of the mechanisms of flooding that were observed during the event and comparison with the findings of the Midleton FRS Hydraulics model flood extent maps;
- Establish the likely maximum extent and depth associated with the flood event;
- Establish the likely number of residential and commercial properties flooded during the event;

Storm Babet was the most significant fluvial flood event in recent memory. There was flooding of residential and commercial properties in Tir Cluain, Moore's Bridge, Willowbank, Millbrook, the Northern Relief Road, Millrace Apartments/Market Square Area, Midleton Train Station car park, the Railway Cottages, Cork Road Bridge, Woodlands, Thomas Street, Riverside way, Kennedy Park, Main Street, Bailick Road, Dickinsons Lane, Connolly Street, Beechwood/Oakwood Estate, Midleton GAA Grounds, People's Park, Baby's Walk, Roxboro Close, Distillery Walk, The Green Way, Midleton Rugby Club, Water Rock, Riversfield estate. Flooding of agricultural lands east of IDL was also experienced.

Please refer to the Midleton FRS Storm Babet Report[4] for more information including estimated flood extents and mechanisms of flooding observed.

[4] Midleton Flood Relief Scheme, Storm Babet Flood Event Report Part 1 & Part 2 (https://www.floodinfo.ie/frs/en/midleton/project-info/project-documents/)

3. Model Development

3.1 Introduction

A one-dimensional (1D) and two-dimensional (2D) model of the primary watercourses and their associated floodplain in Midleton has been constructed to simulate flood events within the scheme area. The 1D model simulates the in-bank flows and has been constructed in Flood modeller Pro 1D (Version 4.4) software. The 2D model simulates the out of bank floodplain flows and it has been developed in Tuflow software (Version 2017-09-AC-iSP-w64). Both the 1D and 2D models are dynamically linked and run together as a coupled model.

3.2 Model Development

A 1D hydraulic model of the Owenacurra and Dungourney River was developed as part of the Lee CFRAM Study. The reader is referred to the Lee CFRAMS Hydraulics Report for a detailed description of this model.

The Midleton FRS hydraulic model was developed using the Lee CFRAMS hydraulic model as a starting point but represents a significantly more accurate and detailed version of the model. The development of the Midleton FRS model using the Lee CFRAMS model involved a number of steps which can be summarised as follows:

- **Replacement of the overland flow domain in Midleton** – The floodplain of the Lee CFRAMS model was represented by reservoir units and parallel channels all of which have been replaced by a 2D grid in the Tuflow model.
- **Additional watercourses and millraces** –A number of watercourses and millraces within the study area that were not included in the Lee CFRAMS model have been included in the Midleton FRS model as listed in the following section.
- **Infill and Validation Survey** – As detailed in Section 2.2, Arup identified a number of areas where additional river survey data was required in order to improve the performance and accuracy of the Midleton FRS model over the Lee CFRAMS model. Additionally, Arup also identified a number of areas where modifications to the river channels (and floodplain) have occurred since the Lee CFRAM Study survey was undertaken. All of these areas were subsequently surveyed as part of the Infill and Validation Survey Management and incorporated into the model.
- **Model Parameters** – A number of the model parameters used in the Lee CFRAMS model were altered in the Midleton FRS model. These include channel roughness and structure coefficients which are described in Section 3.4 of this report.

3.3 Model Extents

3.3.1 Midleton FRS Model

A schematic of the Midleton FRS model for the Owenacurra and Dungourney rivers upstream of the N25 road is presented in Figure 3-1. The 2D model domain is represented by the green shading in the figure and it can be seen that it covers the main urban area of Midleton, Ballinacurra and Water Rock. The key out-of-bank flooding mechanisms within Midleton are therefore modelled in two dimensions which offers a significant improvement over the Lee CFRAMS model. The red nodes in the figure present the location of the 1D model cross sections. It is noted that the downstream boundary of the model is located circa 1.2km downstream of the N25 within the Owenacurra estuary.

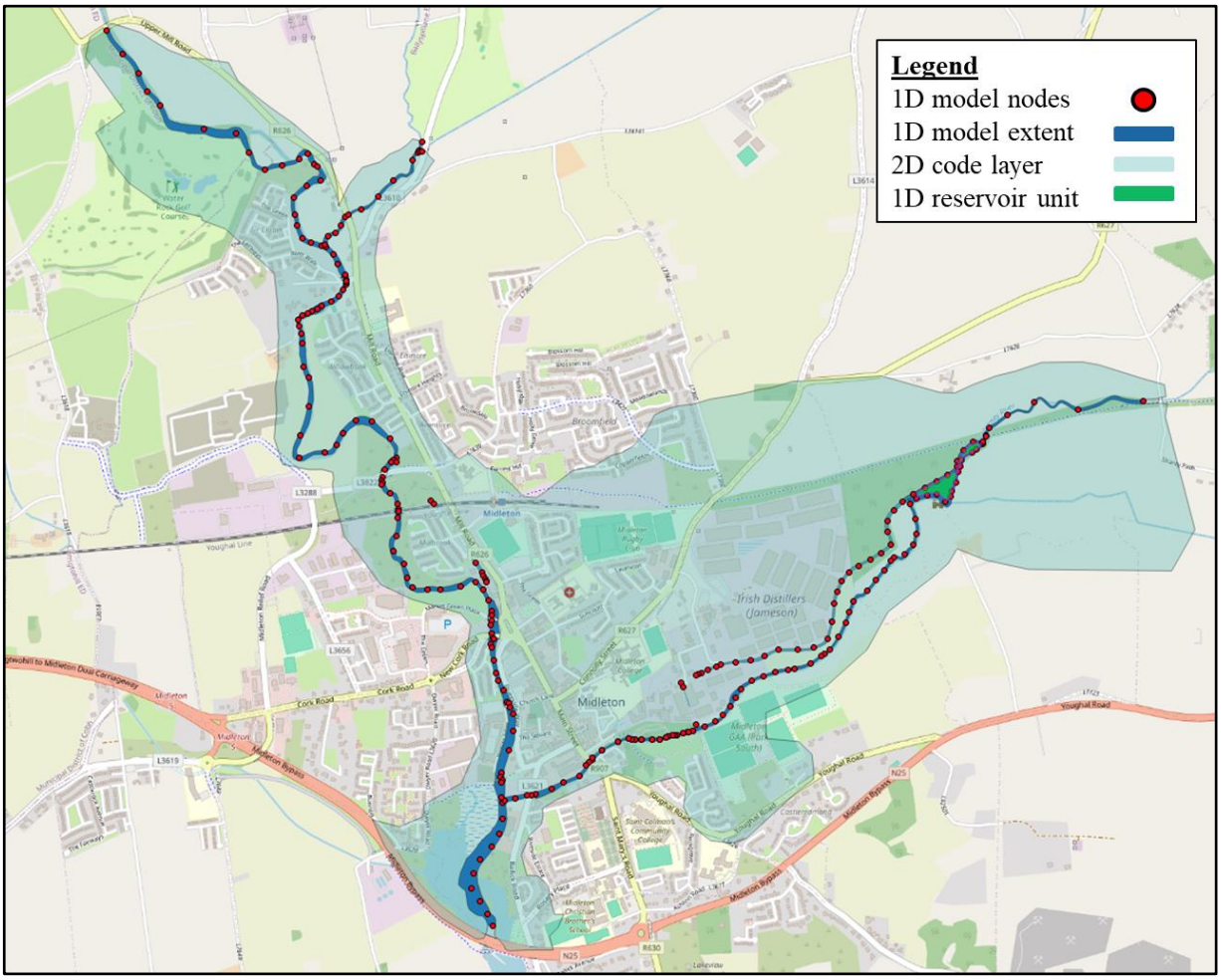


Figure 3-1 Schematic of the Midleton FRS model (© Open Street Map)

Separate hydraulic models of the Ballinacurra and Water Rock watercourses have also been development as part of the study. The Water Rock Model and Ballinacurra have also been updated to a 1D/2D model. The Ballinacurra model has been updated by using a new raster survey covering the missing area at the flow inlet of the model. Water Rock stream has been connected to the Owenacurra model while Ballinacurra remains disconnected to ensure the stability of the model. Schematics of these models are presented in the following figures. A full detailed schematic of each of the hydraulic models is included in the appendices.



Figure 3-2 Ballinacurra Hydraulic model (© Open Street Map)

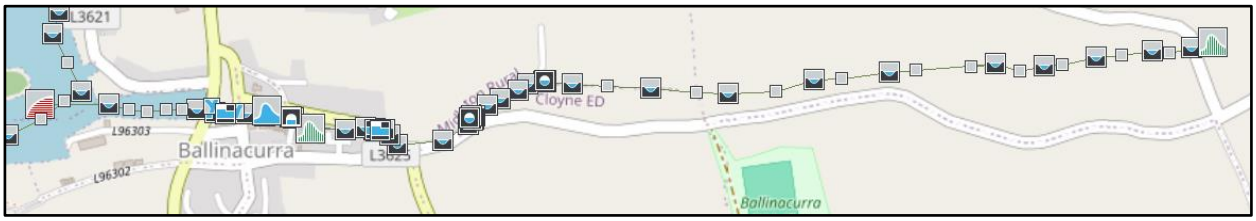


Figure 3-3 Ballinacurra Hydraulic model 1D schematic (© Open Street Map)

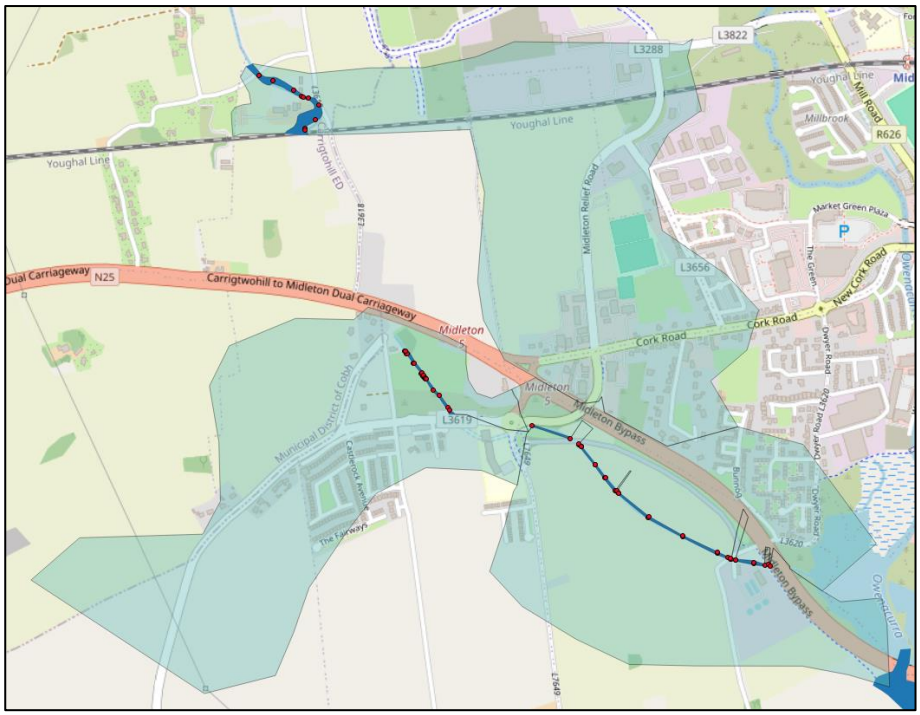


Figure 3-4 Water Rock Hydraulic model (© Open Street Map)

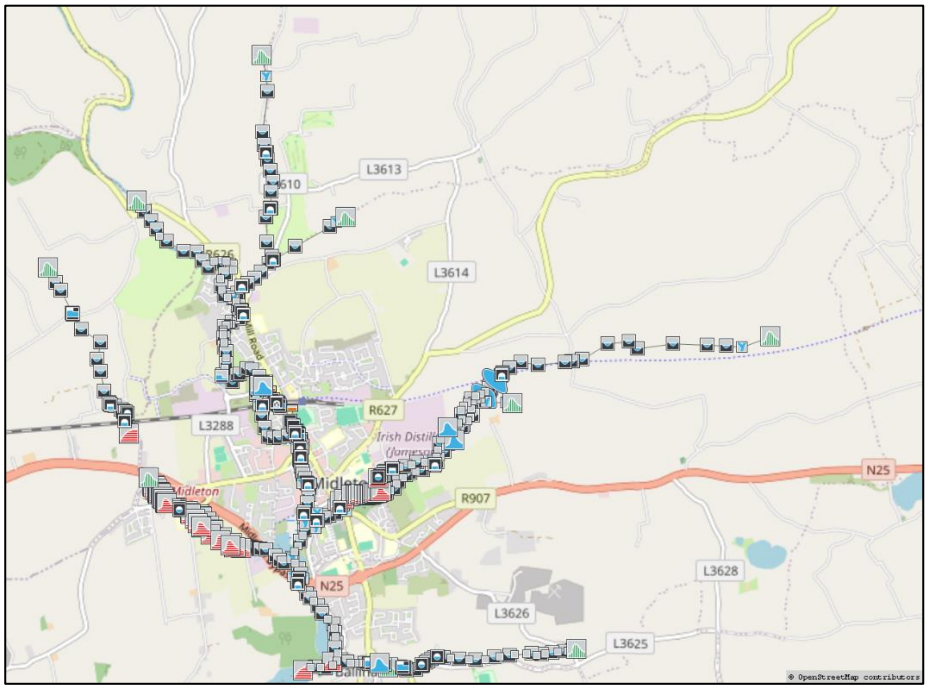


Figure 3-5 1D elements of all the hydraulic models (© Open Street Map)

3.4 Model Parameters

3.4.1 Labelling System

The model nodes derived from the infill survey followed the same labelling format as used for the Lee CFRAMS models (e.g., The Owenacurra River labels are provided in the form of 3OWE_00000, with chainage starting from 0 at Owenacurra Estuary). The original infill survey cross section name has been included in the Flood Modeler Pro Comments to allow for ease of cross checking against the CFRAMS model. This approach also ensures that a direct reference can be made to both the CFRAMS and Infill surveys.

For reaches that were not included as part of the Lee CFRAM Study (e.g., Water Rock and Ballinacurra), the naming convention provided by the surveyor was adopted and informed the model node names. For the majority of cross sections these labels include an arbitrary reach number followed by a chainage (e.g., The Ballinacurra Stream labels are provided in the form of R7_00000, with chainage starting from 0 at Owenacurra Estuary/ Ballinacurra Inlet confluence).

3.4.2 Model Resolution

The 1D model resolution is determined by the distance between adjacent cross sections which changes throughout the model domain. For the key urban area, this distance never exceeds 50m and is frequently much less than this. This is of sufficient resolution to appropriately model the one-dimensional flow in the channel.

The 2D model resolution is defined by the spacing of the 2D grid. Defining this parameter involves a trade-off between accurately resolving the two-dimensional flow in an urban environment using a high-resolution grid and the computational run time of the model which is reduced with the lower resolution grids.

A 2m grid resolution has been selected for the Midleton FRS 2D model domain. This is a very high resolution which accurately resolves flow in the urban areas as it allows for the division and splitting of flow to be captured in the model. The relatively short duration of the design runs combined with the relatively small domain ensure than the computational burden of the model runs is manageable.

3.4.3 Manning's n for the 1D and 2D Models

The roughness values of the 1D model have been defined for three separate components of each cross section: (1) The left bank, (2) The main channel, and (3) The right bank. These components of each cross section in the model are defined through the use of panel markers in FMP.

Some cross sections located in the 2D domain of the model have no left or right bank as they link to the 2D model domain at the point where the left/right bank begins.

The Manning's n roughness values in the 1D model were selected based on a detailed analysis of a number of datasets as follows:

- Model calibration (October 2023, December 2018, April 2018 and December 2015 event).
- The values previously used in the Lee CFRAM Study.
- Survey photographs.
- Site visits undertaken by Arup

Typical values used in the study are presented the table below. A detailed description of the Manning values used is provided in Appendix C of the report.

Table 3-1 Manning's n for rivers

Channel Characteristics	Manning's n
Main Channel	
Clean, straight	0.03
Stones, weeds and meandering	0.045
Banks	
Heavy weeds and vegetation	0.045
Trees and thick vegetation	0.08

Manning's n floodplain values were selected based on an analysis of various datasets and the model calibration. The datasets used were:

- Land use derived from OSi NTF mapping;
- Site visits undertaken by Arup; and
- The calibration of the model against the 2015 flood event and validation against the 2023 flood event.

Typical values used in the study are presented in the table below. The values have been selected based on standard values in relevant industry literature and our extensive experience in undertaking hydraulic modelling.

Table 3-2 Manning's n for floodplain

Land use	Manning's n
Roads	0.02
Open parkland	0.03
Forestry	0.06
Buildings	0.1

A sensitivity analysis on the Manning's number was undertaken as part of the study and is detailed in Section 5 of the report.

3.4.4 Representation of the River Structures

All of the bridges in the model have been modelled using the Bridge ARCH unit as this is the most suitable bridge model within Flood Modeller Pro for modelling the bridges along the Owenacurra/Dungourney River due to their average size. Overtopping of the bridges has been accounted for in the 2D domain.

In-line weirs have been modelled using spill units while culverts have been modelled through use of the conduit units. The reader is referred to Appendix C which presents a datasheet for all the key structures included in the Midleton FRS model.

It is noted that the dimensions of all the hydraulic structures have been taken from the surveyed data.

A sensitivity analysis on the bridge head loss units as well as specification of the bridge hydraulic units was undertaken as part of the study and is detailed in Section 5 of the report.

3.4.5 Representation of Buildings and other Structures in the 2D grid

The buildings in the 2D domain were accounted for by (a) applying a high Manning's n value (0.1) to the grid cells which form part of the building footprint, and (b) setting the floor level equal to the surveyed FFL of the building. Where surveyed FFLs were not available the floor level was set equal to the averaged lidar data across the footprint of the building plus an assumed threshold allowance of 150mm.

Representing the buildings in this manner allows for the storage volume within the buildings to be accounted for. It also allows for flow paths through the buildings to be simulated. The high Manning's value ensures that the reduction in flow and velocity caused by the fabric of the building is appropriately represented.

The approach is deemed accurate and appropriate given that a significant number of buildings were inundated during the 2015 and Storm Babet (2023) events. It is noted that the alternative of accounting for the buildings by blocking out their plan areas from the model domain would reduce storage areas in the model and also lead to incorrect flow paths being simulated by the model.

Other structures such as walls and embankments can influence the movement of water in the floodplain. Where appropriate these structures have been correctly represented in the model through the use of Z lines shapefiles. For the December 2015 calibration model, all of the walls and embankments as indicated Figure 3-6 have been included in the model as there is no record of any of the structures having collapsed during the event.

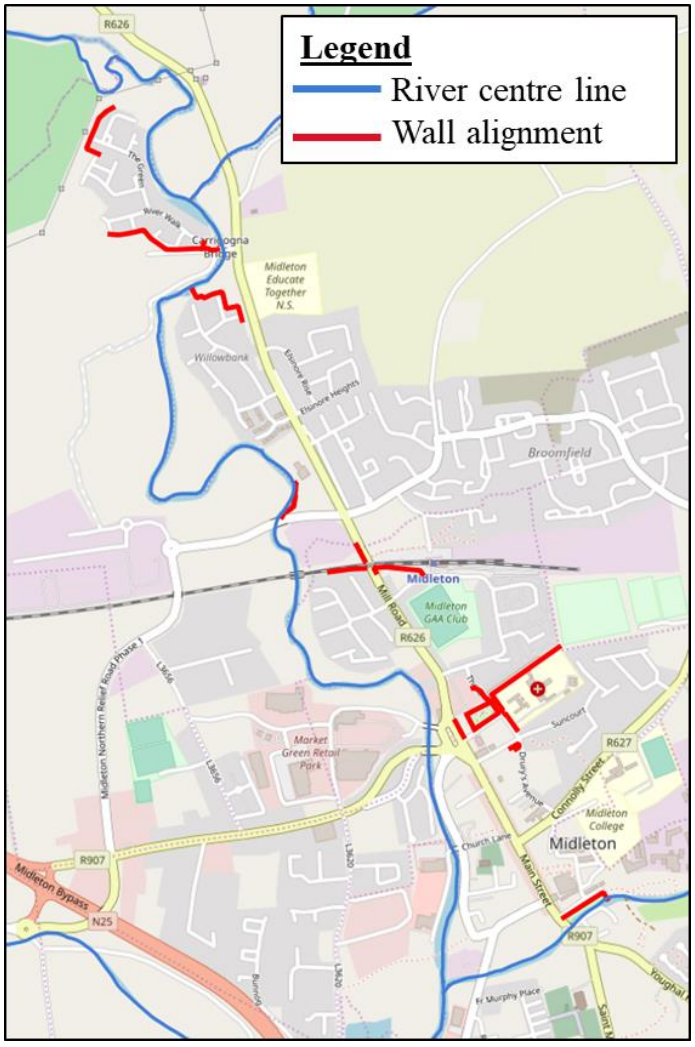


Figure 3-6 Walls and embankment Z lines 2015 Calibration Event (© Open Street Map)

For the October 2023 calibration event (discussed in Section 6 of this report) these defences were generally removed from the hydraulic model as a number of walls collapsed during the event i.e. rear of First Steps Creche, Tir Cluain (Figure 3-7) and boundary of the Willowbank estate (Figure 3-8)



Figure 3-7 Wall failure at rear of First Steps Creche, 2023 Storm Babet Validation Event



Figure 3-8 Wall failure at boundary of the Willowbank Estate, 2023 Storm Babet Validation Event

For the design runs up to and including the Q25 event, it has been assumed that existing walls and embankments will not fail. For the Current Q50, Q100, Q200 and Q1000 events however the informal walls and embankments have been excluded from the model as their structural integrity cannot be guaranteed during these higher order events, given that none of them were designed as formal flood defences structures. This assumption has been validated during Storm Babet given that a number of walls collapsed during the event. It is noted that the walls which failed have been reconstructed so remain in the existing scenario hydraulic model.

For the MRFS and HEFS runs, the threshold is set at the Q25 event, as both boundary peak flows are greater than the boundary peak of the Current Q50 event.

The existing scenario design runs are discussed further in Section 8.

3.4.6 Representation of the cave system in the Water Rock Hydraulic model

The cave system on the Water Rock stream has not been explicitly represented in the model due to significant uncertainty on the internal geometry of the system and the unknown influence of groundwater levels in drowning out the available storage within the cave system, which in turn impacts on water levels in the Water Rock stream upstream of the cave system. Other sources of uncertainty include:

- The irregular and unknown geometry of the entrance to the cave (shown in Figure 3-9);
- The unknown conveyance capacity of the sink hole at the entrance to the Cave which allows water to ingress into the cave.



Figure 3-9 Primary entrance to the Cave System

We have accounted for the cave system entrance by including it as a culvert unit with an associated entry head loss unit in the model. The parameters of the culvert and head loss units were adjusted until a good match was achieved between the maximum observed water level at Water Rock House and the results of our model for the 2015 calibration run. The reader is referred to Section 4 for details on the calibration.

3.5 1D and 2D Model Linkage

There are two main parameters which control the volume of water that spills onto the floodplain (the 2D model domain) from the river channel (the 1D model domain):

- The water level in the river channel;
- The elevation of the bank of the channel, i.e. the elevation at which water spills from the river to the floodplain.

The water level in the river channel is calculated by the 1D model. The elevation of the bank however is defined in the model by the user using the topographic survey data. It is a very important dataset in the model as it controls the volume of water that spills into the 2D domain of the model. Its correct specification is essential in ensuring an accurate and realistic hydraulic model.

The elevation of the left and right banks throughout the 2D model domain of the model were defined from actual surveyed elevations from the river channel survey and were accounted for in the model through the use of Z lines in Tuflow. These Z lines were defined for the entire 1D-2D reach of the model and ensured an accurate representation of the volume of water spilling from the 1D to the 2D domain.

4. Model Calibration

4.1 Introduction

4.1.1 Overview

The Midleton hydraulic model was initially calibrated and validated against three separate flood events:

- December 2018 event – model calibrated against measured water level timeseries on the Owenacurra river. This was an in-bank calibration only;
- April 2018 flood event – model validated against water level timeseries on the Owenacurra river. This was an in-bank validation only;
- December 2015 flood event – model calibrated against flood event data collected by various authorities during the December 2015 flood event, this was an out of bank calibration;

We note that the model was first calibrated against the December 2018 flood event and then validated against the April 2018 event. The model was then used to simulate the December 2015 event.

Each of these model calibration/validations are described in this section of the report.

Following Storm Babet (October 2023) the model was used to calibrate the estimated flow from the additional hydrological analysis carried out. This is detailed further in Section 6 of this report.

4.1.2 Model tolerance

The latest OPW guidance (at the time of writing) was applied with regards to the accepted model tolerance when determining the accuracy of calibrated model.

This guidance states that the model calibration “shall aim to achieve vertical accuracies of +/-100mm, and no greater than +/-200mm when compared to recorded flood event point data”. Furthermore, it is noted that the Scottish Environment Protection Agency (SEPA) guidance document on hydraulic modelling states that “high confidence” in the hydraulic modelling of “local scale or detailed studies” is achieved when the “tolerances for peak water level are in the order of +/-150 mm”. These quantitative tolerance of model performance have therefore been considered as part of the Midleton FRS hydraulic modelling calibration and are referenced further in the following sections.

4.1.3 Blockage assumptions in calibration model

It was assumed initially that all bridge and culvert units were unblocked as part of the calibration hydraulic model runs. As described in the following sections however, this assumption was revised for both Moore’s Bridge and the culverts along the Owenacurra Millrace. Due to the improvement of conveyance and the removal of the original Moore’s Bridge piers following its collapse during Storm Babet the risk of blockage has been reduced. This is discussed further in Section 6.8.

4.2 Calibration Hydrology

4.2.1 2018 flood events

The Ballyedmond gauge was operational throughout 2018. Recorded water levels and flows (derived from the rating curve that was developed by Arup in 2022, which was subsequently revised post-Storm Babet[5]) are therefore available for both of the 2018 calibration/validation flood events and has been used to derive the inflow boundary for the Owenacurra river for these calibration events. Inflows for all the other fluvial boundaries of the model for both of these events (i.e. Dungourney river etc.) were derived by scaling the recorded Ballyedmond data to the relevant catchment using Qmed values (Table 4-1). The uncertainty associated with this approach is noted given the spatial and temporal variation in rainfall and groundwater baseflow across the various catchments. However, given the relative size and proximity of the various catchments, and in the absence of any other data for the other tributaries for this period, this method is deemed to be appropriate.

Tidal data observed on the day of the flood event was used to inform the calibration model.

Table 4-1 Peak inflows for each of the catchments for the April and December 2018 event

Watercourse	Qmed m^3/s (from Hydrology report)	Qmed Ballyedmond / Qmed catchment	Peak Inflow Apr 2018 event	Peak Inflow Dec 2018 event
Owenacurra	25.74	1.00	27.4	27.1
Dungourney	13.36	0.519	14.222	14.066
Glenathonacash	4.89	0.190	5.205	5.148
Elfordstown	3.1	0.120	3.3	3.264
Harrisgrove	3.61	0.140	3.843	3.801

4.2.2 December 2015 event

The Ballyedmond gauge was offline during the December 2015 event and therefore cannot be used to derive inflows for the event. Instead, the rainfall/runoff model FSSR16 was used to calculate the inflows using hourly rainfall data from the rain gauge at Moore Park. The total period for which inflows were derived was from 1 November 2015 until 30 December 2015.

As noted in the Midleton FRS hydrology report, rainfall data from Moore Park is broadly representative of the rainfall patterns across the Owenacurra River Catchment. There is however uncertainty associated with applying rainfall data from one catchment to a neighbouring catchment given the spatial and temporal variability in rainfall that can occur during storm events across catchments. Consequently, we cannot determine with a high degree of confidence the likely peak flows in any of the watercourses during the event.

We have addressed this uncertainty by considering both a low-end and high-end estimate of the peak flow during the event. The low-end estimate was calculated directly using the FSSR16 method (Table 4-2). Various high-end flow estimates were considered by applying percentage uplifts on the low-end flow. Peak water levels associated with each of the high-end flow estimates were then calculated with the hydraulic model and compared against the December 2015 recorded data at a number of locations. It was found that a 20% uplift in flow derived the best match between the modelled and recorded data. A 20% uplift has therefore been adopted as the high-end flow estimate. Both the low-end and high-end flow estimates are presented in the following table.

[5] 2022 rating curve is largely identical to the post-Storm Babet rating curve up to the Qmed threshold and therefor has no impact on the 2018 calibration event.

Table 4-2 December 2015 event – peak inflows

Watercourse	Peak Inflow calculated using FSSR 16 (m^3/s) – Low end estimate	Peak Inflow calculated assuming FSSR 16 +20% (m^3/s) – High end estimate
Owenacurra	43.86	52.416
Dungourney	21.77	26.124
Glenathonacash	8	9.6
Elfordstown	5.84	7.008
Harrisgrove	6.13	7.356

4.2.3 Estimated return periods for each recorded event

The peak flows for the calibration/validation events for the Owenacurra River in the study area are listed in Table 4-4. In order to assess the approximate return period of each event, the design flows for HEP OW3 (located at the upstream end of the scheme area) are presented in Table 4-32018 Calibration/ Validation

By comparing the peak calibration flows against the pre-Storm Babet design flows, it can be seen that both of the 2018 events approximate to a circa Q3 event on the Owenacurra. The low-end estimate of the December 2015 event corresponds to a circa Q25 event on the Owenacurra, while the high-end flow estimate corresponds to a circa Q45 year event.

Table 4-3 Design flows for OW3 pre-Storm Babet

HEP Location	Return Period (1in _ years)							
	2	5	10	25	50	100	200	1000
OW3	24.97	33.21	38.61	45.31	59.21	66.67	77.48	101.94

Table 4-4 Magnitude of calibration events

Event	Calibration Data	Type	Estimated Peak Flow(m3/s)	Return Period(1 in _ years) pre-Storm Babet
December 2018	Water levels recordings	In-bank Calibration	27	~3
December 2015	Wrack Marks / Observations	Out of bank calibration	43-53	~25–45
April 2018	Water levels recordings	In-bank validation	27	~3

4.3 2018 Calibration/ Validation

The calibration/validation events of April 2018 and December 2018 are highlighted with dashed lines in Figure 4-1. It is evident from inspection of the water levels at each of the four gauge locations, that the recording of the flood wave passing through the reach correlates well and that the Dungourney response was quite similar to Owenacurra.[6]

[6] The Dungourney Gauges were located near the upstream boundary, to facilitate data collection for the ground water assessment, and therefore not deemed suitable for hydraulic model calibration.

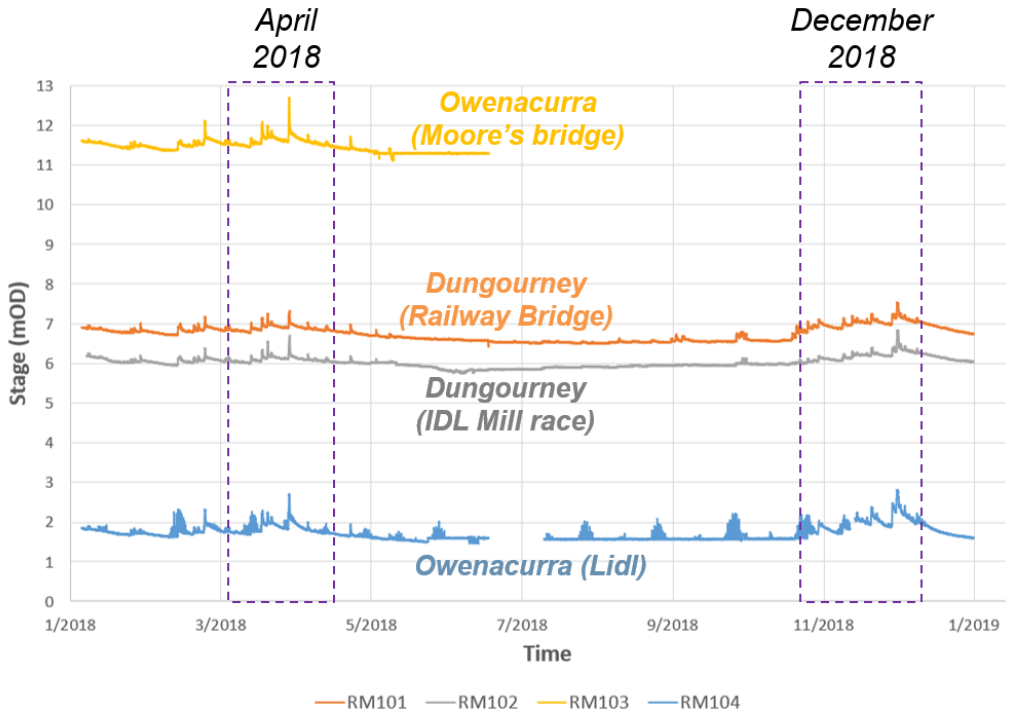


Figure 4-1 Recorded water level timeseries

4.3.1 December 2018 calibration at Lidl bridge

The water level calibration for the December 2018 event at the Lidl Bridge gauge is presented in Figure 4-2.

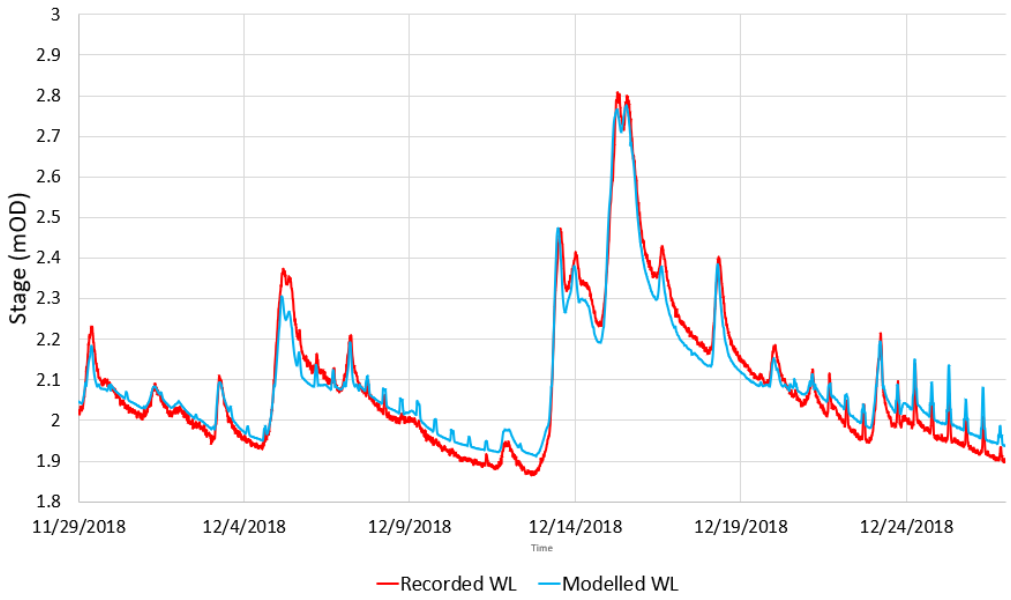


Figure 4-2 Lidl Bridge Water Level Calibration

It can be seen that there is a very good match between the modelled and recorded water levels. The peak modelled water level is within circa 10mm of the peak recorded water level which demonstrates the accuracy of the model in simulating peak water levels at this location for this circa 1 in 3year event. The performance of the model at the peak of the event is therefore well within the OPW's specified tolerance of +/-100mm and SEPA's 'high confidence' tolerance of +/- 150mm.

It can also be seen from Figure 4-2 that the model is well able to simulate changes in water level throughout the event – the modelled water level is in phase with the recorded data and reproduces both the timing and peaks in water level throughout the simulation period. The model overestimates low water levels by circa 10-20mm which is deemed to be marginal.

We note that the recorded peak water level (circa 2.8mOD) is in-bank at this location. This is evident in Figure 4-3 which plots the cross section at the Lidl bridge with the 2.8mOD water level indicated with the dashed red line.

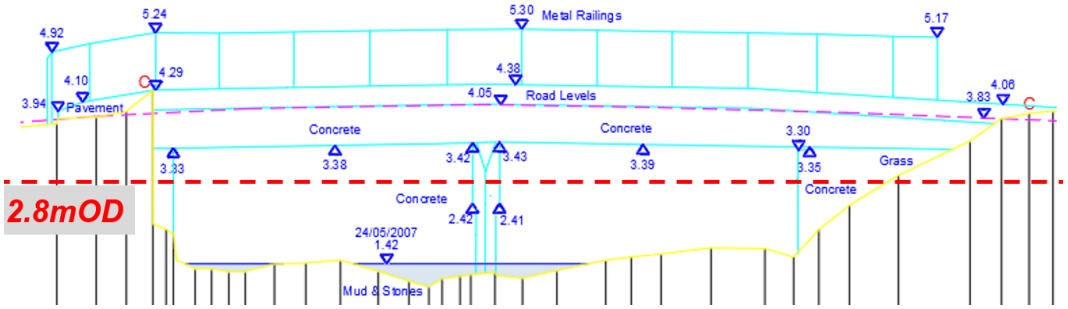


Figure 4-3 Cross section at Lidl bridge

An analysis on the differences between the modelled and measured water levels was carried out. It found that the maximum and minimum differences are circa 140mm and -130mm respectively which represents very good model performance. The average difference is 0mm.

4.3.2 April 2018 validation at Lidl bridge

Having calibrated the hydraulic model against the water level timeseries recorded at the Lidl bridge for the December 2018 flood event, a validation model run was also undertaken using the April 2018 flood event data. The water level validation plot is presented in Figure 4-4.

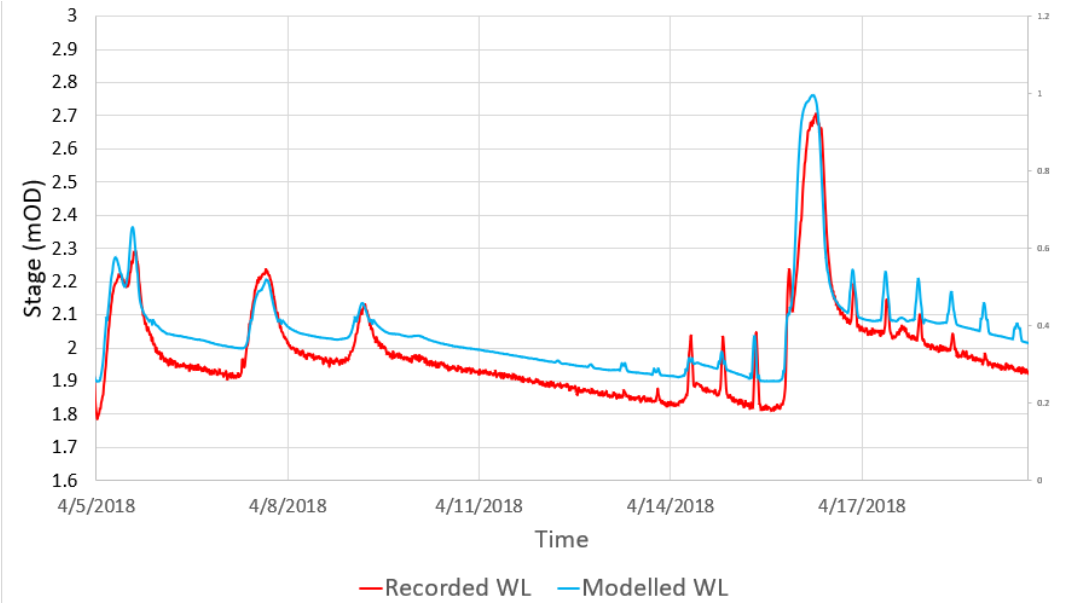


Figure 4-4 Lidl Bridge Water Level Validation - April 2018

It can be seen that the modelled water level timeseries is very well matched to the observed water level timeseries for the duration of the event. The model overestimates the peak water level by circa 80mm. The performance of the model at the peak of the event is therefore well within the OPW's specified tolerance of +/-100mm and SEPA's 'high confidence' tolerance of +/- 150mm.

A statistical analysis on the differences between the April 2018 modelled and measured water levels was carried out. It found that the maximum and minimum differences are circa 290mm and -150mm respectively. The maximum difference occurs on the rising limb of the flood event on the 16th of April and from an inspection of the water levels (Figure 4-4) it can be seen that the difference is largely due to the phase lag between the modelled and measured time series at that moment in time. Any slight shift in either of the time series would reduce this maximum difference. The average difference is circa 70mm.

4.3.3 September 2018 validation at Moore's bridge

The gauge at Moore's Bridge was removed from the Owenacurra River by a third party in July 2018 in an unauthorised action. It is therefore not possible to both calibrate and validate the hydraulic model at this location. We have therefore only calibrated the model at Moore's Bridge. It is noted however that additional survey data at this location is not deemed necessary as a sufficient amount of data has been collected to demonstrate the accuracy of the model at this location.

The model calibration at Moore's Bridge is presented in Figure 4-5.

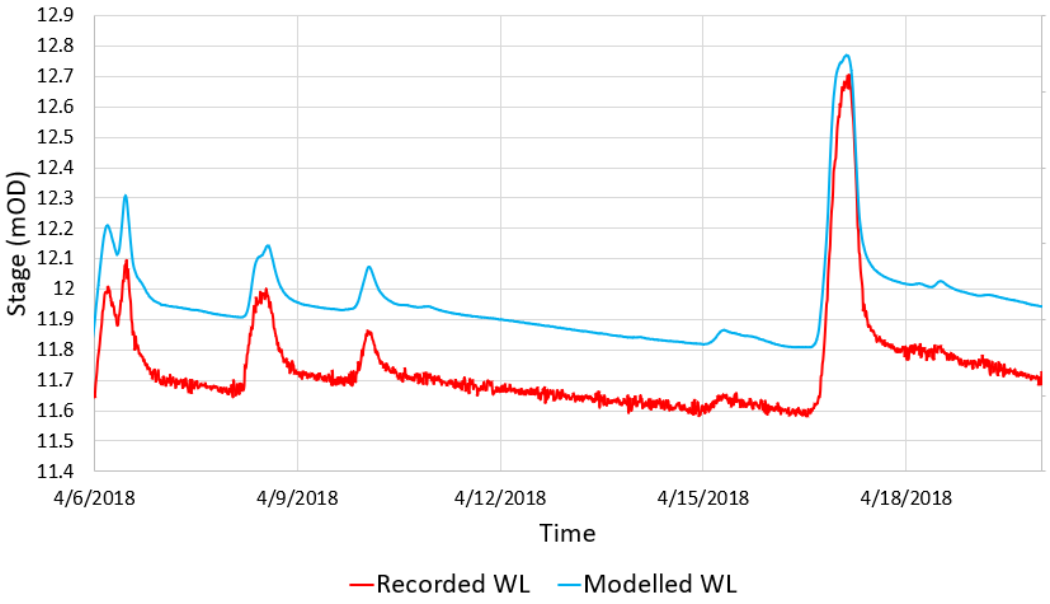


Figure 4-5 Moore's Bridge Water Level Calibration

It can be seen that the modelled peak water level is very well matched to the recorded peak water level as the difference is less than circa 80mm. The performance of the model at the peak of the event is therefore well within the OPW's specified tolerance of +/-100mm and SEPA's 'high confidence' tolerance of +/- 150mm.

The model overestimates low water levels by circa 200mm and there are a number of likely reasons for this including:

- The methodology for deriving inflows on the tributaries is approximate and may lead to uncertainties in the flows, particularly so for low flows;
- The Baseflow contribution at low flows can be a significant fraction of the total flow and may be under- or overestimated in the model;

The difference at low flows is not deemed critical to the study given that the objective of the study is to assess flood risk in Midleton and hence the primary objective of the hydraulic model is to accurately reproduce peak water levels on the primary watercourses. The calibration of the model at Moore's Bridge clearly demonstrates the ability of the model to accurately represent peak water levels at this location and is therefore considered suitable for use in the study.

Figure 4-6 presents the cross section at Moore's bridge with the peak observed water level from the April 2018 event superimposed on the plot. It is evident that the peak water level was in-bank at this location during the event.

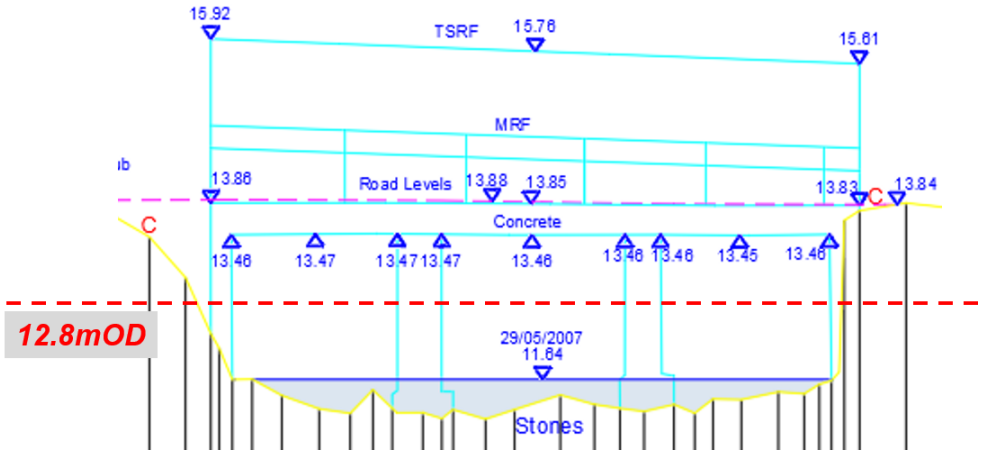


Figure 4-6 Moore's Bridge Cross Section

A statistical analysis for the validation run at Moore's bridge calibration was conducted.

The average difference between the modelled and recorded water levels was found to be 150mm which is higher than the difference in the peak water levels (circa 80mm). The average difference is influenced by the differences at low flows which act to skew the average differences.

4.4 December 2015 flood event calibration

Midleton was extensively flooded by a fluvial flood event in December 2015. As described in Section 2 of this report, a considerable amount of data was collected during and after the event by various authorities. This data has been used to calibrate the model.

An overview of the model calibration run for the whole of the scheme area (assuming no blockages of any of the structures) is presented in Section 4.5. The model calibration runs (considering both unblocked and blocked scenarios) for individual areas of the town, are then assessed in detail in Section 4.5.1 to Section 4.5.7, which

As noted in Section 4.2.2, in order to address the uncertainty over inflows for the event[7] we have considered both a low-end and high-end estimate of the flows. Both scenarios are considered throughout the presentation of the calibration results.

The calibration model does not include any discharges from groundwater. The flood extent in the vicinity of the Rugby club in Midleton is therefore underestimated by the hydraulic model, as expected given the significance of the groundwater component at that location.

4.5 Overview

Once a very good calibration against both of the flood events from 2018 was achieved (as detailed in the previous sections of the report), the December 2015 event was simulated with the hydraulic model using both the low-end and high-end flow estimates. The model simulation covered a 46-hour period and was run for the period from 29/12/2015 04:00 to 31/12/2015 02:00.

The modelled and recorded maximum flood extent for the event is presented in Figure 4-7 for the unblocked and low-end inflow scenario.

[7] Which is due to applying rainfall data from the rain gauge at Moore's Park to the catchment

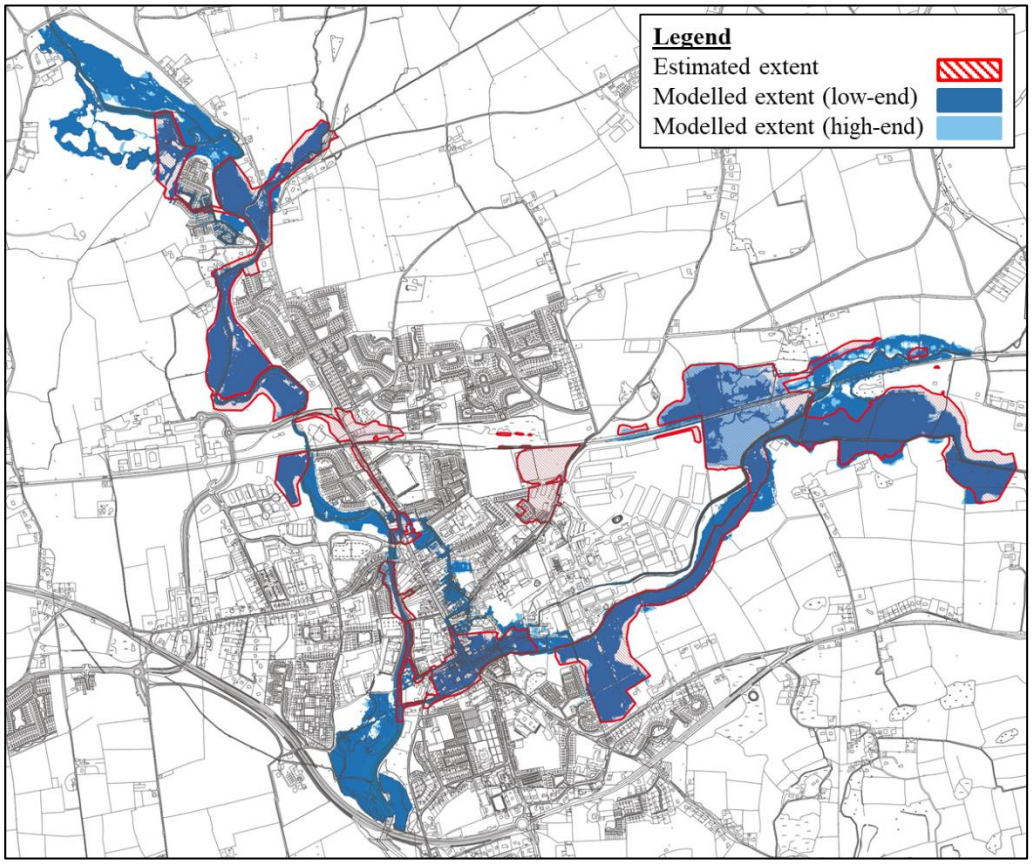


Figure 4-7 December 2015 flood event– maximum extents for entire area

It can be seen that the maximum flood extent, as predicted by the model, is very well matched to the recorded maximum extent in some areas (i.e. in Tir Cluain housing estate, North of the Northern Relief Road and along the Dungourney river), while in other areas of the model is underpredicting the maximum flood extent (i.e. North of the railway crossing at Millbrook Crescent and at Moore's Bridge). Each of these areas is considered in detail in the following sections of the report.

Maximum water levels from the hydraulic model are compared against observed peak flood levels at a number of locations in Table 4-5 below. These locations are mapped out spatially in the zoomed in figures (Figure 4-8, Figure 4-14, Figure 4-18, Figure 4-19, Figure 4-21, Figure 4-24 and Figure 4-25) in the following sections of the report. It is noted that Lauriston Estate and the Rugby Club area are not included in the table as the 1D/2D model does not simulate groundwater flooding.

Table 4-5: Modelled and observed water levels from the 2015 event

Location	Observed (mOD)	Modelled (mOD)	Comment	Model tolerance
Tir Cluain (West)	16.2mOD	16.3mOD	Model overestimates by circa 0.1m	Within OPW/SEPA model tolerance
Tir Cluain (East)	15.7mOD	15.8mOD	Model overestimates by circa 0.1m	Within OPW/SEPA model tolerance
Moore's Bridge	Circa 15.9mOD	13.4mOD	Model underestimates	Outside OPW/SEPA tolerance
Garden of Private Property ds of Moore's Bridge on right bank	13.9mOD	n/a	Model does not predict any flooding within the garden of the property	n/a
Railway Cottages	7.4mOD	7.51mOD	Model overestimates by circa 0.1m. Modelled depths however are sensitive to the specification of culvert blockages as detailed later in the report.	Outside OPW tolerance but within SEPA tolerance
Darling Buds Preschool	6.9mOD	6.9mOD	Minimal difference between model and recorded	Within OPW/SEPA model tolerance
Maxol Station/ Supervalu	5.15mOD	n/a	Model does not predict any flooding at this location	n/a
Community Centre	4.7mOD	4.7mOD	Minimal difference between model and recorded	Within OPW/SEPA model tolerance
Woodlands Estate	3.5 - 3.6mOD	4.1 – 4.2mOD	Model overestimates by circa 0.4m	Outside OPW/SEPA tolerance
Thomas Street	3.3 - 3.4mOD	n/a	Model does not predict any flooding at this location. The flooding is likely to be due to a local drainage capacity issue	n/a
Lower Main Street	2.9 - 3.2mOD	3.15mOD	Accurate model prediction	Within OPW/SEPA model tolerance
The Baby's Walk	3.1 – 3.2mOD	3.0 – 3.1mOD	Accurate model prediction	Within OPW/SEPA model tolerance

4.5.1 Calibration at Tir Cluain/Moore's Bridge

Figure 4-8 presents the modelled and estimated maximum flood extent for the area of Tir Cluain and Moore's Bridge.

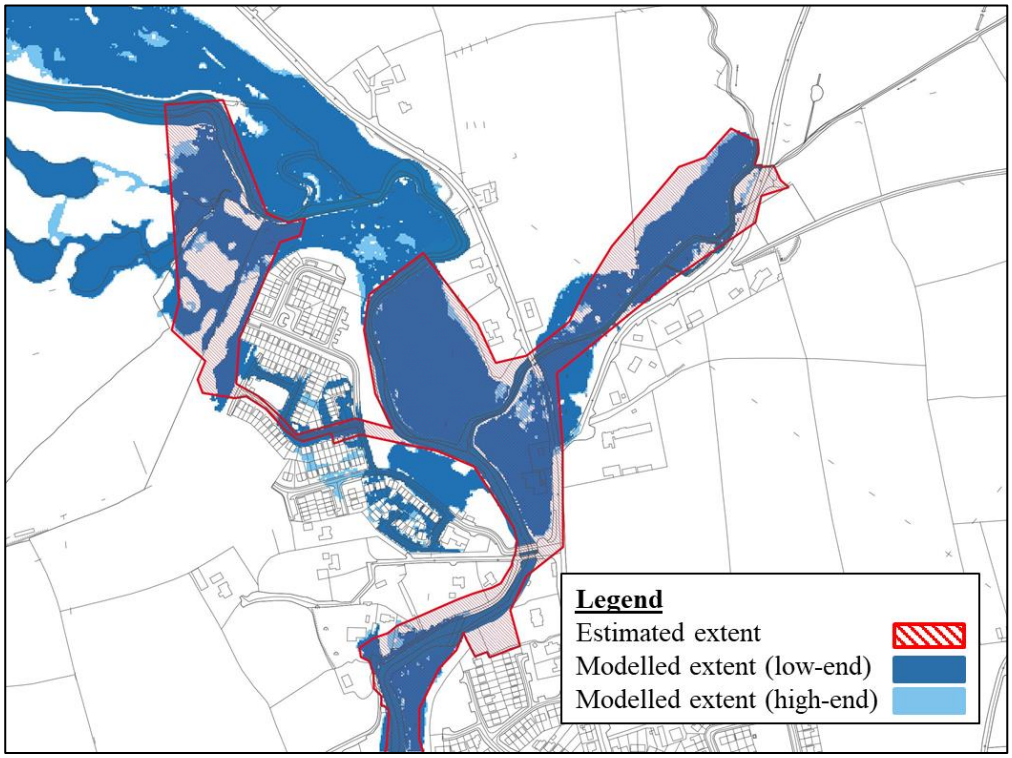


Figure 4-8 December 2015 Maximum flood extent calibration at Tir Cluain/Moore's Bridge

It can be seen that the model captures the primary mechanism by which Tir Cluain was flooded during the event, with water entering the estate from the Northwest through the perimeter fence from Water Rock golf course. The entry point predicated by the model is further to the north than suggested by the anecdotal estimate of the extent. We note however that flood water is unlikely to have entered the estate at this location given that the ground levels at this point are higher than the ground levels at the location where the model is predicting water entering the estate.

Once flood water entered the estate during the event, water flowed along the main road and inundated a number of residential properties as indicated on the figure. The model captures this overland flow route quite well although we note that the modelled extent is larger than the estimated extent. This difference can be attributed to uncertainty in the design inflow and to the influence of landscaping features (such as kerbs and paved areas etc.) which influence overland flow paths at shallow depths. These features have not been explicitly included in the model.

Maximum modelled flood depths in Tir Cluain for the low-end estimate of flows vary spatially across the site from circa 70mm to 180mm. This correlates well with anecdotal data which suggests that the flood waters in the estate were "ankle deep". Flood depths for the high-end estimate of flows are on average circa 200mm higher than the low-end estimate i.e. maximum flood depths are circa 270mm to 380mm.

It can also be seen from Figure 4-8 that the modelled extent at the confluence of the Owenacurra and the Glenathonacash Stream (in the vicinity of the junction at the local road to the East Cork Golf Club and the R626) is well matched to the recorded extent. The model is able to reproduce the mechanisms of flooding and overland flow paths in the vicinity of the junction.

Figure 4-8 indicates that the flood extent downstream of Moore's Bridge is underestimated by the model on both the right and left bank which is a consequence of the model underpredicting maximum water levels throughout the reach. We have compared the model results against the estimated peak water levels were derived from a post flood event survey of wrack marks along the reach which is presented in Figure 4-9 below.

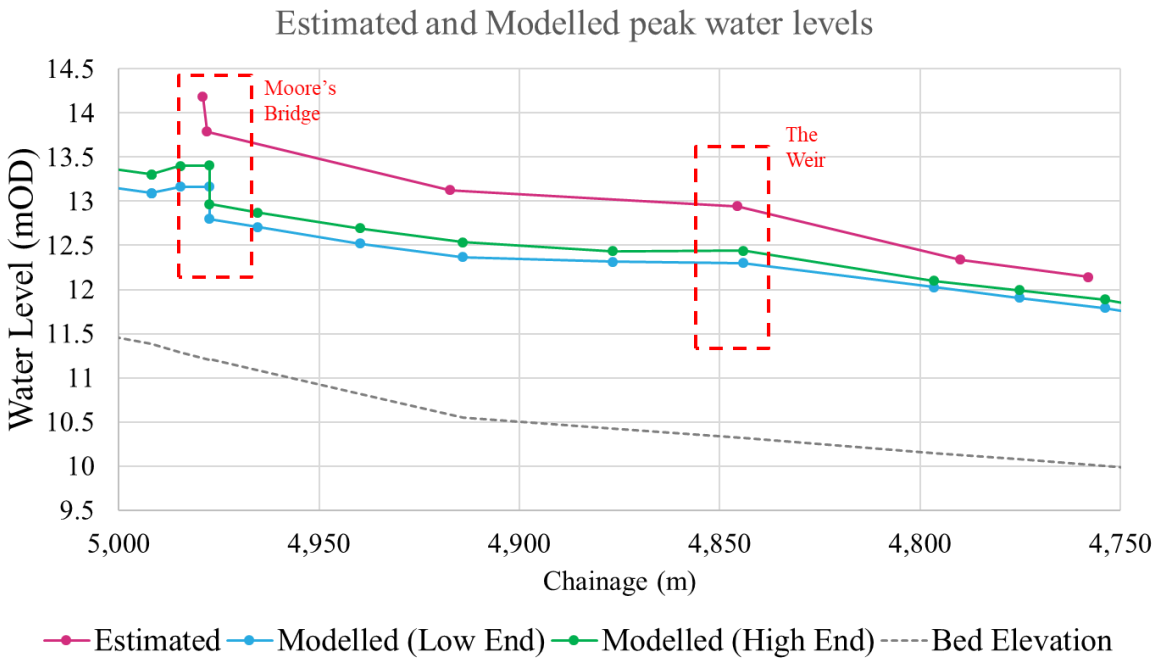


Figure 4-9 Peak water levels downstream of Moore's Bridge: modelled and recorded

The chainage on the graph covers the reach from Moore's bridge to the sharp bend in the river circa 160m downstream of the bridge. The 'modelled peak water level' refers to the maximum modelled water level as extracted from the model.

It can be seen that the model underestimates peak water levels immediately downstream of the bridge by circa 0.7m for the low-end flow estimate and by circa 0.6m for the high-end flow estimate. The difference between the model and the recorded levels reduces further downstream of the bridge.

As noted earlier in the chapter, the hydraulic model can accurately reproduce peak water levels from the April 2018 event at Moore's Bridge. This however is not the case for the December 2015 event as the model is unpredicting peak water levels by circa 0.6m/0.7m which is deemed as significant.

It is therefore likely that water levels in the December 2015 event throughout this reach were influenced by factors not accounted for in the set-up of the hydraulic model. These could have included some or all of the following:

- Changes in the channel geometry;
- Changes in the vegetation;
- Blockages that occurred in the channel and/or floodplain during the event;
- Ineffective flow areas associated with the circa 90-degree bend on the river that is located circa 250m downstream of the bridge.

Each of these are now considered.

Changes in the channel geometry

We are not aware of any significant changes in the channel geometry that have occurred in recent years throughout this reach. This is therefore unlikely to be the cause of the model underprediction.

Changes in the vegetation

Vegetation in the channel is subject to seasonal change and there is therefore likely to be some variation in Manning's value between April and December. While we have not accounted for this variation in the calibration model by considering different Mannings values for different stages of the year, these variations are very unlikely to account for the noted underprediction of peak water level for the December 2015 event.

Blockages in the channel

Figure 4-10 presents a photograph of Moore's Bridge taken after the event. It can be seen from the image that the trunk of a small tree is caught in one of the openings of the bridge and is acting as an obstacle to the flow. It is also evident that other debris has collected upstream of the bridge as a consequence of both the small tree and the bridge piers. It is possible that during the December 2015 event a much greater volume of debris was lodged upstream of the bridge and succeeded in blocking the opening at this location. Such an event would therefore have significantly elevated water levels upstream of the bridge.



Figure 4-10 Blockage at Moore's Bridge

Figure 4-11 presents a photograph of the Owenacurra downstream of Moore's Bridge which was taken after the event. It is evident from the figure that a large tree has fallen into the channel and is acting as an obstacle to the flow. It is possible that debris also collected on the upstream face of the fallen tree during the December 2015 event and therefore increased the overall size of the blockage in the channel.



Figure 4-11 Tree in the river downstream of Moore's Bridge

Figure 4-12 presents a photograph of the old weir in the channel downstream of Moore's bridge. There is significant vegetation on top of the weir which may well have become blocked during the event.



Figure 4-12 Remnants of old weir on left bank downstream of Moore's Bridge

There is therefore evidence to suggest that up to three separate blockages occurred in the vicinity of Moore's Bridge and/or downstream of the bridge during the December 2015 event. While the severity and magnitude of the blockages cannot be determined due to the absence of data, it is clear that any or all of these three blockages would have elevated water levels in the river during the event. As our model has not considered blockages through this reach, comparing its results against recorded data from the event is therefore not a like for like comparison.

A sensitivity analysis on the head loss coefficients of the structures at this location is considered later in this report for the design event in Section 5. The underprediction of the model at this location will be considered in detail as part of the freeboard analysis for the preferred scheme in the Options report for the project.

Ineffective flow areas

Another source of uncertainty which may contribute to the model underpredicting peak water levels from the December 2015 along the reach is the presence of ineffective flow areas associated with the circa 90-degree bend on the river located circa 250m downstream of Moore's bridge. From our site visits we have observed ineffective flow areas on the right-hand side of the channel (upstream of the bend) and on the left-hand side of the channel (immediately downstream of the bend).

Figure 4-13 presents an image of the bend in the river taken on a previous site visit. The thick dashed red lines distinguish between the effective and ineffective flow areas, and the thin solid red lines highlight the approximate extent of the ineffective flow areas. It is evident from the plot that a significant fraction of the cross-sectional area of the river at this location is ineffective at the time of the site visit. The 1D model at this location may therefore be too efficient as it assumes that the full cross-sectional area of the watercourse is conveying flow.

The ineffective flow areas at this location will be further considered as part of the optioneering of the scheme.



Figure 4-13 Ineffective flow areas downstream of Moore's Bridge

4.5.2 Calibration at Railway crossing/Northern Relief Road

Figure 4-14 presents the modelled and recorded maximum flood extent for the area in the vicinity of the Northern Relief Road.

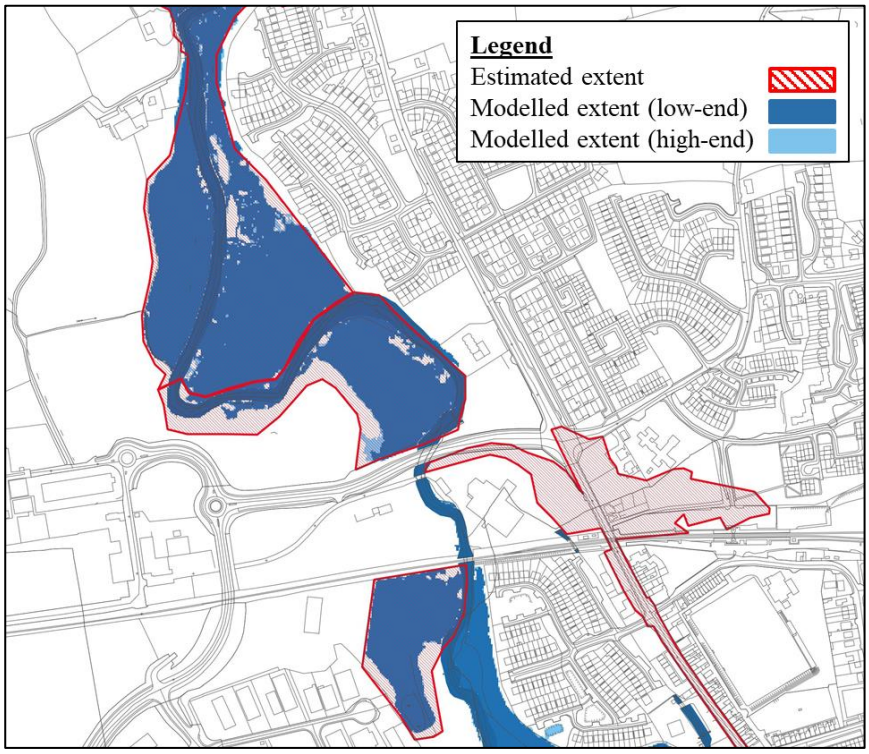


Figure 4-14 December 2015 Maximum flood extent calibration at Railway crossing/Northern Relief Road

It can be seen that the flood extent immediately adjacent to the Owenacurra upstream of the Northern Relief Road is a good match to the estimated extent.

The brick wall along the left bank of the river has been included in the model given that there is no record of it having collapsed (or partially collapsed) during the event. The wall ensures that water is kept in bank at this location. It is noted however that there is an opening in the wall through the entrance and exit of a small shed which was constructed to house hydraulic equipment and is integrated into the brick wall.

These openings have not been included in the model given that the finished floor level of the shed (8.59mOD) is above the modelled flood level for the December 2015 event.

The estimated extent downstream of the railway line does not indicate that any flooding occurred adjacent to the watercourse. This extent is likely to be incorrect as the threshold of flooding for this reach is the 10-year event which is less than the magnitude of the December 2015 event.

The model does not capture the out of bank flooding on the left bank of the Owenacurra River between the Northern Relief Road Bridge and the Old local bridge. Consequently, the model does not simulate flooding of the R626 and the Railway cottages or the associated overtopping of the railway line.

The minimum elevation of the left bank of the Owenacurra river between the Northern Relief Road and the Old access Road bridge is circa 7.81mOD. The peak water level in the model at this location is 7.604mOD for the low-end estimate and is therefore circa 0.2m below the level of the left bank. The peak water level in the model for the high-end flow scenario is 7.80mOD which is marginally below the level of the bank. The peak modelled water level is therefore close to the threshold of flooding of the watercourse at this location.

We note however that existing ground levels between the Owenacurra and the Railway Cottages (circa 8.145mOD) are higher than the level of the left bank. Floodwater will therefore have to overtop this high point for overland flow to reach the Railway cottages.

It is possible that water levels during the event were influenced by factors not accounted for in the set-up of hydraulic model such as changes in channel geometry or blockages in the channel. As we are not aware of any significant changes in the channel geometry that have occurred in recent times, blockages are considered the most likely mechanism to have caused elevated water levels during the event at this location.

In addition to the overtopping of the left bank of the Owenacurra, the Railway cottages and the surrounding area is also at risk from overtopping of the left bank of the millrace immediately upstream of the railway culvert as indicated in Figure 4-15. We note that there is a record of this mechanism having occurred during the December 2015 event. We have therefore assessed the risk of blockage at both of these locations.



Figure 4-15 Mechanisms of flooding North of the Railway Line

Through inspecting photographs of both the old access road bridge and the railway culvert (Figure 4-15) it is evident that both structures are at risk of blockage:

- From the post flood event photographs, it is evident that a service pipe has broken from its bracket at the old access road bridge and is protruding into the channel. Debris in the channel during the December 2015 event was therefore able to snag on the pipe and cause a blockage at the upstream face.

- There is significant vegetation immediately upstream of the Millrace Railway culvert which will block flow into the culvert. At the time at which the photograph was taken (April 2017) the right hand opening of the twin culvert is almost completely blocked. There is also significant vegetation further upstream of this location. Figure 4-16 presents a photograph at the location marked as point A in Figure 4-15. It can be seen that there is significant vegetation at this location which can block the flow.



Figure 4-16 Downstream face of culvert on Millrace (Refer to Point A on Figure 4-15)

- There is a significant amount of silt in the channel. Amelio noted that there is approximately 800mm of silt and sand in the bed of the railway culvert and provided a photograph of the culvert (Figure 4-17) in order to illustrate the extent of the siltation.



Figure 4-17 Photograph of the Millrace railway culvert – upstream face

We have therefore undertaken a sensitivity analysis on the calibration model by considering blockages at (1) the local access road downstream of the Northern Relief Road, and (2) at the railway culvert. Various combinations of blockage scenarios were assessed as part of the sensitivity analysis at both locations.

Figure 4-18 presents the findings of the blockage sensitivity which shows the modelled and estimated flood extent associated with 40% blockage at the old access road bridge and 95% blockage at the millrace culvert.

It can be seen from the figure that in this scenario the area of the Railway cottages is flooded by water from the Owenacurra and the millrace culvert.

It can also be seen that water has overtopped the railway line and has inundated Mill Road at shallow depths. The modelled route of flooding from the Owenacurra is also captured by the model.

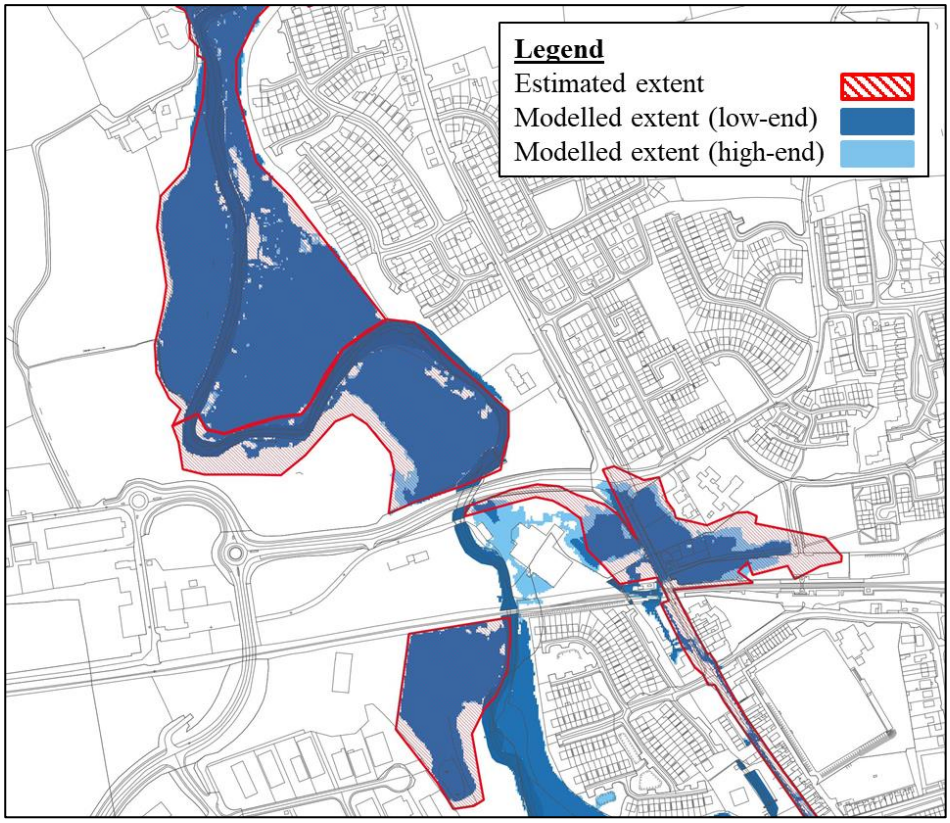


Figure 4-18 40% blockage at the old access road bridge and 95% blockage at the millrace culvert

The estimated peak water level at the Railway cottages during the event was recorded as 7.4mOD from a post flood event survey. The peak modelled water level at the Railway cottages is 7.27mOD for the low-end scenario and 7.51mOD for the high-end scenario. This demonstrates the ability of the blockage scenario model to reproduce the observed flood level and extent at this location as the low flow estimate and high flow estimate bound the recorded peak flood level.

It is therefore evident from the sensitivity analysis that the model can reproduce the mechanisms of flooding and flood extents downstream of the Northern Relief Road when blockages in the channel are included. Although there is no direct record of a blockage having occurred at either of these locations during the December 2015 event, it is noted that there is no record of these structures having been inspected during the event. There is however clear evidence to suggest that a blockage at both locations did occur at one or both of these locations during the event. There is no way however to assess the severity of what may have actually occurred during the event. This uncertainty will be addressed as part of the optioneering for the scheme.

4.5.3 Calibration at downstream end of the culvert millrace

Figure 4-19 presents the model calibration for the downstream section of the millrace culvert for the unblocked scenario. It can be seen from the figure that water escapes the left bank of the millrace immediately upstream of the culvert as shown in Figure 4-20. Once water exits the millrace at this location it flows east onto the main street and then flows south before inundating a large area of the town at shallow depths.

There is no record of water having escaped the millrace at this location during the 2015 event, nor any evidence to suggest that the area of the town to the east of Main Street (Durys Avenue/ Connolly Street) was flooded during the event. It is therefore likely that the model overestimates water levels at the downstream end of the millrace and that peak water levels remained in bank during the event.

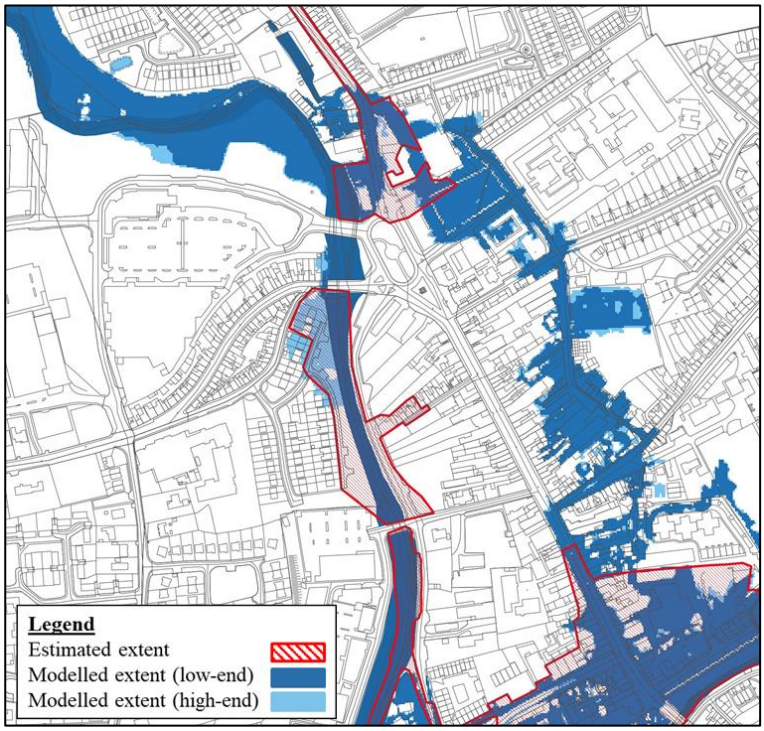


Figure 4-19 Water escaping the downstream end of the millrace

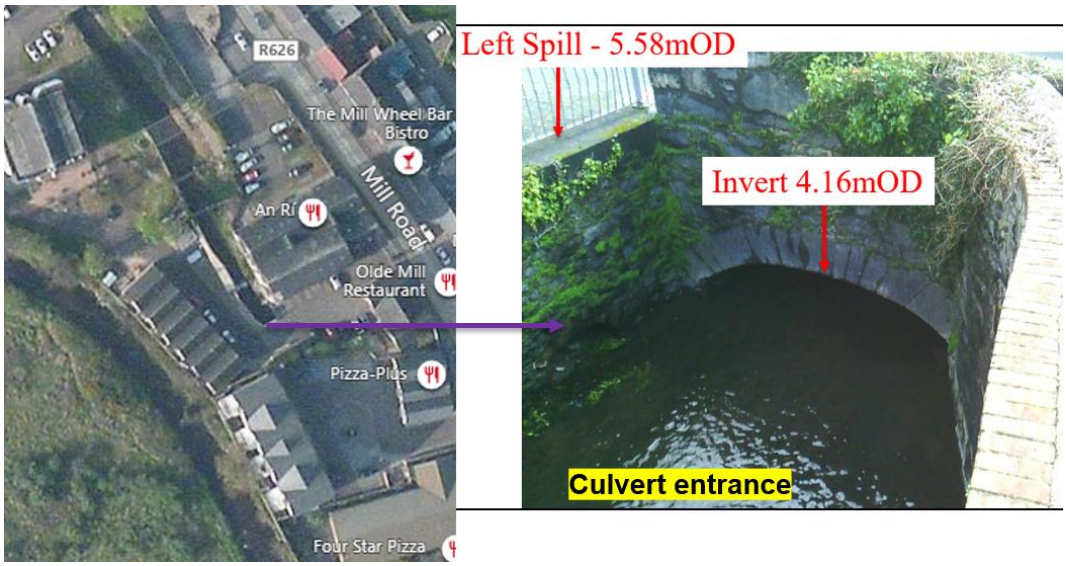


Figure 4-20 Downstream end of the millrace

There are a number of reasons as to why the model may be overestimating water levels at the downstream end of the millrace for the December 2015 event:

- Water likely escaped the millrace upstream of this location during the event as described in the previous section of the report. This scenario would lead to a reduced flow rate in the millrace and hence reduced water levels at the downstream end. As this mechanism may not be captured by the model, the flows and water levels at the downstream end of the mill race may be overestimated;
- Siltation and/or additional blockages may be throttling the flow in the millrace further upstream. The model may therefore be over predicting the flow rate through the millrace and hence overestimating water levels at the downstream end;
- There is an old and disused sluice gate at the entrance to the millrace. It is our understanding that this structure is no longer in use. However, it was likely acting as a throttle on the flow during the 2015 event and therefore reducing water levels downstream of it.

Based on our assessment of the mechanisms of flooding and recorded flood extents along the mill race culvert, we have utilised the 95% blockage of the railway culvert upstream scenario (which was also analysed and presented in the previous section of the report) in order to assess water levels at the downstream end of the Millrace for the December 2015 event. In this scenario, the flow rate through the downstream end of the culvert is reduced and hence peak water levels are also reduced. Table 4-6 presents the results of the model. It can be seen from the table that peak water levels are sufficiently reduced in order to prevent water escaping the millrace at the downstream end which is in keeping with what occurred during the event.

Table 4-6 Water levels at downstream end of culvert

Scenario	Water level at culvert (mOD)	Difference between WL and Overtopping level (5.58mOD)
Unblocked, low-end flow estimate	5.744	0.164
Unblocked, high-end flow estimate	5.778	0.198
95% blockage at railway culvert, low-end flow estimate	5.082	-0.498
95% blockage at railway culvert, high-end flow estimate	5.481	-0.099

4.5.4 Calibration in the town centre (Owenacurra dominated)

Figure 4-21 presents the modelled and estimated maximum flood extent for the main area of the town impacted by the Owenacurra for the millrace blocked scenario.

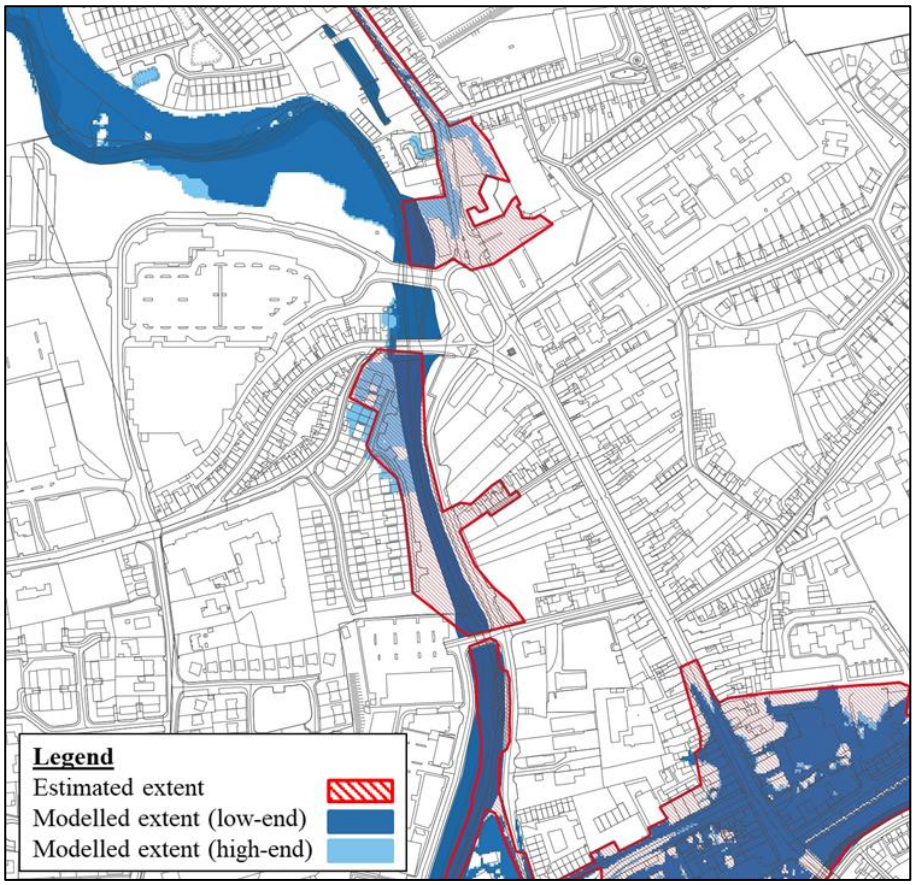


Figure 4-21 Flood extent calibration in the town (from Owenacurra)

It can be seen from the figure that the modelled flood extent along the main channel of the Owenacurra is generally well matched to the recorded extent for the high-end flow scenario.

The estimated extent indicates that Thomas Street was flooded during the December 2015 flood event. This may not be accurate, and it is unlikely that by overland flow from the left bank of the Owenacurra was a mechanism of flooding for a number of reasons as follows:

- There is no record of Riverside Way (i.e. the road adjacent to the Owenacurra) being inundated during the event;
- There is a brick wall in between the Owenacurra and Thomas Street (Figure 4-22) which would have limited floodwater from the Owenacurra inundating Thomas Street (it is however noted that there are two sets of steel doors located along the brick wall and water may have been able to flow through the gaps between the gates and the walls).

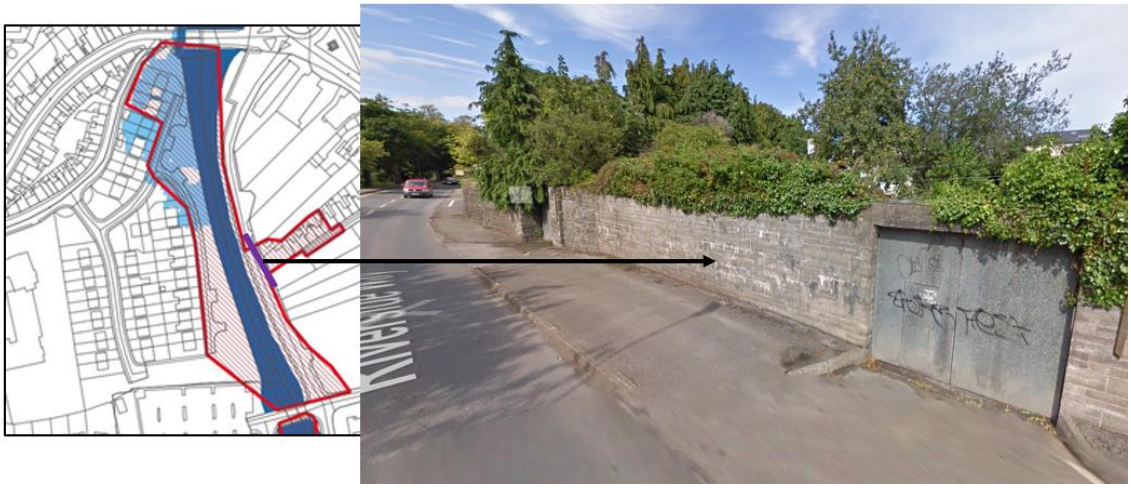


Figure 4-22 Boundary wall between Thomas Street and the Owenacurra

- Based on the model, the threshold of flooding of the Owenacurra in the immediate vicinity of Thomas Street is circa 73m^3/s, which is significantly greater than the best estimate of flow experienced during the event i.e. there is unlikely to have been a sufficient volume of water in the river to overtop the left hand bank during the event which could have led to Riverside Way and Thomas Street being inundated.

The source of the flood water on Thomas Street is therefore likely to have been pluvial i.e. the local drainage system was unable to accommodate the volume of rain falling in the localised urban catchment. This mechanism would have been made more severe by the high-water level in the Owenacurra which would have prevented surface water from Thomas Street from draining into the river.

4.5.5 Calibration at the upstream end of the Dungourney River

During the flood event, CCC installed a trench box to cut off (or at least significantly reduce) the inflow to the IDL Millrace during the December 2015 flood event (Figure 4-23).



Figure 4-23 Photo of trench box installed by CCC to limit the inflow to the millrace

This scenario has been considered by including a blockage unit in the model on the millrace immediately downstream of its confluence with the Owenacurra. It has been assumed that the Trench box leads to a 90% blockage of the millrace at the peak of the event.

Figure 4-24 presents the modelled and estimated maximum flood extent for the upper reach of the Dungourney river within the study area. While the trench box is included in the model, we note that all the other structures in the Dungourney have been assumed to be unblocked.

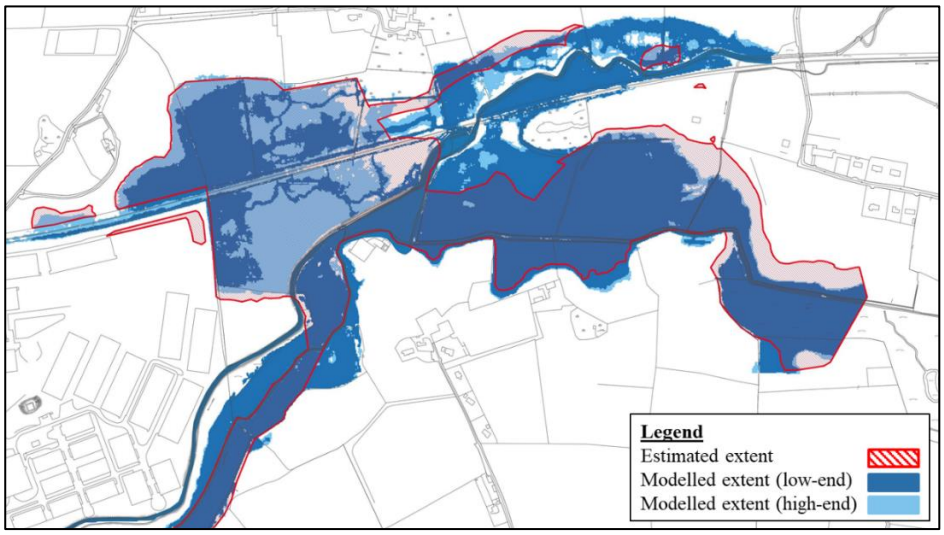


Figure 4-24 Calibration extent – upper Dungourney (with trench box)

It can be seen that the modelled extent is well matched to the recorded extent for the upper reach. We note that discharges from groundwater have not been included in the model. The inundation of the area in the vicinity of the Rugby Club and Lauriston estate is therefore, as expected, underestimated in the modelled flood extent.

4.5.6 Calibration at the downstream end of the Dungourney River in the centre of Midleton

Figure 4-25 presents the modelled and estimated maximum flood extent for the lower end of the Dungourney river.

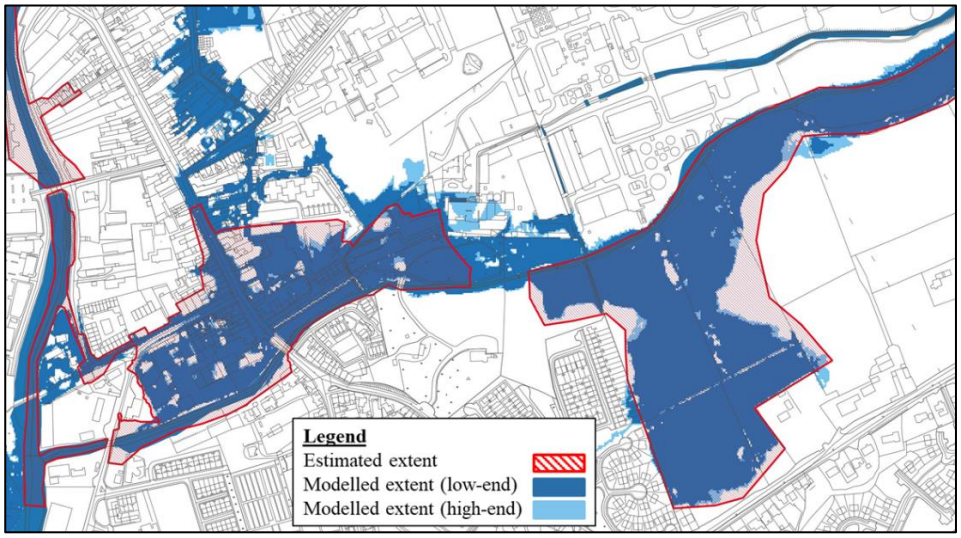


Figure 4-25 Calibration extent – Lower Dungourney (with trench box)

It can be seen from the figure that the modelled extent is well matched to the estimated extent in the lower reach.

4.5.7 Assessment of the impact of the Trench box

In order to assess the impact of deploying the trench box on the Dungourney, we have re-run the hydraulic model with the trench box removed (i.e. by assuming a zero blockage at the upstream end of the millrace). Figure 4-26 presents the findings of the analysis of the low flow estimate with Figure 4-27 showing the equivalent analysis for the high flow estimate.

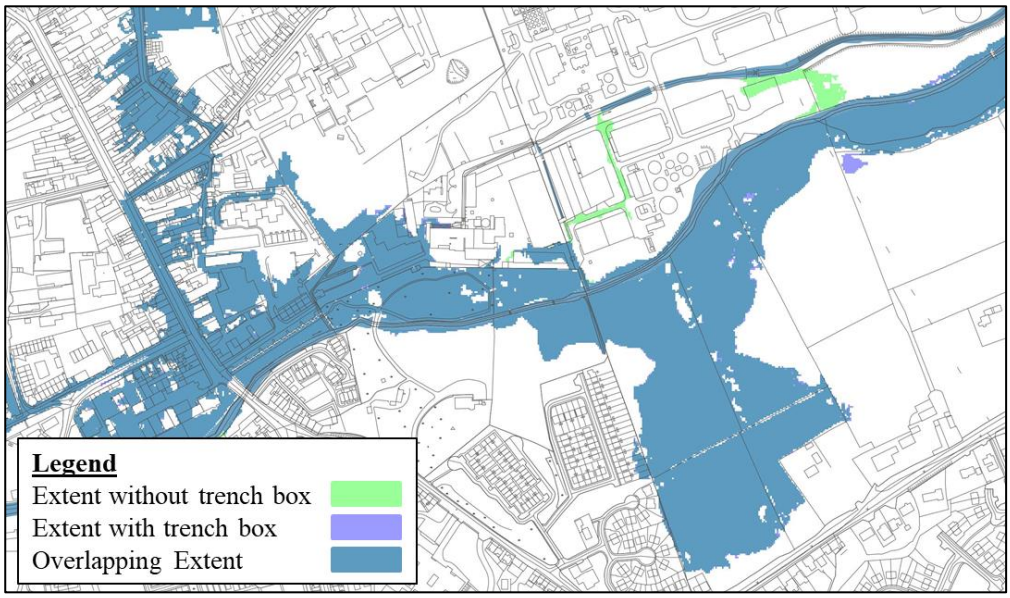


Figure 4-26 Calibration extent for low flow estimate – with and without trench box

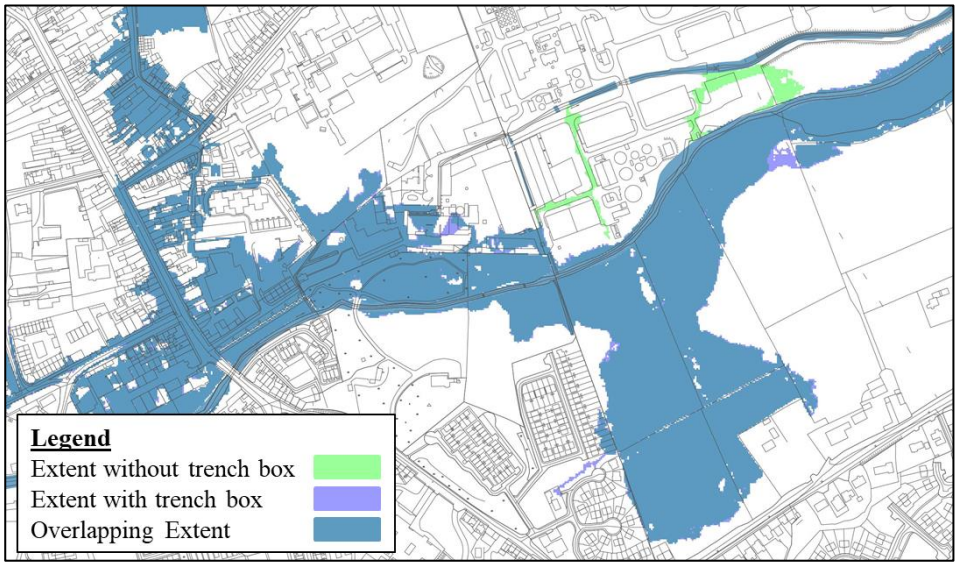


Figure 4-27 Calibration extent for high flow estimate – with and without trench box

It can be seen that with the trench box removed a greater volume of water would have entered the IDL Millrace at the upstream end and flow downstream. In this scenario, parts of the IDL site would have been inundated by water overtopping the right bank of the IDL Millrace at two separate locations for both the low and high-end scenarios. Once water overtops the watercourse, it would have flowed south towards the Dungourney following the existing ground contours Table 4-7 and Table 4-8 compare the predicted flood levels at a number of locations for the with and without trench box scenarios, for the low and high flow scenarios respectively.

Table 4-7 Water levels on Dungourney low flow estimate with and without trench box

Scenario	Location 1 (mOD) Broderick Street East	Location 2 (mOD) Town Centre	Location 3 (mOD) Distillery Walk
No trench box, low-end flow estimate	3.12	3.12	2.97
With trench box, low-end flow estimate	3.12	3.13	2.98

Table 4-8 Water levels on Dungourney high flow estimate with and without trench box

Scenario	Location 1 (mOD) Broderick Street East	Location 2 (mOD) Town Centre	Location 3 (mOD) Distillery Walk
With trench box, high-end flow estimate	3.16	3.17	3.03
No trench box, high-end flow estimate	3.14	3.15	3.01

With the trench box in place, a greater volume of water entered the Dungourney River. However, the increase in peak water level in the Dungourney (associated with having the trench box in place) is very minor (circa 0.02m) and does not materially change the downstream flood extents. It can therefore be concluded that the deployment of the trench box during the December 2015 prevented minor flooding of the IDL site but did not have any significant impact on peak water levels in the town centre.

4.5.8 Calibration at Water Rock

The upper section of the Water Rock model (upstream of the Cave system) was calibrated against the peak water level from the 2015 flood event. It was assumed that the cave was 95% blocked to replicate the lack of capacity of the cave. A small pass on flow was required to ensure the hydraulic model to not run dry and result in stability issues. The estimated peak water level at Water Rock House was 11.0mOD, estimated based on observed levels during the event (Figure 4-28). The modelled peak water at this location was 10.9mOD which is 0.1m lower than the observed level.

The model is therefore able to reproduce the peak water levels upstream of the Cave system. The performance of the model at the peak of the event is therefore within the OPW's specified tolerance of +/- 100mm and SEPA's 'high confidence' tolerance of +/- 150mm.



Figure 4-28 Water Rock House during the December 2015 flood event

The findings of our calibration model indicate that there is a significant overland flow route to the East of Water Rock House i.e. when the local access road is overtopped, there is a sufficient head of water over the road to cause a large volume of water to flow east. The road was overtopped during the December 2015 event and a number of the properties adjacent to the road were inundated.[8] During the course of the study, there have been some suggestions that properties in the North Point Business Park, which lies further east, may also have experienced some flooding. This anecdotal data would support the findings of the calibration model. This data however is not considered reliable, and no firm evidence of the business park being inundated has been found.

Some anecdotal data on the impact of the December 2015 event downstream of the Cave system was collected as part of the project. The downstream section of the model was not however calibrated against the data as the outflow from the Cave system during the event cannot be determined with any degree of confidence, and therefore any attempt to model the event would very likely lead to incorrect conclusions being drawn. The accuracy of this section of the model was therefore ensured by following best practice in the model build and adopting standard values for the various model parameters as detailed in relevant industry literature.

4.5.9 Calibration at Ballinacurra

The Ballinacurra Hydraulic model was not calibrated against the December 2015 flood event due to the lack of anecdotal data from the site during the event. The accuracy of the model was therefore ensured by following best practice in the model build and adopting standard values for the various model parameters as detailed in relevant industry literature. A sensitivity on the key model parameters of the model will be undertaken as part of the optioneering.

[8] The January 2016 Flood Risk Review report states that the 'lands at and adjacent to Water Rock House experienced flooding"

4.6 Summary of the December 2015 model calibration

The model is well calibrated to the December 2015 event in certain areas of Midleton but is less well calibrated in other areas for the unblocked scenario. The performance of the model at the peak of the event in a number of locations is within the OPW's specified tolerance of +/-100mm and SEPA's 'high confidence' tolerance of +/- 150mm. There are however a number of areas where the model is underpredicting water levels and therefore falls outside of the tolerance. The underprediction in these areas can be attributed to the uncertainty associated with a number of items:

- Calibration hydrology – utilising recorded rainfall from a neighbouring catchment to derive historic flood flows is quite uncertain due to the spatial and temporal variation of rainfall patterns during storm events;
- Occurrence of blockages at structures and within the channel downstream of Moore's Bridge which can lead to under – or over prediction of water levels in the model;
- Formation of ineffective flow areas (particularly downstream of Moore's Bridge) which can lead to an underprediction of water levels;
- A considerable amount of calibration data is anecdotal and is therefore itself subject to uncertainty.

4.7 Conclusions of the hydraulic modelling calibration

The 1D/2D hydraulic model of the Owenacurra/Dungourney is very well matched to two circa 1 in 3-year return period events that occurred in 2018. The model is also well calibrated to the December 2015 event in certain areas of Midleton. There is a strong basis to suggest that blockages occurred during the event and when these are considered, the model is capable of reproducing the mechanisms of flooding which were observed during the event and reproducing peak water levels across Midleton.

The upper section of the Water Rock model is calibrated against the peak water levels from the 2015 event. The lower section of the Water Rock model and the Ballinacurra models are not calibrated against recorded data due to a lack of data. The accuracy of these models was therefore ensured by following best practice.

5. Sensitivity Analysis – Current scenario

5.1 List of Sensitivity Runs

A number of sensitivity analysis runs were undertaken in order to assess how the Q100 design water levels for the existing scenario may vary under different model assumptions. It is noted that the model sensitivity analysis was completed prior to the hydrology updates subsequently completed following Storm Babet (2023). For this reason, the Q100 flow referred to in Section 5 (this Section) of the report, is the pre-Storm Babet design flow. The sensitivity runs also use the pre-Storm Babet design flows such that there is a like for like comparison.

A complete list of the sensitivity runs is presented in the table below. The results of the sensitivity model runs are presented in the following sections of the report.

Table 5-1 List of sensitivity model runs

Sensitivity Parameter	SA model no.	Model runs
Manning's value	1a	+20% increase in the Manning's number across both the 1D and 2D model
	1b	-20% increase in the Manning's number across both the 1D and 2D model
Culvert/Bridge Head Loss Coefficients	2	Specification of a higher head loss coefficient parameter at critical hydraulic structures. Three separate bridges were examined: - Lidl Bridge (Owenacurra) - Lewis Bridge (Dungourney) - Bridge upstream of Clohessy's Yard (Owenacurra Tributary)
Bridge unit type	3	Specification of an alternative bridge unit at key hydraulic structures. The following bridge was considered: - Lewis Bridge changes from an Arch Bridge unit to a USBPR unit
Small Catchment Hydrology	4	Specification of an alternative (and more conservative) hydrological estimation method for the small catchments in the study area.
Design Flows	5	20% increase in the design inflows (discussed as part of the MRFS runs)
2D grid resolution	6	Change the grid resolution of the 2D model from 2m to 4m

5.2 Changes to the Manning's number

A 20% increase and a 20% decrease was applied to the Manning's number for both the 1D and 2D model domains to test the sensitivity of the model to changes in roughness values.

Figure 5-1 presents a longitudinal plot of the maximum water levels along the main section of the Owenacurra River for both the baseline and the Manning's sensitivity simulations. It is noted that the dashed lines on the plot relate to the left and right bank elevations along the reach.

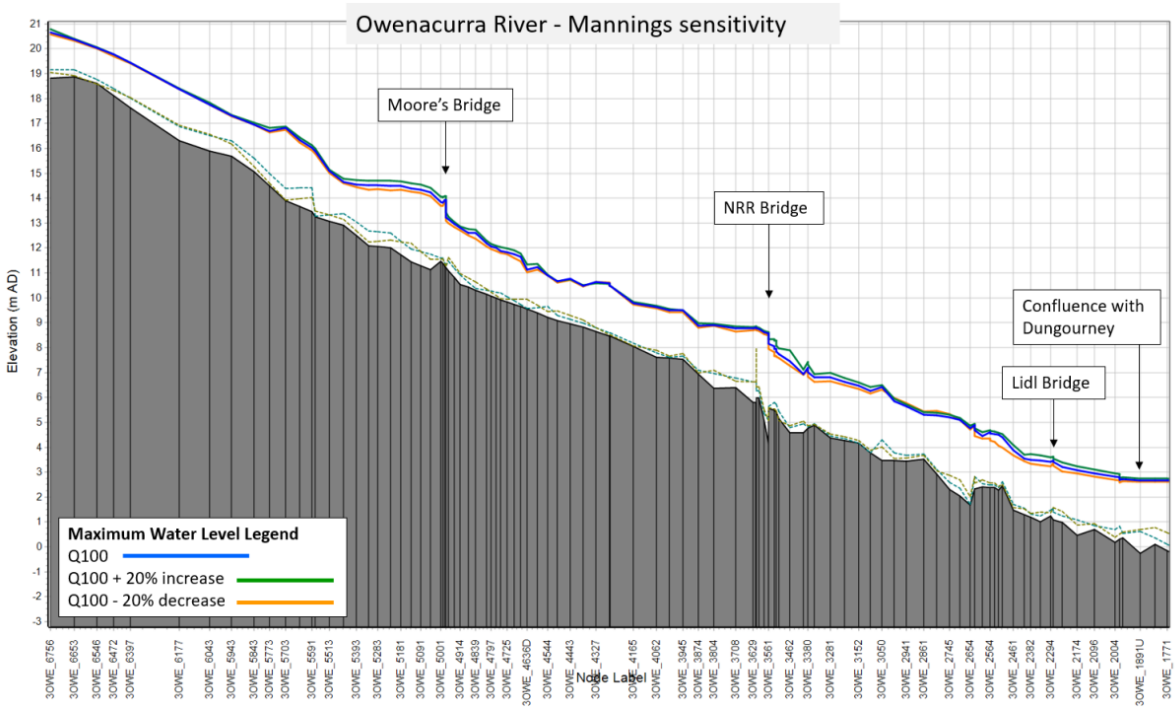


Figure 5-1 20% increase/decrease in the Manning's number –Owenacurra River

It can be seen from the plot that the 20% increase in Manning's number generally results in higher water levels throughout the reach. The increase however is not constant and varies along the reach. Upstream of both Moore's Bridge and the Lidl bridge the increase in maximum water level is circa 180mm while in the areas where the floodplain is relatively wide and unconstrained (i.e. circa 600m upstream of the NRR where the Owenacurra meanders adjacent to the Willowbank Housing estate) the increase in maximum water level is generally circa 20mm.

The decrease in Manning's number generally results in the lower maximum water levels throughout the reach. The decrease in levels however also varies throughout the reach. The most significant decreases are upstream of structures (Moore's Bridge and Lidl Bridge) where the reduced Manning's number simulation results in maximum water level circa 160mm lower than the baseline. In the relatively wide sections of the floodplain the decrease in maximum water level is circa 20mm.

Figure 5-2 presents a longitudinal plot of the maximum water levels along the main section of the Dungourney River for the baseline scenario and the Manning's number sensitivity analysis model simulations.

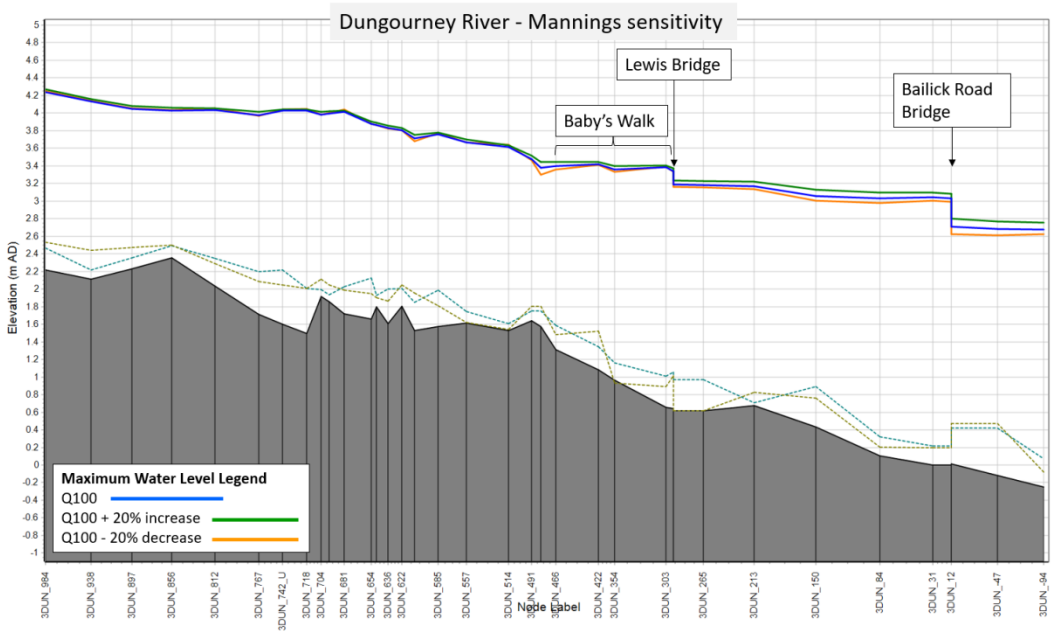


Figure 5-2 20% increase/decrease in the Manning's number – peak Q100 water levels along the Dungourney River

It can be seen that the change in maximum water levels generally follows the same pattern as for the Owenacurra river – the increase in the Manning's number results in higher water levels throughout the reach, while the decrease in the Manning's number reduces water levels.

The changes in water level varies along the reach. Between Lewis Bridge and Bailick Road Bridge the increase in maximum water level is circa 70mm. Upstream of the Lewis Bridge where the floodplain is relatively wide and expansive the increase is circa 40mm. The decrease in maximum water levels associated with a reduced Manning's value is circa 55mm downstream of the Lewis Bridge and 25mm upstream of the Lewis Bridge.

It is therefore evident from the results that the model is sensitive to variations in the Manning's number. The largest variations in maximum water levels generally occur upstream of hydraulics structures and where the floodplain is relatively constrained.

The higher Manning's number scenario will lead to a greater volume of water exiting the channel (when compared against the baseline) and will increase flood risk in certain areas of Midleton. The reduced Manning's will lead to a reduced volume of water exiting the channel (when compared against the baseline) as the watercourse will convey a greater volume of water in this scenario.

The Q100 extent for both of these sensitivity scenarios however is not significantly increased or decreased from the baseline as the change in the volume of water entering the floodplain is small in the context of the overall volume of watering entering the floodplain in the baseline scenario.

5.3 Culvert/Bridge Head Loss Coefficients

5.3.1 Lidl Bridge

The model was simulated with an increased head loss coefficient at the Lidl Bridge. The baseline model has the coefficient set at 1, and this was increased to 2 for the sensitivity run.

Figure 5-3 presents a longitudinal plot of the Q100 maximum water levels along the relevant reach of the Owenacurra River upstream of the bridge for both the baseline scenario and the increased head loss sensitivity.

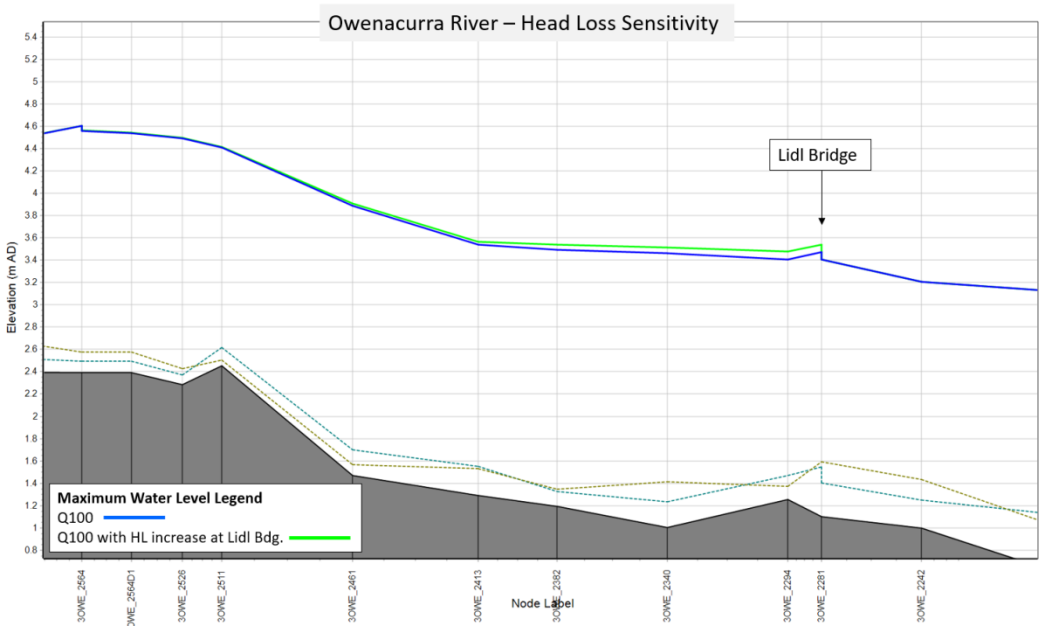


Figure 5-3 Increased head loss coefficient at the Lidl Bridge on the Owenacurra River

It can be seen from the plot that the increased head loss coefficient has a minor impact on the maximum water levels upstream as the increase in the maximum water level is less than 70mm. This higher water level does not increase flood risk upstream of the bridge as it does not materially increase the volume of water spilling from the Owenacurra River to the floodplain.

5.3.2 Bridge upstream of Clohessy's Yard

The model was simulated with an increased head loss coefficient at the Bridge upstream of Clohessy's Yard (Carrigogna Bridge). The baseline model has the coefficient set at 1, and this was increased to 2 for the sensitivity run.

Figure 5-4 presents a longitudinal plot of the Q100 maximum water levels along the relevant reach of the tributary upstream of the bridge for both the baseline and the increased head loss sensitivity model runs.

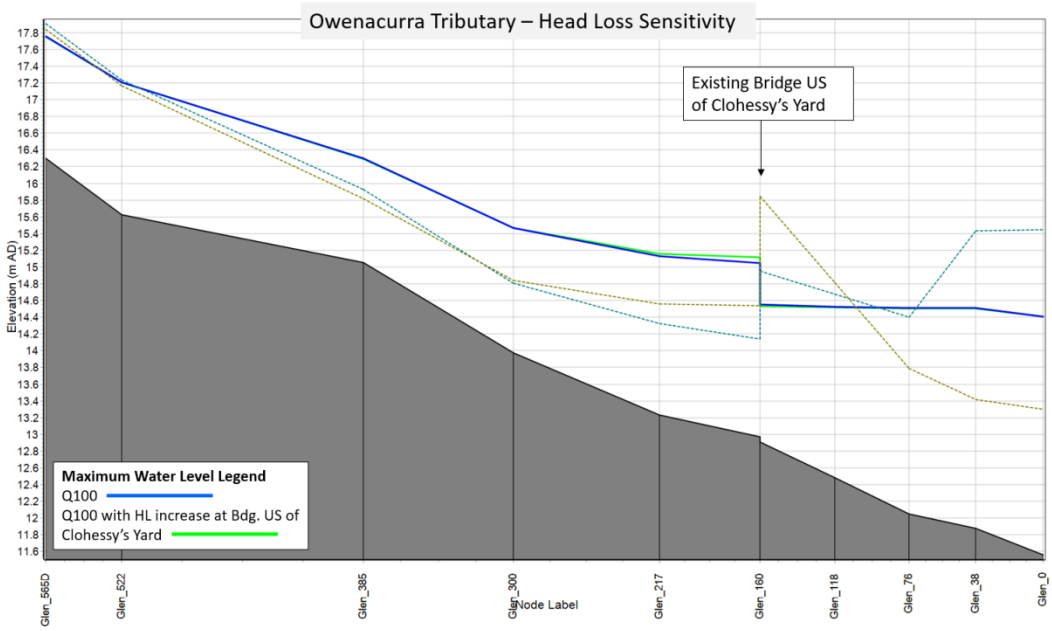


Figure 5-4 Increased head loss coefficient at the Bridge upstream of Clohessy's Yard (Carrigogna Bridge)

It can be seen from the plot that the increased head loss coefficient has a relatively minor impact on the maximum water levels upstream as the increase in level is less than 65mm. This increased maximum water level does however lead to a marginally greater volume of water spilling from the tributary to the floodplain and hence increases flood risk upstream.

The findings of the sensitivity analysis at this location will be considered as part of the optioneering.

5.3.3 Lewis Bridge

The model was simulated with an increased head loss coefficient of 2 at the Lewis Bridge in The Baby's Walk.

Figure 5-5 presents a longitudinal plot of the Q100 maximum water levels along the relevant reach of the tributary upstream of the bridge for both scenarios. (It is noted that the plot also presents the findings of the Bridge Unit replacement sensitivity which is discussed in the following section).

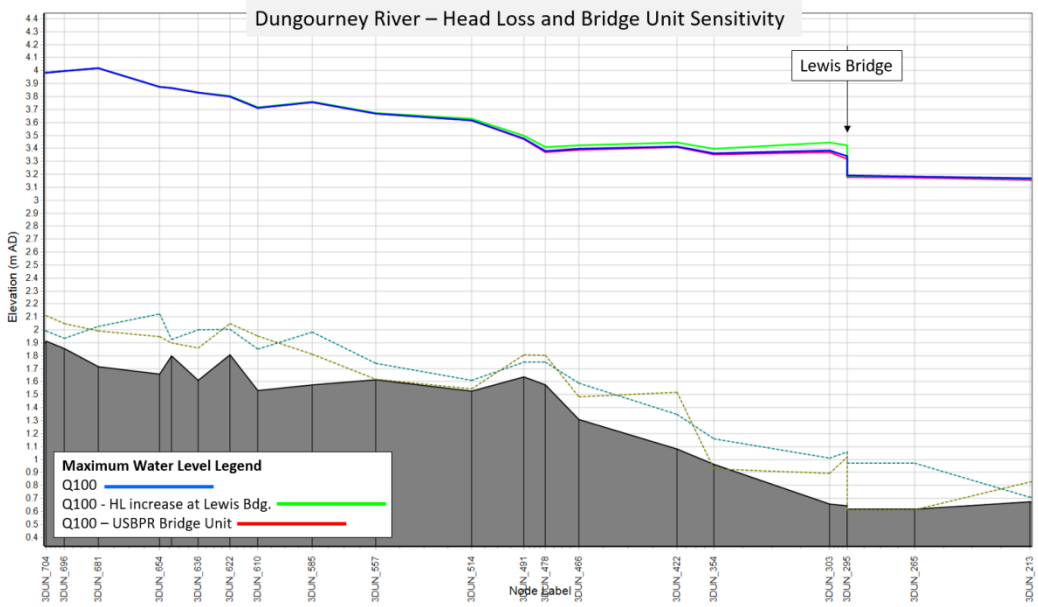


Figure 5-5 Increased head loss coefficient at Lewis Bridge / USBPR Bridge at Lewis

It can be seen that the increased head loss coefficient has a relatively minor impact on the maximum water levels upstream as the levels are increased by circa 65mm. This increased maximum water level does however lead to a marginally greater volume of water spilling from the tributary to the floodplain and hence increases flood risk upstream.

The findings of the sensitivity analysis at this location will be considered as part of the optioneering.

5.4 Bridge unit type

For the sensitivity analysis, the model was simulated with an alternative Bridge Unit for Lewis Bridge. The baseline model used an Arch Bridge while the sensitivity run used the USBPR unit. Figure 5-5 above presents a longitudinal plot of the Q100 maximum water levels along the relevant reach of the tributary upstream of the bridge for both the baseline scenario and the USBPR sensitivity. It can be seen from the plot that the increased head loss coefficient has a very insignificant impact on the maximum water levels upstream. There is therefore no material change in the flood risk with the alternative bridge unit.

5.5 Small Catchment Hydrology

The baseline hydrological estimation utilised the IH124 method to derive design flows for small catchments. The Flood Studies Supplementary Report (FSSR) No 16 method was however also used to derive an alternative set of design flows for small catchments. The results from the hydrology report are reproduced in Table 5-2. It can be seen from the table that the FSSR16 method produced reduced design Q100 flows at two of the small catchment HEP's:

- Bal1 (Ballinacurra Catchment)
- HAG2 (Tributary of the Dungourney).
- These HEPs were therefore not assessed as part of the small catchment sensitivity.

Table 5-2 Small Catchment sensitivity

HEP	Q100 - IH124	Q100 - FSSR 16	Delta
Bal1	2.6	1.8	-0.8
EL1	8.0	15.07	+7.07
GL1	12.7	22.3	+9.6
HAG2	9.3	7.4	-1.9
OAT1	4.9	5.4	+0.5
OAT3	9.2	11.1	+1.9
OW5	19.5	34.6	+15.1

The hydraulic model was rerun with the updated FSSR16 flows set as the inflow boundary condition for the three small catchments that experience a significant increase in flow using the FSSR16 method: EL1, GL1, and OW5.

Figure 5-6 presents the simulated Q100 flood extent for both the baseline and sensitivity scenario for the critical reach. Figure 5-7 presents the increase in flood depth associated with the sensitivity.

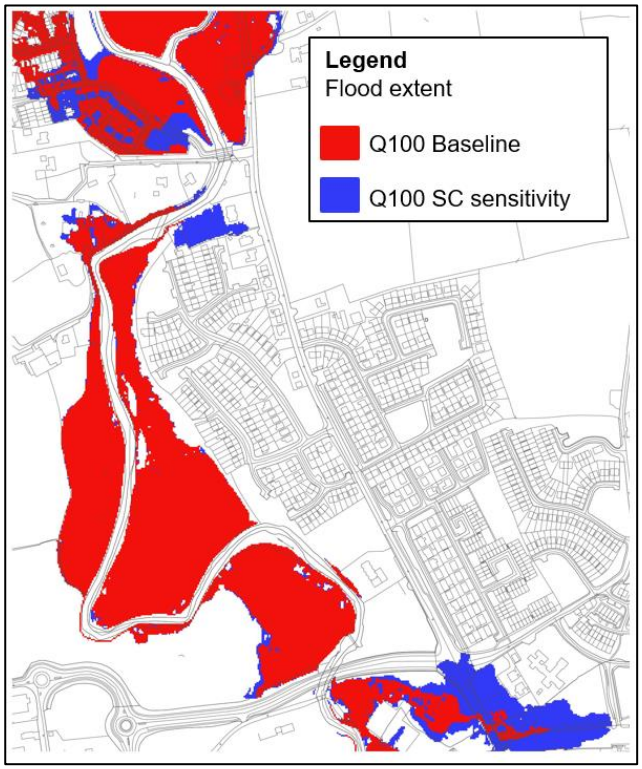


Figure 5-6 Small Catchment Hydrology Sensitivity – flood extent

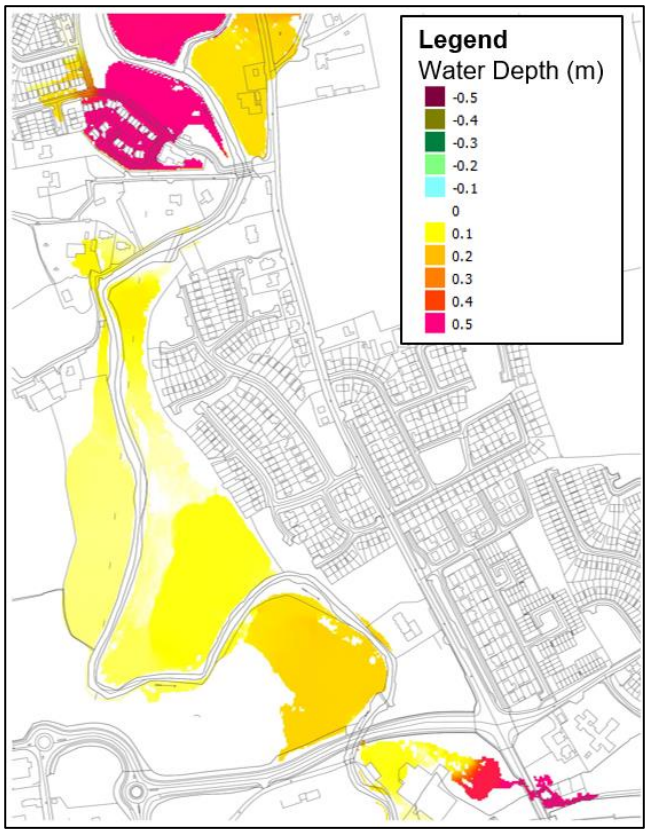


Figure 5-7 Small Catchment Hydrology Sensitivity – increase in flood depth

The higher design flows on the small catchments results in greater flood depths and extents upstream of Moore's Bridge and downstream of the Northern Relief Road in the Railway Cottages.

Upstream of Moore's Bridge, the increase in flood depth for the event is circa 0.5m. Downstream of the NRR the increase is circa 0.5m in the Railway cottages. The small catchment sensitivity therefore results in an increased level of flood risk along the Owenacurra. This will be considered as part of the Optioneering of the scheme.

It is noted that that the Q100 on the Water Rock Stream also experiences a relatively minor uplift in flow using the FSSR 16 method (OAT 1 and OAT 3 in Table 5-2) which will increase flood risk in the catchment. This will also be assessed as part of the Optioneering.

5.6 2D model grid resolution

The grid spacing of the 2D component of the model was increased from 2m to 4m in order to assess the sensitivity of the model and also to accommodate the climate change scenario runs as discussed later in Section 9.1.

In order to assess the sensitivity of the results, the results from the baseline 2m grid model were compared with the results of the coarser 4m grid model for the 1% AEP event. The comparison in the modelled maximum flood extents is presented in Figure 5-8 while the peak water level comparison is presented in Figure 5-9.

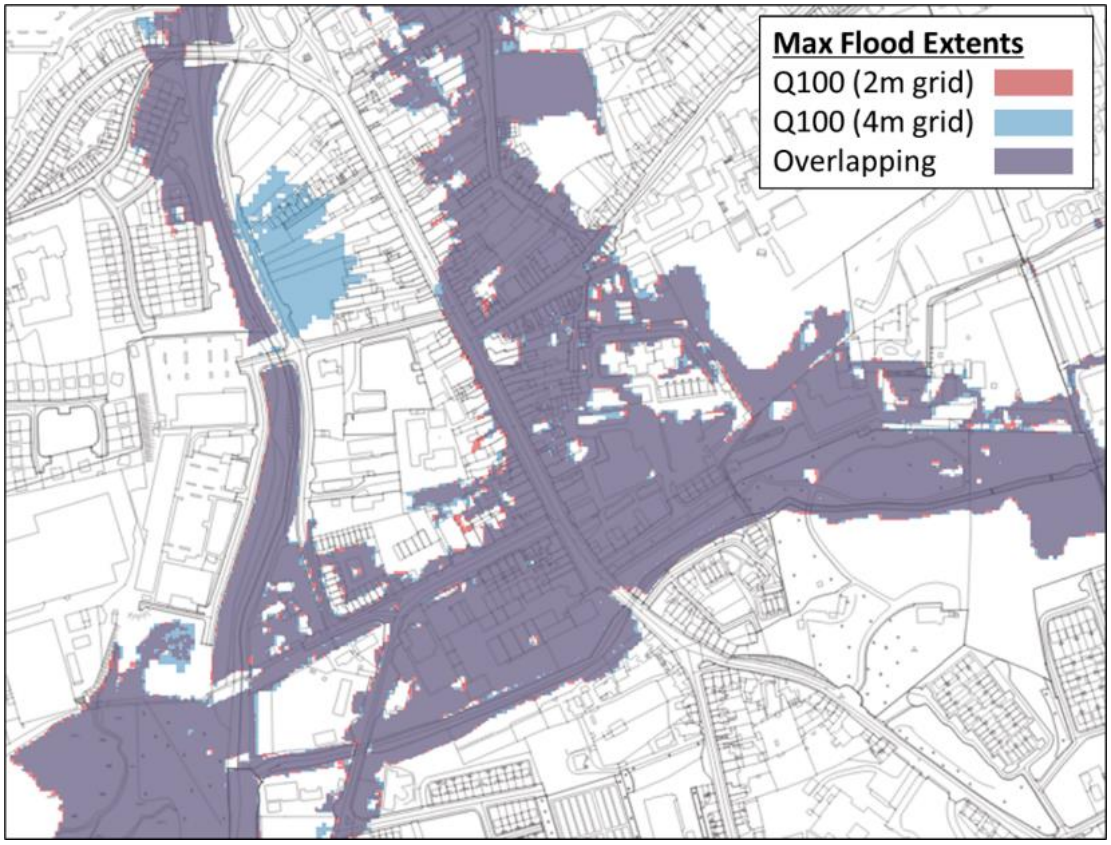


Figure 5-8 Fluvial AEP maximum flood extent – 2m vs 4m grid comparison

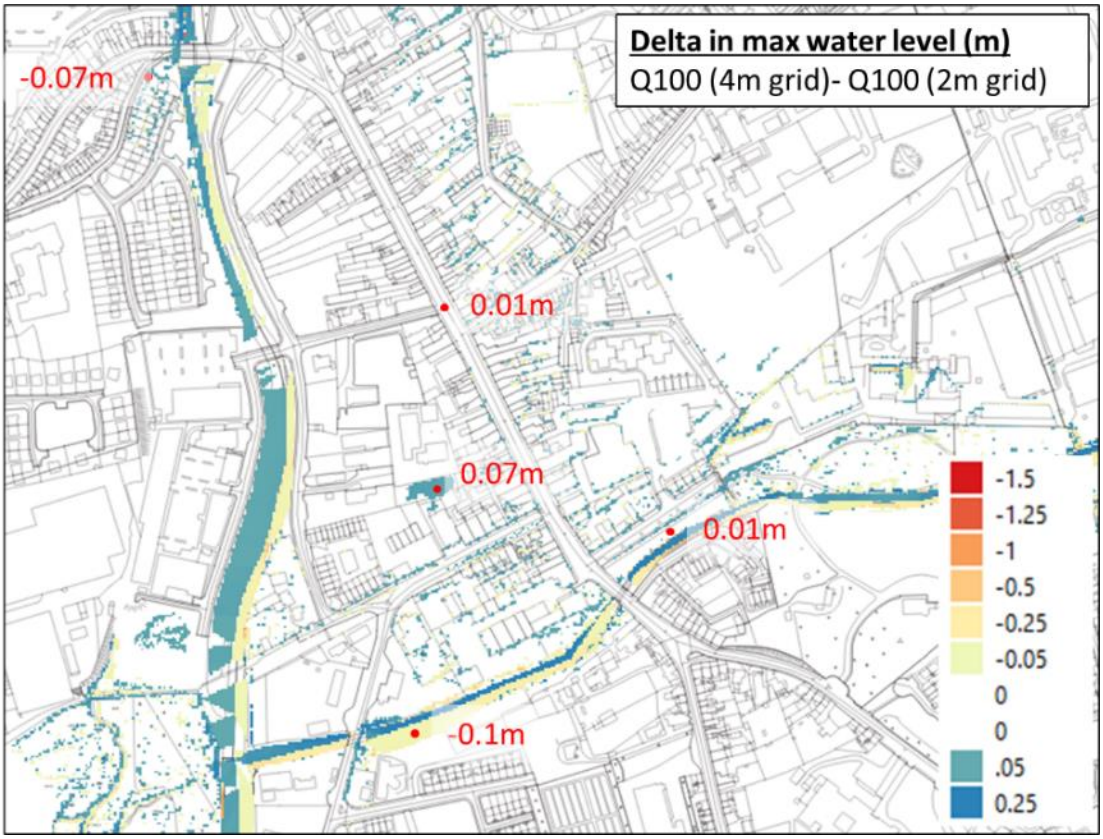


Figure 5-9 1% Fluvial AEP max water level (4m – 2m grid) comparison

It can be seen from the plots that the modelled flood extents are largely the same for both models. The only notable difference is a greater predicted flood extent on the right bank of the Owenacurra in the vicinity of Thomas Street and upstream of the Lidl bridge.

The Q100 event is bank full at this location and the water is kept in-bank for the 2D grid model due to the 1D/2D spill elevation being marginally set above the water level. The spill level of the 1D/2D interface for the 4m grid model is however set marginally lower due to the coarser resolution of the grid which leads to water from the river exiting the channel and flowing overland to the low-lying area of Thomas Street.

From the maximum peak water level comparison (Figure 5-9) it can be seen that the differences in water levels across the study area are minor (<0.07m). The only noticeable differences are along the 1D/2D interface where differences of circa 100mm are evident.

It can therefore be concluded that the difference between the 4m grid and the 2m grid modelled results are not significant.

5.7 Summary of Sensitivity Analysis

As detailed in Section 5.2 to 5.6 above, the sensitivity analysis carried out using the Q100 pre-Storm Babet flows does not suggest that changes to the model assumptions made would significantly alter the water levels. It can therefore be determined that if the same analysis were to be repeated using the updated Q100 design flows (post- Babet) there would be material change to these findings and that the conclusion drawn above remain valid.

6. October 2023 flood event validation

6.1 Overview

On the 18th of October 2023, following Storm Babet, Midleton experienced its most significant fluvial flood event in living memory.

The existing scenario FMP/Tuflow model of Midleton was used to simulate Storm Babet using the estimated peak flow of 103.65m^3/s and the lower bound estimate of 88m^3/s.

The results from the model were compared with the observed extents and depths to assess if the estimated peak flow is a realistic representation of the flow which occurred during the event. The resulting inundation and water levels in Midleton in the model were found to be more similar to observed patterns from the higher peak flow estimate.

It is noted that the mechanisms of flooding observed during the event were well replicated within the hydraulic model.

6.2 Relevant model assumptions

A number of assumptions were adopted as part of the development and running of the hydraulic model for the Storm Babet validation and are listed as:

- The peak flows of all water courses feeding into the model were scaled based on the findings of the gauge review for the Owenacurra. This was considered appropriate as the catchments are in close proximity such that the hydrological forcing on all the catchments would have been very similar during the event;
- The hydrograph shape at Ballyedmond was use as the hydrograph shape for all the other catchments; While it is likely that the hydrograph shapes of the smaller catchments would have a shorter time to peak, the differences are not significant enough to affect the accuracy of the model and impact the results;
- The timing of the peaks of the inflow hydrographs were set such that they would align at the confluences downstream. This may be a conservative assumption as it was reported that the Dungourney may have peaked at circa 17:00hrs through The Babys Walk circa 3 hours after it peaked on the Owenacurra;
- The peak on the Owenacurra coincided with low tide. The recorded tidal data was used as the downstream boundary of the model. It is noted that the tidal gauge in the estuary recorded a water level of circa -0.1mOD at the time of the fluvial peak of the Owenacurra;
- All the walls were removed from the model as per the assumption made for the existing scenario 1 in 100-year model runs. While a number of walls in the floodplain remained intact during the Babet event, a number of walls collapsed during the event, partially validating previous assumptions made and as set out in Section 3.4.5. This assumption needs to be considered when comparing the new Storm Babet model extents against the observed Storm Babet extents given that some walls omitted from the model would have diverted flood water during the event;
- Additionally, it is noted that all of the wall collapses occurred after the flood had been in progress for a number of hours and hence would have influenced the overland flow routes up to the moment in time at which the collapse occurred;
- The Storm Babet model run assumed zero blockage at all the structures. It was evident from post event site visits however that blockages occurred at Moore's Bridge during the event such that this also needs to be considered when comparing model results to the observed extents;
- Where surveyed Finished Floor Levels (FFLs) were not available, the floor level was set equal to the averaged lidar data across the footprint of the building plus a threshold allowance of 150mm. While in most cases this is an accurate assumption, for a small number of properties it may lead to a slight over- or underestimation of the FFLs in the model which in turn may impact on the overland flow routes. This also needs to be considered when comparing the results of the model to the observed flood extents.

6.3 Assessment of flood extents– Model vs Observed

Flood extent mapping of the (1) observed Storm Babet flood extent, and (2) the modelled Storm Babet flood extent is presented in the following sections for the key areas of the study:

- Tir Cluain/ Moore's Bridge
- Town Centre- North including Willowbank and Millbrook
- Town Centre- South
- Upper Dungourney- East of the IDL site

The figures below show the flood extent comparison between the modelled Babet event (103.65m^3/s peak flow) and the observed flood extents during the flood event.

6.3.1 Validation at Tir Cluain/ Moore's Bridge

There were a number of local differences between the modelled flood extent and the observed extent as presented in Figure 6-1

In some places, the model over-predicted flooding extent, for example where the model assumed local walls were absent, but in reality, some boundary walls remained intact during the event and temporarily restricted or redirected floodwater.

In other places, the observed flooding was greater than the model suggested, especially where walls failed, where Moore's Bridge was partially blocked due to its collapse, or where local flow routes developed that were not fully represented in the model due to the model resolution e.g. curbs.

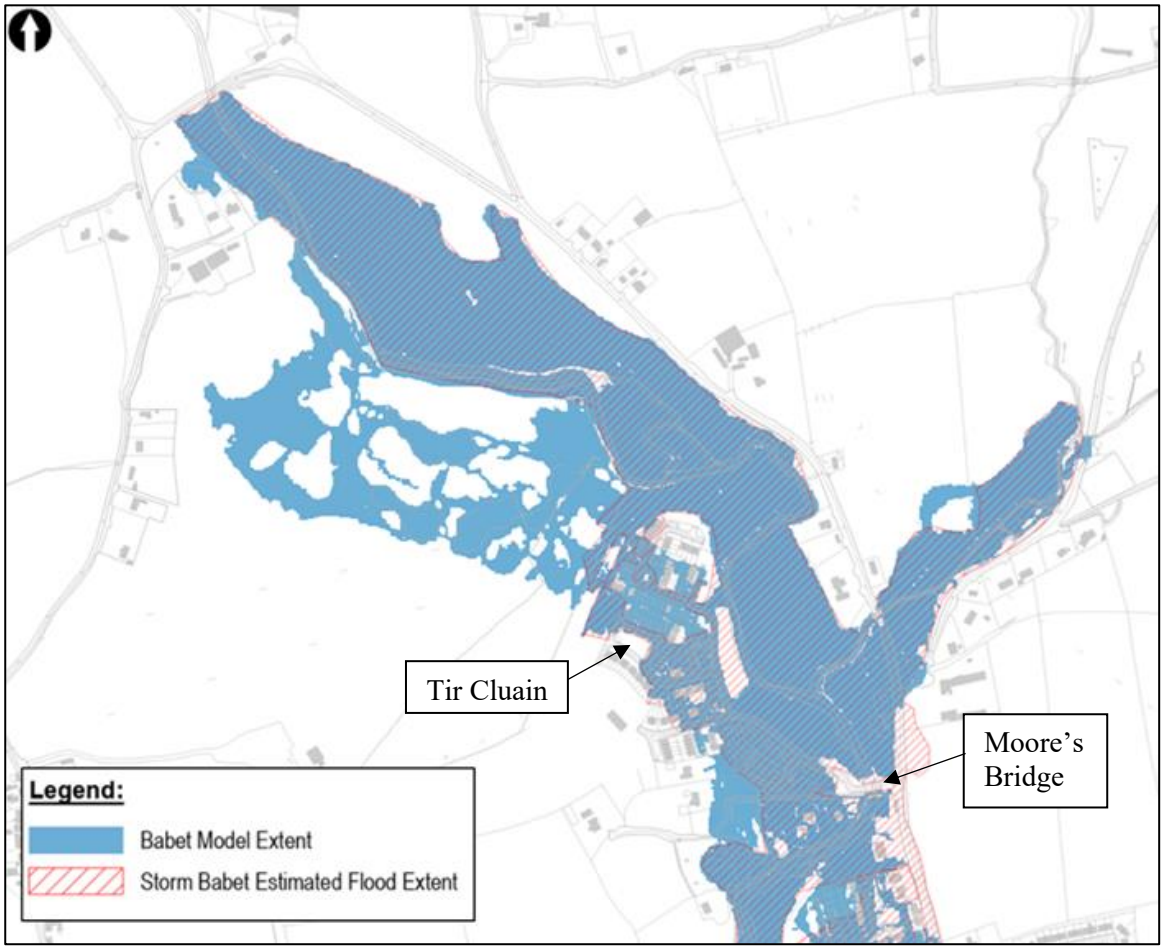


Figure 6-1 Babet storm flood extent comparison recorded vs model Tir Cluain/ Moore's Bridge

A recurring theme is that wrack marks and observed levels were sometimes inconsistent, so there is some uncertainty in recorded local water levels, but we can conclude that the model generally accurately simulates the relevant flood mechanisms, while local structures, wall failures, blockage, and timing effects caused the biggest differences between the modelled and observed flood extents.

Overall, the validation model captured the main flood pathways and extents observed during Storm Babet reasonably well, with the same broad mechanisms seen in the hydraulic model also observed during the event.

In summary, the flood extent produced by the model was a good representation of what was observed in the Tir Cluain/ Moore's Bridge area.

6.3.2 Validation of Town Centre- North (including Willowbank and Millbrook)

At Willowbank, whilst the modelled mechanism was broadly accurate, wall failures (discussed in Section 3.4.5) allowed floodwater into the estate in slightly different locations, while other walls within the estate that remained standing changed how water moved locally. The modelled extents were generally slightly greater than observed.

The flow path observed on Mill Road was not replicated to the same extent in the model. This is likely due to Moore's Bridge being significantly blocked and also having partially collapsed during the event. The blockage would have greatly elevated water levels upstream of the bridge and very likely led to water spilling out onto Mill Road, leading to the observed mechanism and extents. Blockage of the bridge was not assessed in detail in the validation model run.

At the Northern Relief Road / rail crossing, the model captured the main routes and extents including flooding of the Northern Relief Road west of the bridge.

At Millbrook/ Mill Road Lower, the modelled extents was a good match to the observed extents. Localised differences can be attributed to the impact that the property walls/ buildings had on flow paths.

In the Millrace Apartments / Market Square area, the modelled mechanisms and flood extent was a very good match to what was observed on the day of the event.

Overall, there was a strong correlation between the hydraulic model and the witnessed flood extent and mechanisms in this area, as shown in Figure 6-3 below.

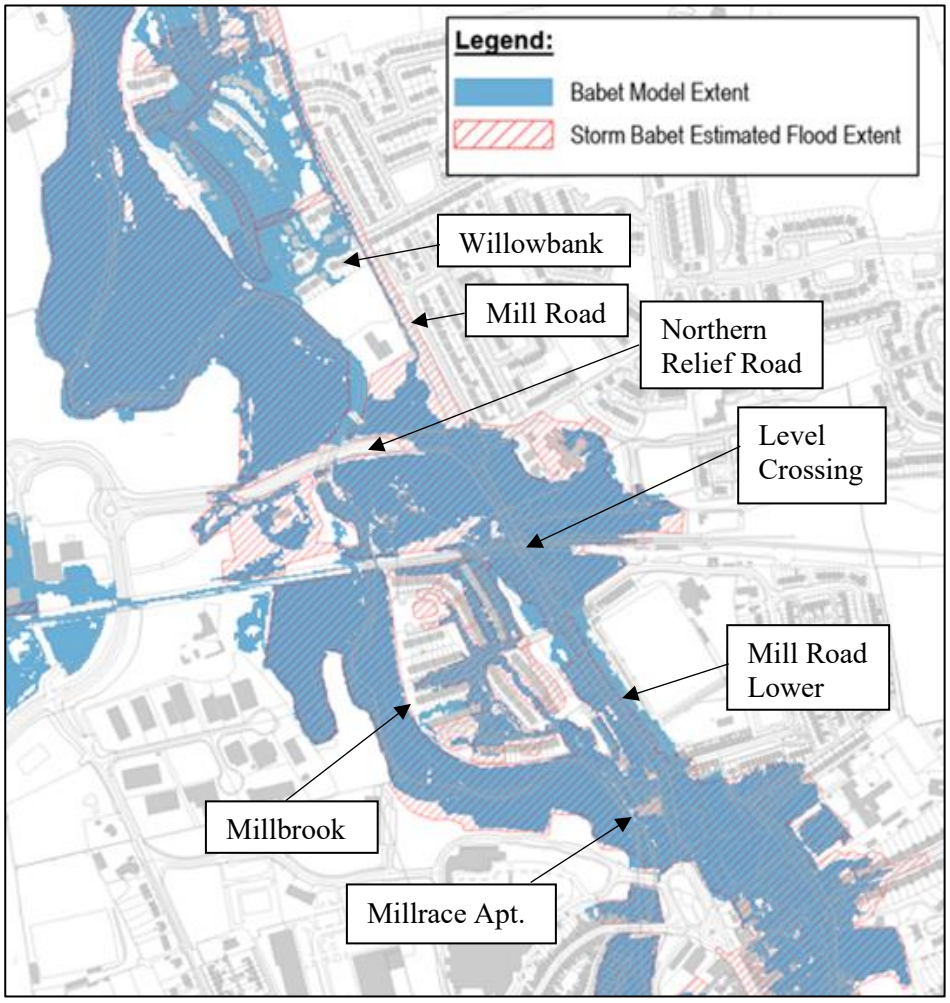


Figure 6-2 Babet storm flood extent comparison recorded vs model Town Centre North

6.3.3 Validation of Town Centre- South

In the area from Cork Road Bridge to Lidl Bridge, the model and observations were generally in good agreement.

In other parts of the town centre—especially Upper Main Street / Drury's Avenue, the observed flooding was more severe than the model suggested. This is likely due to the influence of structures/ walls on localised flow paths/extents.

In the Baby's Walk / Lower Main Street area, the hydraulic model flood extent and mechanisms matched well with what was witnessed on the day of the event.

In Beechwood / Oakwood, south of the Dungourney, the observed flood extent was greater than the model results. This is likely due to an existing low point in the embankment between the GAA Grounds and Beechwood not being captured in the LIDAR data (A topographical survey was carried out after the flood event which confirmed there was a local low point).

Overall, the model captured the main flood mechanisms across Town Centre- South reasonably well, and in most locations the same flow paths/ mechanisms seen in the hydraulic model were also observed during Storm Babet as shown in Figure 6-3.

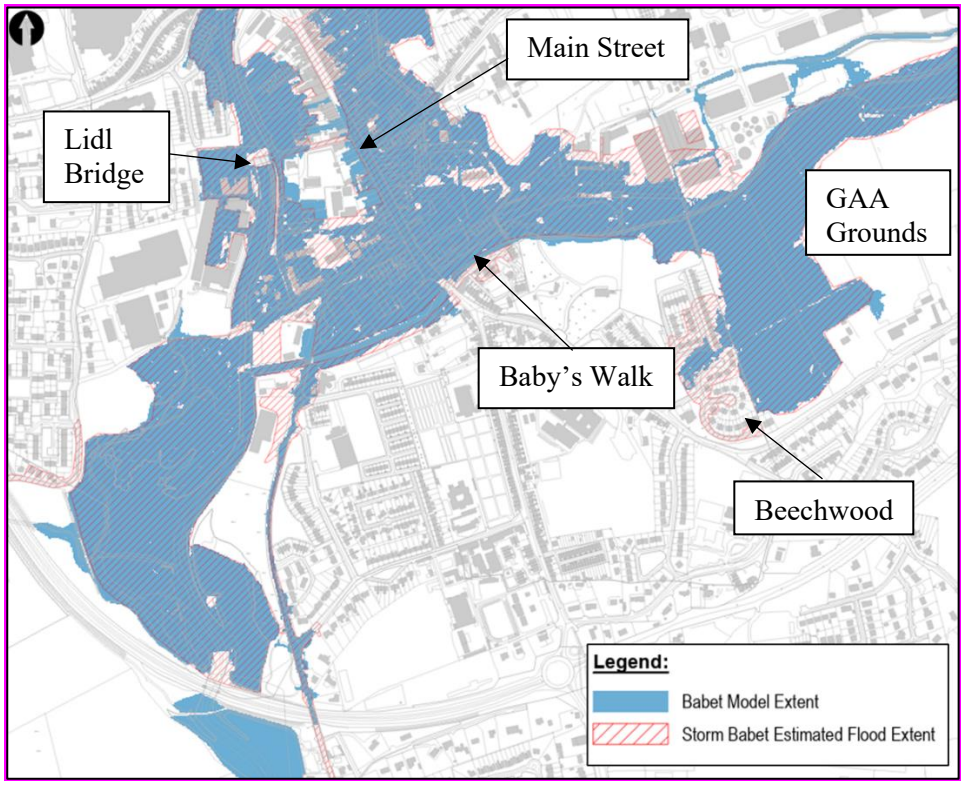


Figure 6-3 Babet storm flood extent comparison recorded vs model Town Centre South

6.3.4 Validation of Upper Dungourney- East of the IDL site

The modelled flood extents in the Upper Dungourney area aligned well with those observed during the event. There were no reports of properties flooding from groundwater/ fluvial sources in this area during Storm Babet. The Green Way, Midleton Rugby Club and agricultural lands to the East of the IDL Site were flooded. The comparison of the estimated flood extent from Storm Babet vs the modelled extent is presented in Figure 6-4. It is noted that the Storm Babet estimated flood extent was informed using drone imagery which captured the area immediately north and east of the IDL site. While it is likely the floodplains further east, along the Dungourney, also flooded during Storm Babet and would align with the Babet model extent, the estimated extent in this area has not been verified as the focus was on built up/inhabited zones.

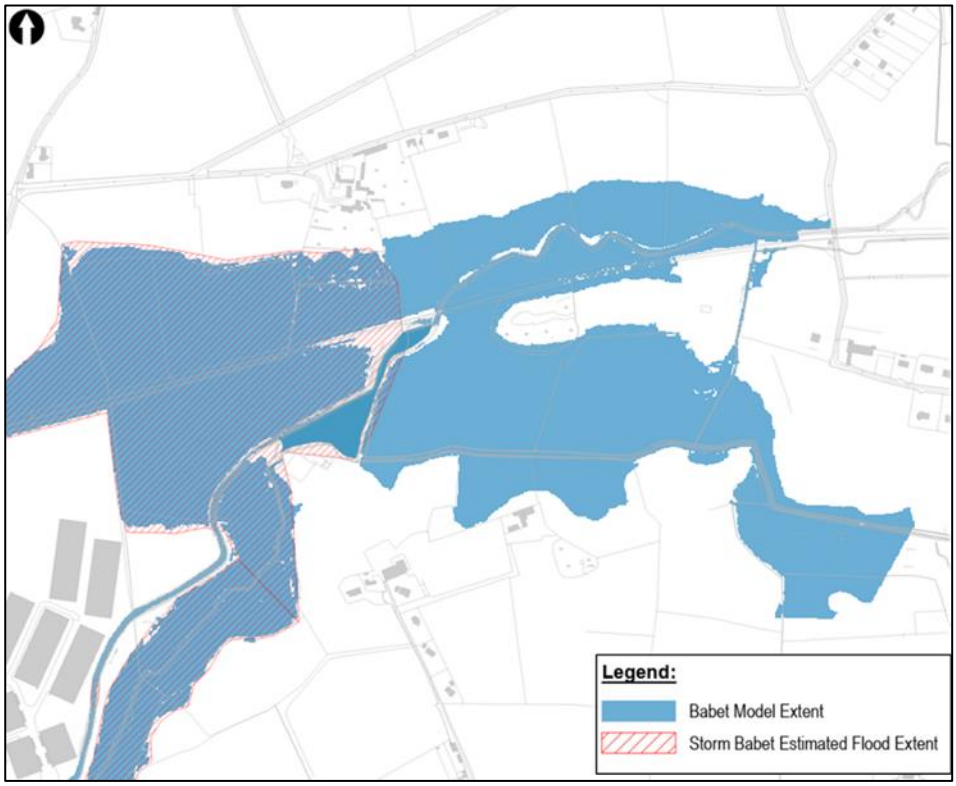


Figure 6-4 Babet storm flood extent comparison recorded vs model Dungourney Upstream

6.4 Assessment of water levels– Model vs Observed

A number of observed and surveyed wrack marks across Midleton have been compared against the modelled Storm Babet maximum water levels in Table 6-1. Positive values denote that the wrack mark is at a higher elevation than the model and negative values indicate that the wrack mark is lower than the model.

Table 6-1 Comparison of Wrack Marks vs Max Modelled Water Levels (mOD)

Location	Wrack Mark (mOD)	Modelled Water Levels (mOD)	Delta (mm)
Rear of Tir Cluain	16.81	16.82	-10
Clohessy's Yard	15.04	15.03	10
Right Bank immediately Downstream of Moore's Bridge	13.83	13.16	670
Right Bank along Moore's Lane	12.56	12.46	100
Willowbank Court	11.5	11.47	30
Entrance to Willowbank (Mill Road)	11.06	10.9	160
Owenacurra Millrace Offtake	9.45	9.13	320
Bridge Downstream of NRR Bridge	9.1	8.36	740
Level Crossing on Upper Mill Road	8.16	8.06	100
MyPlace	5.5	5.4	100
Midleton Garda Station	5.03	5.03	0
The Woodlands	4.55	4.78	-230
Thomas Street	4.12	3.84	280

Location	Wrack Mark (mOD)	Modelled Water Levels (mOD)	Delta (mm)
Water's Edge- Lidl	3.79	3.77	20
An Stor- Drury's Avenue	5.15	4.44	710
O'Farrell's Funeral Home	3.22	2.98	240
Junction of Main Street and Broderick Street	3.62	3.39	230
Three Store- Lower Main Street/ Distillery Walk	3.79	3.39	400
Babys Walk directly upstream of Lewis Bridge	3.6	3.42	180
Downstream of Lewis Bridge	3.6	3.3	300
Bailick Road (Uisce Éireann)	3.2	2.97	230
Bailick Road (Chadwicks)	3	2.71	290
Beechwood Drive	4.78	4.05	730

From an inspection of the differences, it is evident that there is a reasonably good match between the wrack marks and the hydraulic model across most of the study area. Seven of the 22 locations have a delta of less than 100mm which is deemed to be a very good match. At four of the locations, the delta is in excess of 400mm. The differences at these four locations is discussed further below:

- Right Bank immediately Downstream of Moore's Bridge (Delta +670mm)
  - As discussed with respect to the model assumptions, the blockage/ failure of Moore's Bridge during the Storm Babet Flood event was not included in the hydraulic model run, resulting in the model underestimating the water levels locally. It is noted that further downstream on Moore's Lane there is a very good match between the wrack marks and the modelled results i.e. the delta is circa 100mm.
- Bridge Downstream of NRR Bridge (Delta +740mm)
  - As discussed with respect to the model assumptions, the blockage of bridges during the Storm Babet Flood event was not included in the hydraulic model, resulting in the model underestimating the water levels locally. This bridge was surcharged during the flood event.
- An Stor - Drury's Avenue (Delta +710mm)
  - This wrack mark is located circa 250m east of the Owenacurra River and is therefore at the outer extent of the floodplain. The discrepancy between the observed and modelled water level can therefore be attributed to the complexity of the overland flow path between the river and this location and its representation in the model including the assumed floodplain Manning's values, representation of buildings and their impact on storage, inclusion/exclusion of garden and/or boundary walls, the impact of the drainage system.
- Beechwood Drive (Delta +730mm)
  - Water levels in the Beechwood Drive and Oakwood Lodge Estates are underestimated in the model by circa 730mm. There are two reasons for this:
  - The existing stone wall running between The People's Park and The Baby Walk and which is aligned perpendicular to the Dungourney has been excluded from the model. Given that the wall did not fail during Storm Babet, it would have exerted a significant afflux on the water levels upstream in the People's Park and on the GAA pitches during the event and this was not captured by the model. Hence the maximum water levels upstream would have been underestimated by the model.

– The peak of the fluvial event on the Dungourney may have occurred later than what has been assumed in the model at a time closer to high tide. This may also have contributed to the model underpredicting water levels on the reach.

6.5 Water Rock

The Water Rock catchment was ungauged during the Storm Babet flood event. Accordingly, there was a lack of hydrology data available to accurately and robustly calibrate the model in these areas.

6.6 Ballinacurra

On the morning of the Storm Event the high tide was recorded at 7.20am. The hightide resulted in tide locking of the culvert to the estuary, this compounded with heavy rainfall resulted in a channel full event. The Ballinacurra Stream did not break its banks as water stayed in channel upstream of the culvert inlet and at Kearney's Cross. There was some surface water flooding in the Main Street but properties had Individual Property Protection in place. There were no reports of properties flooding in this area.

6.7 Conclusions of the 2023 Event hydraulic modelling validation

The already calibrated, validated and sensitivity tested model was used to simulate the Storm Babet, October 2023 event. As detailed in the assumptions, Section 6.2, all walls in the study area were excluded from the model to inform the 2023 validation event. In general, the model was deemed to perform well based on comparison of observed/recorded flood extents and levels with the hydraulic model results.

Following Storm Babet, the assumption on the inclusion/exclusion of the existing walls in the existing scenario models as proposed in Section 3.4.5 of this report are deemed to remain appropriate.

6.8 Model Amendments to reflect Post Babet replacement of Moore's Bridge

Moore's Bridge was significantly blocked and partially collapsed during Storm Babet. Figure 6-5presents an image of the bridge from the day after the event which shows the collapsed span of the bridge on its Western side.



Figure 6-5 Moore's Bridge Collapse

In October 2024, post Storm Babet, the bridge was rebuilt with the new soffit raised to 14.058mOD. (Soffit of old bridge was 13.46mOD) The piers within the channel were also removed, to reduce blockage risk and improve the conveyance. Figure 6-6 depicts Moore's bridge after reconstruction.



Figure 6-6 Moore's bridge after reconstruction

It is evident that the new bridge design differs significantly from the design of the bridge prior to Babet.

The recorded data for the calibration/validation events during 2015, 2018 and 2023 was gathered while the old bridge design was in place and therefore was appropriate for use in these calibration events.

However, in finalising this Hydraulics Report and Existing Flood Extents mapping, it was necessary to update the existing case model to reflect the current bridge. This has been completed utilising as-built survey data for the new bridge.

As there is no flood event data to perform a further calibration with the new bridge, industry standard parameters have been applied to the bridge. Due to the localised impact of the bridge, it is not expected that the uncertainty of these values will have a major impact on the model more widely.

The model used to inform the detailed design runs was also updated to reflect the new Moore's Bridge arrangement.

The existing scenario and climate change scenario design runs are discussed further in Section 8 and 0 respectively.

7. Hydrological Estimation and Hydrogeological assessment

7.1 Hydrological Estimation Undertaken as Part of the Study

A comprehensive hydrology report was completed and issued in 2018 as part of Stage 1 of the MRFS.

This report was subsequently updated in 2022 and documented analyses undertaken between 2018 and 2022.

Following Storm Babet in October 2023, a further review of the hydrology was recommended and subsequently undertaken to utilise the additional data from such an extreme event and improve our understanding of the catchment's response to such an extreme event. Following same, the flow estimations for the different return periods were recalculated for all the watercourses within the model.

The analysis utilised a number of hydrological estimation methods to establish a range of design flows at various points in the study area which has subsequently been used as inputs to the hydraulic modelling.

A summary of the hydrological estimation is provided in this chapter as well as a description of how the hydrology was anchored in the hydraulic models. For further information please refer to the Midleton FRS Hydrology Report.

7.1.1 Design flow estimation

A detailed hydrological analysis was undertaken to determine design flows for the Midleton FRS. The analysis applied a number of methods to establish a range of possible design flood flows at the various Hydrological Estimation Points (HEP) in the study area as shown in Figure 7-1 below.

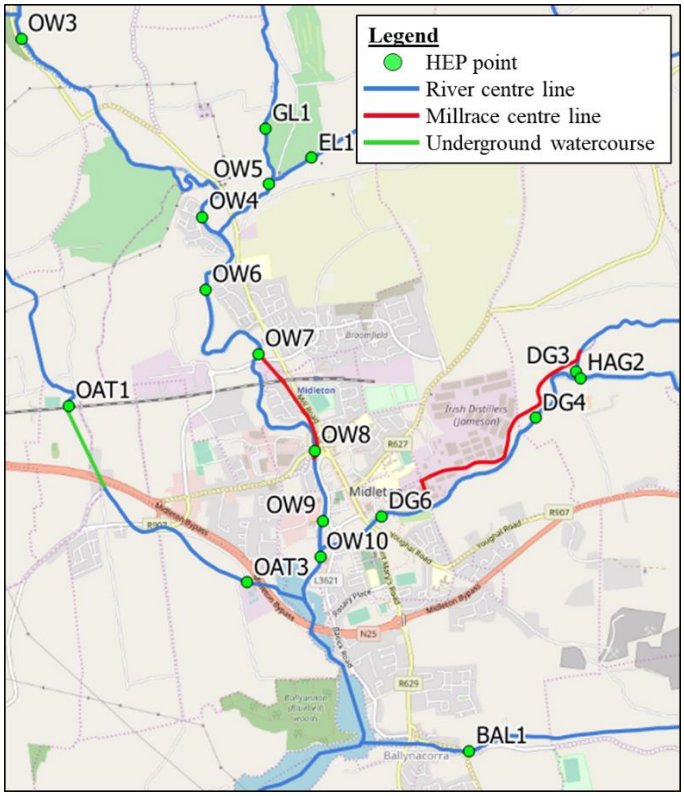


Figure 7-1 HEP points (© Open Street Map)

A set of index flood flow (Qmed) estimates were produced at the HEPs in the study area. Given that many catchments in the study area are small and predominantly ungauged, a range of methods were examined including FSU, FSR, FSR RR and IH124.

A rating review of the existing hydrometric gauge at Ballyedmond was undertaken. Annual maximum data for two missing years of data at the gauge was generated using the FSSR 16 method. The revised rating curve was then used to update the high flow series at the gauge. The updated flows were analysed and a final Qmed value of 24.81m^3/s was generated for the gauge.

It was deemed appropriate to adopt the IH124 68%ile Confidence Limit index flows for HEPs on small catchments and the FSU index flows for remaining HEPs as they are conservative and consistent with the other hydrological estimation methods. Allowance for uncertainty has been incorporated by including Factorial Standard Error (FSE) adjustments. The adopted index flows are presented in the following Table 7-1.

Table 7-1 Index Flows

HEP Location	Index Flow (m^3/s)	HEP Location	Index Flow (m^3/s)
BAL1*	0.99	OW3	26.13
DG3	13.56	OW4	26.32
DG4	15.07	OW5*	7.54
DG6	15.72	OW6	32.11
EL1*	3.10	OW7	32.22

HEP Location	Index Flow (m^3/s)	HEP Location	Index Flow (m^3/s)
GL1*	4.89	OW8	32.84
HAG2*	3.61	OW9	33.22
OAT1*	1.88	OW10	46.47
OAT3*	3.56	* HEP located on small catchment	

A flood frequency analysis was carried out to establish a study growth curve and in turn a set of design peak flows. The adopted growth curve was a compound growth curve produced using the Single Site Analysis at Ballyedmond for events up to the 25-year return period and the FSR RR method for the more extreme events. The study growth curve is presented in Table 7-2. The design flows for the HEPs are presented in Table 7-3.

Table 7-2 Study Growth Curve

Return period (years)	Study Growth Curve
2	1.04
5	1.53
10	1.91
25	2.45
50	2.91
100	3.23
200	3.72
1000	4.65

Table 7-3 Design Flows in m^3/s

HEP Location	Return Period (1in _ years)							
	2	5	10	25	50	100	200	1000
BAL1*	1.03	1.51	1.89	2.43	2.88	3.19	3.68	4.60
DG3	14.05	20.72	25.85	33.24	39.46	43.74	50.44	63.05
DG4	15.61	23.03	28.72	36.94	43.85	48.61	56.05	70.07
DG6	16.29	24.02	29.96	38.53	45.75	50.71	58.47	73.09
EL1*	3.21	4.74	5.91	7.60	9.02	10.00	11.53	14.41
GL1*	5.07	7.47	9.32	11.99	14.23	15.77	18.19	22.74
HAG2*	3.74	5.52	6.88	8.85	10.51	11.64	13.43	16.78
OAT1*	1.95	2.87	3.58	4.61	5.47	6.06	6.99	8.74
OAT3*	3.69	5.44	6.79	8.73	10.36	11.48	13.24	16.55
OW3	27.07	39.93	49.80	64.04	76.04	84.29	97.19	121.49

HEP Location	Return Period (1in _ years)							
	2	5	10	25	50	100	200	1000
OW4	27.27	40.22	50.17	64.51	76.59	84.90	97.90	122.38
OW5*	7.81	11.52	14.37	18.48	21.94	24.32	28.05	35.06
OW6	33.27	49.06	61.20	78.70	93.44	103.57	119.44	149.30
OW7	33.38	49.23	61.41	78.97	93.76	103.93	119.85	149.81
OW8	34.02	50.18	62.59	80.49	95.56	105.93	122.15	152.69
OW9	34.42	50.76	63.32	81.42	96.67	107.16	123.57	154.46
OW10	48.14	71.01	88.57	113.90	135.23	149.89	172.85	216.06
* HEP located on small catchment								

7.1.2 Hydrograph Shape Analysis

Two different methods were used to estimate the design hydrograph shape in the original 2018 analysis:

1. FSU hydrograph shape width analysis – this methodology estimates flood hydrographs in ungauged catchments by fitting a curve to a set of recorded flood hydrographs from hydrologically similar gauges;
2. The FSR rainfall-runoff method, or the unit hydrograph method – This is the traditional method of hydrograph generation and provides the shape and volume of a flood hydrograph. The unit hydrograph is derived from the catchment characteristics.

The FSU methodology was adopted as it provided a number of advantages in comparison to the FSR rainfall-runoff method including:

- The FSU method is based on river gauge recordings and utilises long term time series data from the Ballyedmond Gauge which therefore provides catchment specific hydrograph characteristics;
- The FSU method does not consider rainfall recordings and therefore bypasses any errors in rainfall recording and uncertainty in its conversion to a hydrograph shape;
- Both the Owenacurra and Dungourney River catchments are reasonably large and therefore similar to most catchments available for hydrograph width analysis as part of the FSU database.

As part the revision to the hydrology undertaken in 2024 post Storm Babet, the option to derive the design storm hydrograph directly from observed events at Ballyedmond was explored. The Archer et al. (2000)[9] generalised (standardised) hydrograph approach was used in which the duration of exceedance for given percentiles of event hydrograph flows are calculated for observed events. Ten high ranking AMAX events used analysed, including Storm Babet, and the design shape generated was the median across the events

After comparing the FSU, the FSR R-R and Archer methods to observed events in the AMAX record, it was evident that both the FSU and FSR Rainfall-runoff methods appear to be overestimating the hydrograph shape; therefore, the generalised (standardised) hydrograph based on observed events, including Storm Babet, has been now adopted for design. The standardised hydrograph is presented in Figure 7-2 Standardised Hydrograph adopted for the Scheme Figure 7-2.

[9] Archer D, Foster M, Faulkner D, Mawdsley H (2000) The synthesis of design flood hydrographs. In: Proceedings of flooding: risks and reactions CIWEM/ICE conference, London, pp 45–57

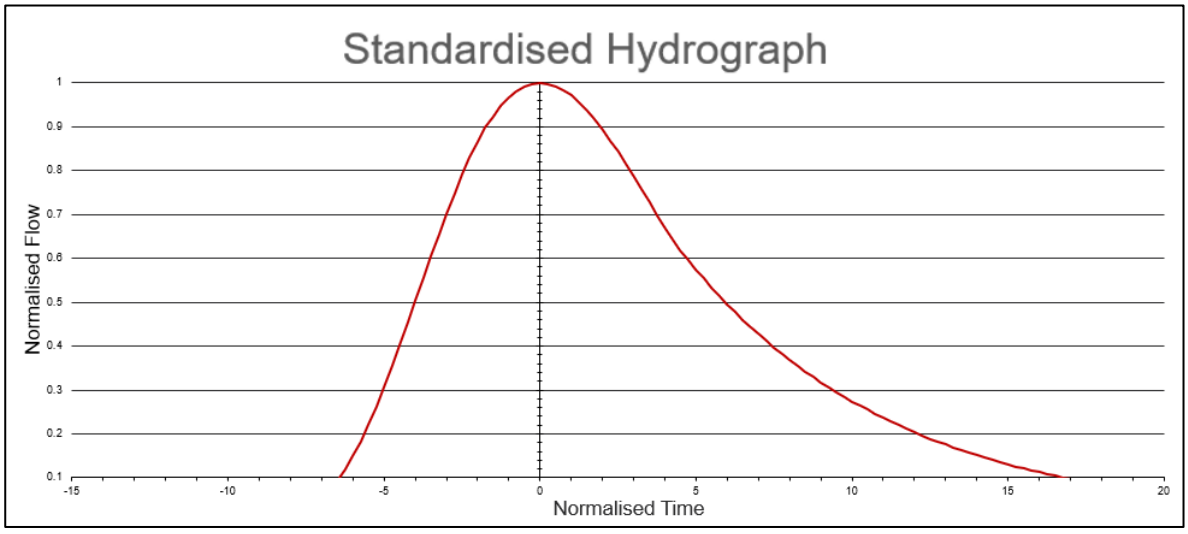


Figure 7-2 Standardised Hydrograph adopted for the Scheme

The reader is referred to the hydrology report for a detailed analysis of the hydrograph shape analysis.

7.1.3 Small Catchment Hydrology Sensitivity

We undertook a sensitivity analysis of the small catchment hydrology by considering an alternative and more conservative hydrological estimation method to the original IH124 method used. We utilised the FSSR 16 rainfall runoff method to provide a second estimate of the design peak flows for the small catchments of the study area. The impact of adopting these more conservative inflows as the design flows for the small catchments is considered as part of a hydraulic modelling sensitivity analysis which is presented in Section 5.5of this report.

7.2 Integrating the Design Flows into the Hydraulic Model

7.2.1 Insertion of the Hydrological Estimation Points

The design flows estimated at the upstream boundary HEP points were not used as the upstream boundary to the model. Instead, the higher design flow from the HEP at the downstream end of the various reaches was used as the inflow boundary. This conservative approach was used in order to ensure the model does not underestimate the design flow anywhere along the reach. This approach is illustrated graphically for the upstream boundary of the Owenacurra River in Figure 7-3.

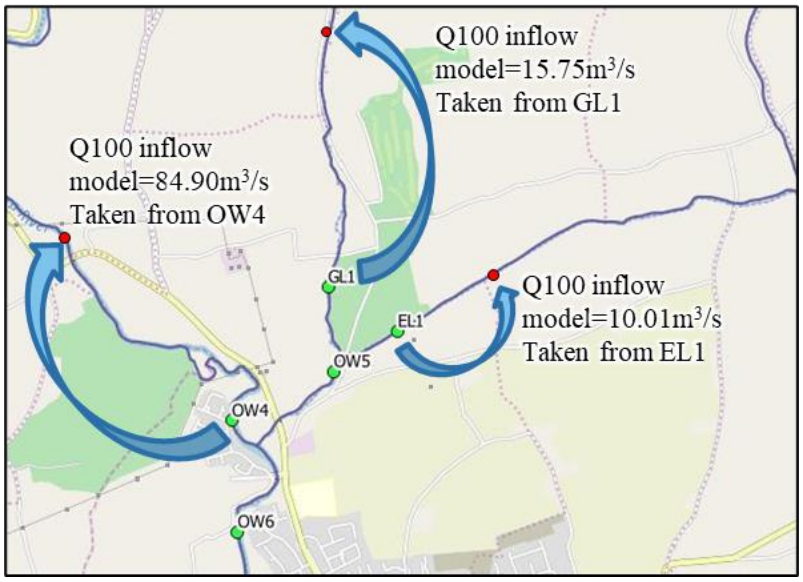


Figure 7-3 Inclusion of HEPs in the hydraulic model (© Open Street Map)

It can be seen from the figure that the Q100 design flow for OW4 is 84.9m3/s. This flow estimate was used as the Q100 inflow for HEP OW3 which is located upstream.

7.2.2 Anchoring of design flows in the Hydraulic Model

In hydrological estimation, the sum of the design flows from two or more sub catchments can exceed the estimated design flow for the whole catchment due to differences in the averaging of catchment characteristic over catchment areas. This can lead to an overly conservative modelling of flows downstream of confluences and consequently an overestimate of design water levels in a reach.

As can be seen from

Table 7-3, the design flows in the study area from one or more sub catchments are generally within 5.5% of the design flow for the whole catchment.

To further illustrate this, Figure 7-4 presents the 1% AEP design flows at the Owenacurra/Dungourney confluence.

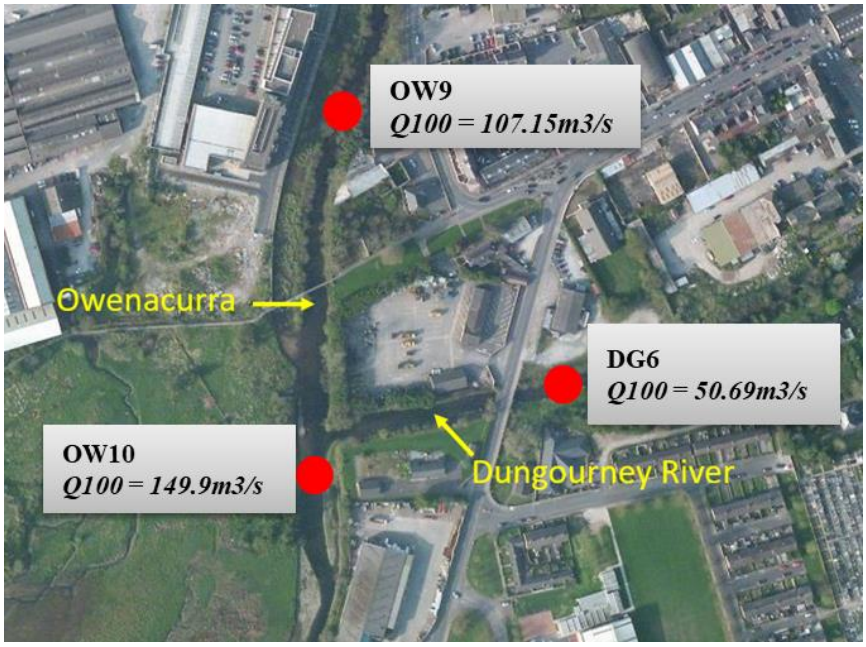


Figure 7-4 Design Q100 flows at the Dungourney/Owenacurra confluence

It can be seen that the Q100 downstream of the confluence is 149.9m^3/s while the sum of flows upstream of the confluence is 157.84m^3/s (107.15m^3/s + 50.69m3/s) which represents an increase of 4.8%. Without any adjustment of the design flows on either of the reaches in the hydraulic model, the flow downstream of the confluence would therefore be overestimated by 5.3%.

We have adopted a conservative approach in our model set up by not making any adjustment to the design flows upstream of confluences to account for the minor overestimate of the flow downstream of the confluence. In the example presented above, our design flow will therefore be overestimated by 5.3%. This assumption will be re-evaluated as part of the optioneering stage to ensure its conservatism does not result in flood relief measures that fail to meet any economic, social or environmental criteria.

7.2.3 Low Flows in the Hydrograph

A minimum flow of circa 1m^3/s was applied to the hydrographs to ensure hydraulic model stability at the start of each model run.

7.2.4 Coincidence of the Design Hydrograph Peaks

It was assumed that the design flow peaks of all sub-catchments occur simultaneously. We note that this is a conservative approach as the flashier sub-catchments are likely to peak before the main Owenacurra and Dungourney rivers.

7.2.5 Downstream Tidal Boundary Conditions

The downstream boundary condition of the hydraulic model for both the calibration and design runs is the tidal signal in the Owenacurra Estuary. As noted in the project hydrology report, the calibration model runs utilised recorded tidal water levels from the Port of Cork Gauge at Cobh as the downstream boundary for the calibration runs.

The tidal boundary for the design runs has been derived using two datasets:

- The extreme value tidal water level analysis undertaken as part of the Lee CFRAM Study – this data has been used to set peak water levels in the outer harbour;
- Recorded tidal water levels from the Cobh tidal gauge - this data has been used to define the shape of the tidal curve in the outer harbour;

The tidal signal in the outer harbour area however is not equivalent to the tidal signal in the Owenacurra estuary (where the downstream boundary of our 1D/2D model is located) due to the hydrodynamic and metrological effects associated with the propagation of tide through the harbour. As noted in the hydrology report, a two-dimensional MIKE 21 model of Cork Harbour has therefore been developed and used to calculate the design water levels in the Owenacurra estuary.

The peak tidal water levels for the Owenacurra estuary are presented in Table 7-4. The reader is referred to the project hydrology report for further detail on the design tidal water levels.

Table 7-4 Peak Tidal Water Levels

Design Event (AEP)	Peak Tidal Levels in Owenacurra Estuary (mOD Malin OSGM 15) Water levels include a 150mm seiche allowance
50%	2.64
20%	2.74
10%	2.81
4%	2.88
2%	3.00
1%	3.07
0.5%	3.09
0.1%	3.30

7.2.6 Urban Drainage Network

Arup reviewed the available existing data and reports on the Urban Drainage Network in Midleton which included a detailed report by Byrne Looby Partners from 2015.[10] As part of their study, BLP assessed the performance of the existing surface water drainage network along Main Street, Youghal Road and St. Mary's Road in Midleton and proposed a series of engineering measures to improve the performance.

As part of this project, Arup has developed a new hydraulic model of the surface water drainage network in Midleton using Microdrainage software. CCTV and Manhole survey data collected as part of the Midleton DAP[11] was incorporated into the model in order to infill the data gaps as previously identified by BLP. The key finding of the drainage modelling is that much of the existing drainage network in the town centre is undersized and consequently, some low-lying areas of the town are at risk of surface water flooding for low period events.

Pluvial flood depths for the existing scenario have been used to derive pluvial flood damages as part of the optioneering assessment. Optioneering to address the pluvial flood risk is discussed in the Options report.

7.2.7 Fluvial Tidal Joint Probability

Midleton is at risk of both tidal and fluvial flooding. Both sources will therefore contribute to the design flood event, and their dependence needs to be assessed through the use of Joint Probability analysis.

A joint probability analysis has therefore been undertaken as part of the study in order to derive Joint Probability pairings of tidal and fluvial events. The reader is referred to the hydrology report which describes this work in detail.

The fluvial/tidal scenarios for the existing scenario in Midleton are tabulated in Table 7-5 (Fluvial dominated) and

Table 7-6 (Tidal dominated) below. The scenarios highlighted in red represent the required standard of protection of the scheme, i.e. the preferred flood relief option for Midleton will be required to defend up to and included these fluvial/tidal events.

[10] Midleton Surface Water Drainage Scheme – Surface Water Drainage Scheme. Byrne Looby. October 2015

[11] The Midleton Drainage Area Plan is current being advanced by Irish Water and involves the modelling of the surface water and foul sewer networks in the town. Data from the study has been made available to the Midleton FRS project.

Table 7-5 Design fluvial-tidal joint probability scenarios – Fluvial Dominant

Scenario	Design Event	Fluvial contribution	Tidal contribution
Fluvial	Q2	Q2	T2
Fluvial	Q5	Q5	T2
Fluvial	Q10	Q10	T2
Fluvial	Q25	Q25	T2
Fluvial	Q50	Q50	T2
Fluvial	Q100	Q100	T5
Fluvial	Q200	Q200	T10
Fluvial	Q1000	Q1000	T50

Table 7-6 Design fluvial-tidal joint probability scenarios –Tidal Dominant

Scenario	Design Event	Fluvial contribution	Tidal contribution
Tidal	T2	Q2	T2
Tidal	T5	Q2	T5
Tidal	T10	Q2	T10
Tidal	T25	Q2	T25
Tidal	T50	Q2	T50
Tidal	T100	Q5	T100
Tidal	T200	Q10	T200
Tidal	T1000	Q50	T1000

7.3 Hydrogeological assessment

As part of the Midleton FRS project, we undertook a series of hydrogeological assessments to determine the risk of groundwater flooding in Midleton. The study divided Midleton into different flood cells as indicated in Figure 7-5 below.

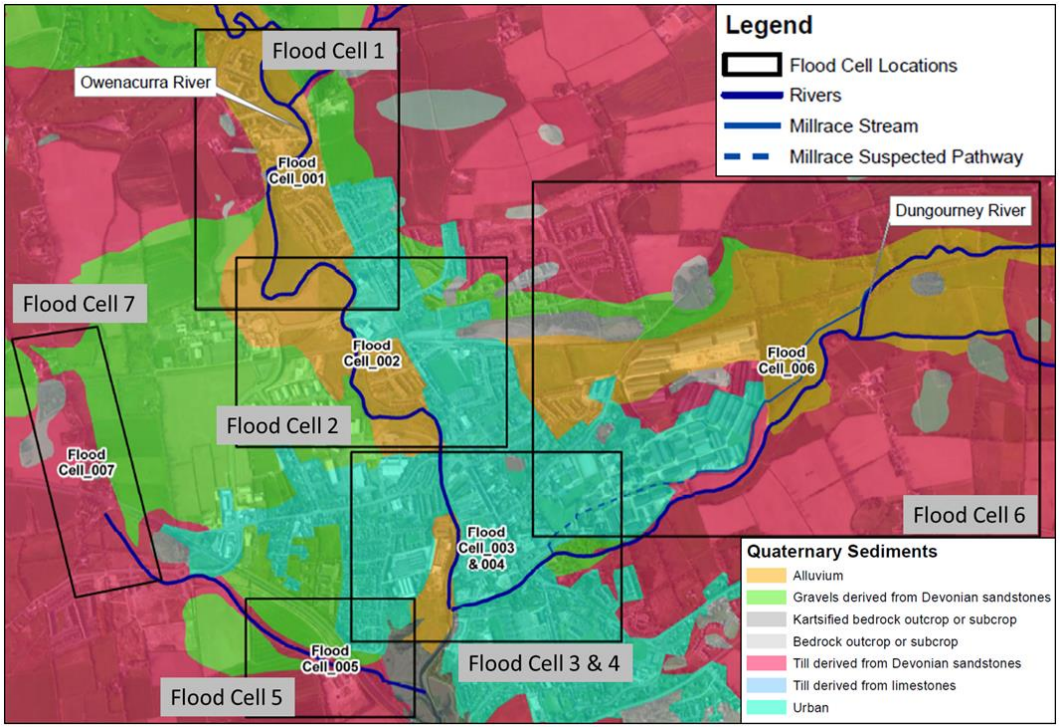


Figure 7-5 Flood cells

The conclusions of the assessment for the individual flood cells are summarised as:

7.3.1 Flood Cell 2, 3 and 4

There is a negligible contribution from Groundwater flooding in these three flood cells, evidenced by the absence of any historic record of groundwater flooding in these areas.

7.3.2 Flood Cell 1

The results of the groundwater assessment in this flood cell concluded that the groundwater contribution to the Owenacurra River is negligible. Modelling of additional inflows to the river from groundwater sources is therefore not required.

7.3.3 Flood Cell 5 and 7

The results of this assessment concluded that the groundwater contribution to both the Owenacurra River and the Water Rock (Oatencake) Stream is negligible. Modelling of additional inflows to the river from groundwater sources is therefore not required.

7.3.4 Flood Cell 6

Analysis of the river water level and groundwater monitoring data collected in Flood Cell 6 indicate that the Dungourney River is in direct hydraulic connection with the gravel and limestone aquifers in this area. The following key points highlight the groundwater-surface water interactions:

- The limestone aquifer is semi-confined with gravels, silts and clays of varying thickness and sequence overlying the bedrock. The groundwater levels in the bedrock aquifer above rockhead elevation. Groundwater levels in the limestone aquifer are generally lower than groundwater levels in the overlying gravel aquifer indicating a mainly downward movement of groundwater.
- The groundwater monitoring data highlights groundwater is flowing from northeast to southwest through the Lauriston Mews/Midleton Rugby Club study area.
- The prevailing groundwater flow direction is towards the River Dungourney. Recharge from the sandstone aquifer to the north flows south into the limestone and gravel aquifers, which then ultimately discharge into the rivers as baseflow.

- The river responds quicker to storm events, with river levels rising above the antecedent groundwater elevation at peak events. This results in a temporary reverse in the groundwater flow direction with river water discharging into the aquifer and locally raising groundwater levels along the river corridor. This can result in groundwater levels rising above the local ground elevation with artesian conditions being observed in the immediate river floodplain.
- The response in groundwater level in the gravel aquifer dampens with distance from the river which demonstrates the influence of aquifer storage effects as water travels from the river through the aquifer. The groundwater flooding issues observed in Lauriston Mews/Midleton Rugby Club are therefore not considered to be a direct response to peak river levels or river water loss into the aquifer but are a result of saturation excess in the aquifer in response to very high rainfall recharge in the catchment during storm events.

These factors are of importance as they indicate that while the prevailing groundwater flow direction is to the river, during a storm even the aquifer responds slow and groundwater is not a significant contributing factor to high water levels seen in the river, but rather during peak events the Dungourney River is losing water to the aquifer.

As groundwater does not contribute to high water levels in the Dungourney river in this area, the hydraulic model has not included any groundwater source discharges. Neither has the model included hydraulic sinks to account for the impact of the Dungourney River losing water to the aquifer. Our approach is therefore conservative as the volume of water that is lost during extreme events is assumed to be contained within the watercourse and floodplain.

8. Hydraulic Modelling Results for the existing (present day) scenario

8.1 Fluvial/Tidal Design Model Runs

The calibrated model was used to simulate the design model runs for both fluvial dominated events and tidal dominated events. In total, 17 design model runs have been simulated as listed in Table 8-1.

Table 8-1 Design model runs

Model Run No.	Scenario	Design event	Fluvial Boundary	Tidal Boundary
1	Fluvial	50%	50%	50%
2	Fluvial	20%	20%	50%
3	Fluvial	10%	10%	50%
4	Fluvial	4%	4%	50%
5	Fluvial	2%	2%	50%
6	Fluvial	1%	1%	20%
7	Fluvial	0.5%	0.5%	10%
8	Fluvial	0.1%	0.1%	20%
9	Tidal	50%	50%	50%
10	Tidal	20%	50%	20%
11	Tidal	10%	50%	10%
12	Tidal	4%	50%	4%
13	Tidal	2%	50%	2%
14	Tidal	1%	20%	1%
15	Tidal	0.5%	10%	0.5%
16	Tidal	0.1%	2%	0.1%
17	Pluvial	1%	n/a	n/a

8.2 Flood Risk Maps and Design water levels at hydraulic model nodes.

Appendix A presents the current scenario flood maps for both fluvial dominated and tidal dominated sources of flooding. Fluvial flood extent maps are presented for the 10%, 1% and 0.1% AEP events while fluvial flood depth maps are presented for the 1% AEP event.

Tidal flood extent maps are presented for the 10%, 0.5% and 0.1% AEP events while Tidal depth maps are presented for the 0.5% AEP event. Four flood maps for the Mid-Range Future Scenario are also presented in Appendix A for the relevant areas.

Maximum water levels at each of the nodes in the 1D model, and for the full range of return periods events, are presented in Appendix B for both fluvial and tidal dominated events. Longitudinal plots of maximum water levels are also presented.

The results are discussed in the following sections of this report.

8.3 Discussion of the Fluvial Flood Risk for the Current Scenario

The baseline design model runs for the existing scenario assumes no blockages at any of the culverts or bridges in the model. This includes the structures for which there is a historic record of blockages as discussed earlier in the report in Section 4. A sensitivity on the occurrences of blockages and their impact on flood risk, will be considered at the optioneering stage of the project.

The discussion of the existing scenario in the following sections of the report has been broken down into the following individual areas:

- Tir Cluain/Moore's Bridge (Owenacurra flood risk)
- Northern Relief Road/Railway cottages (Owenacurra flood risk)
- Town Centre (Owenacurra flood risk)
- Upstream of the IDL site (Dungourney flood risk)
- The Baby's Walk/Lower Main Street (Dungourney flood risk)
- Downstream of the N25 road (Tidal dominated flood risk)
- Ballinacurra
- Water Rock (Oatencake) Stream

As noted in Section 8.2, the Existing Scenario flood extent maps for the fluvial and tidal events are presented in Appendix A.1 and A.2 respectively.

8.3.1 Tir Cluain/Moore's Bridge (Owenacurra)

The Owenacurra River gets out of bank north of Tir Cluain estate in the Q2 year event however flood water does not enter the estate until the Q10 event. During this event, the volume of water collecting to the East of the estate is greater and overtops the embankment into the estate. Once water enters the estate it flows eastward and floods the access roads to a shallow depth of circa 0.02-0.05m. 11 residential properties are inundated for the Q10 event.

The same mechanisms of flooding occur for the Q100 event. The perimeter brick wall surrounding sections of the estate has been removed from the model run for this event. The model therefore represents the mechanism of flood waters entering the estate from the Owenacurra floodplain at a number of locations. The Q100 event results in maximum flood depths on the access road of circa 0.3m. More than sixty-five residential properties in the estate are inundated by the Q100 event.

The southern area of Tir Cluain is at risk of flooding from overland flow as described above. However, it is also at risk from the Owenacurra overtopping its banks upstream of the access bridge into the estate.

Due to the raised soffit levels of the New Moore's bridge, as discussed in Section 6.8 the bridge is no longer being overtopped for a Q100 existing scenario design event. However, three downstream properties on the right bank are still at risk of flooding during the Q100 design event.

The primary mechanisms of flooding on the Elfordstown/ Glenathonacash Stream is overtopping of the left bank of the stream immediately upstream of the stone arch bridge (Figure 8-1) situated along the main Midleton to Lisgoold road. Water levels at this location are a function of both the Owenacurra backwatering up the stream as well as an afflux caused by the bridge itself.



Figure 8-1 Bridge opening on the Elfordstown/ Glenathonacash stream

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8-2 for the Tir Cluain/Moore's Bridge area.

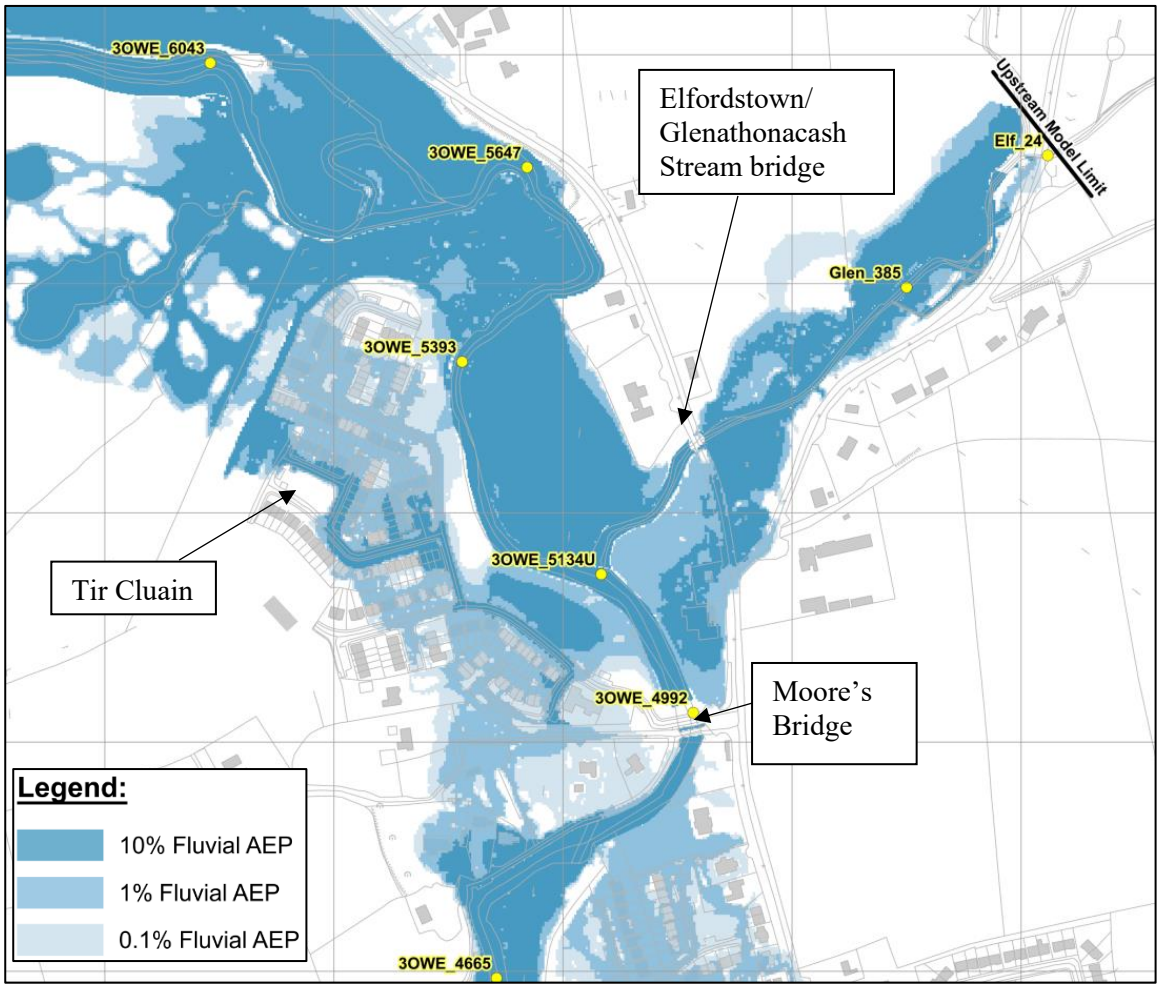


Figure 8-2 Existing Scenario Fluvial Flood Extents- Tir Cluain/Moore's Bridge

8.3.2 Northern Relief Road/Railway cottages (Owenacurra)

The Northern Relief Road (NRR) Bridge and Weir cause a significant afflux along the Owenacurra River upstream of the structures. The soffit of the NRR is 8.43mOD and the maximum Q100 water level upstream of the bridge is 8.65mOD. The bridge opening is therefore fully surcharged in the design event.

The design runs have assumed that the existing brick wall and brick shed on the left bank of the Owenacurra, upstream of the NRR, keeps water in-bank up to the Q25 event. The wall and shed however have been removed from the model for the Q50 event (and above) and accordingly, water can overtop the left bank of the Owenacurra at this location for events of this magnitude. Once water escapes, it flows in a South Easterly direction and proceeds to flood the R626 road and the Railway cottages. As the cottages are located at a low point in the topography, flood depths at the properties for the Q100 event are in excess of 1.9m.

The area of the Railway cottages is also at risk from two other mechanisms of flooding, as follows:

- Flood water escapes the Owenacurra River immediately downstream of the NRR and floods the site adjacent to the river in both the Q50 and Q100 events. For the Q100 event, the volume of water on the site is sufficient to overtop the high point in the site and cause water to flow across it and inundate the area of the railway cottages.
- In the Q100 event, flood water escapes the millrace immediately upstream of the railway culvert and flows overland towards the Railway cottages. As discussed earlier in the report, this location is prone to siltation and blockages which have not been considered as part of the baseline model runs, and which could further exacerbate this situation.

The railway crossing that intersects the R626 road (also known as Mill Road) in the vicinity of the Railway cottages is elevated above existing ground levels and therefore acts as a control on flood levels. In the Q100 event, the volume of water collecting on the northern side of the railway line is sufficient to overtop the crest level and cause water to flow down Mill Road. This mechanism acts as a release for the stored water and prevents greater flood depths occurring at the Railway cottages.

A large number of residential/commercial properties are inundated along Mill Road and water is conveyed as far as the Supervalu supermarket and Maxol Petrol Station at the Cork Road Roundabout. The Millbrook housing estate to the East of the R626 is also partly inundated. The existing embankment at the left bank of Owenacurra river near Millbrook is not overtopped in the 1 in 100year event.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8-3 for the Northern Relief Road/Railway cottages area.

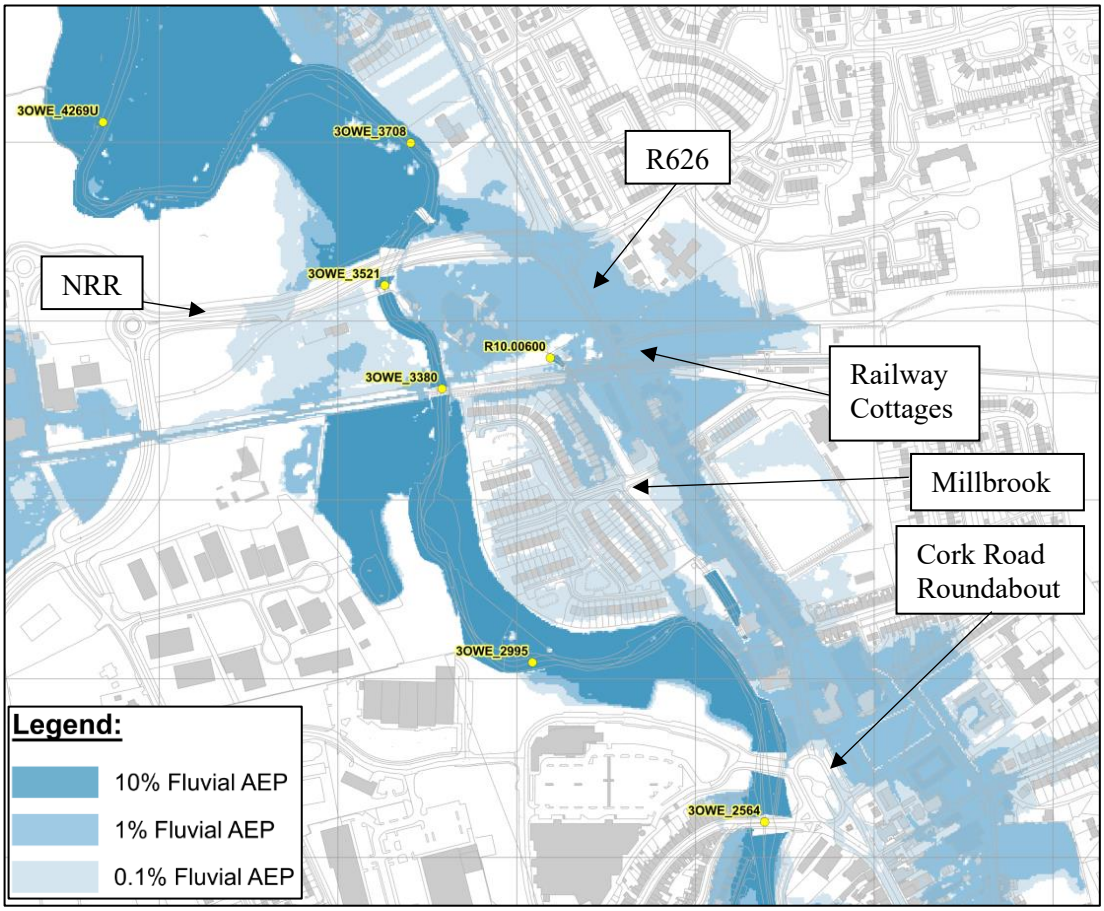


Figure 8-3 Existing Scenario Fluvial Flood Extents- Northern Relief Road/Railway cottages

8.3.3 Town Centre (Flood risk from Owenacurra)

There are a number of mechanisms by which water from the Owenacurra floods the main town centre in Midleton as described below:

- flood water can overtop the railway line and flow south along the R626 road as far as the Cork Road Roundabout;
- Water overtops the downstream end of the Owenacurra Millrace in the Q100 event and enters the site of the "Millrace" apartment block complex. From here, it flows out onto the R626 road and proceeds to flow south towards the Supervalu supermarket.
- Water overtops the left bank of the Owenacurra upstream of the Market Green Plaza Bridge in the Q100 event. The volume of water escaping the channel at this location is relatively minor and flood risk is primarily driven by overland flow coming from the R626 as discussed above.
- Water overtops the right bank of the Owenacurra at the Woodlands estate for the Q5 event but does not inundate any of the properties. For the Q100 event, sixteen properties are inundated to a depth of circa 0.35m.
- Water exceeds the right bank level upstream of Lidl Bridge during a Q100 event. The flood extent reaches the Lidl car park and partially floods Lidl. On the left bank, the water flows south along Riverside way overtopping church lane and ponding in a depression south of St John church.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8-4 for the Town Centre.

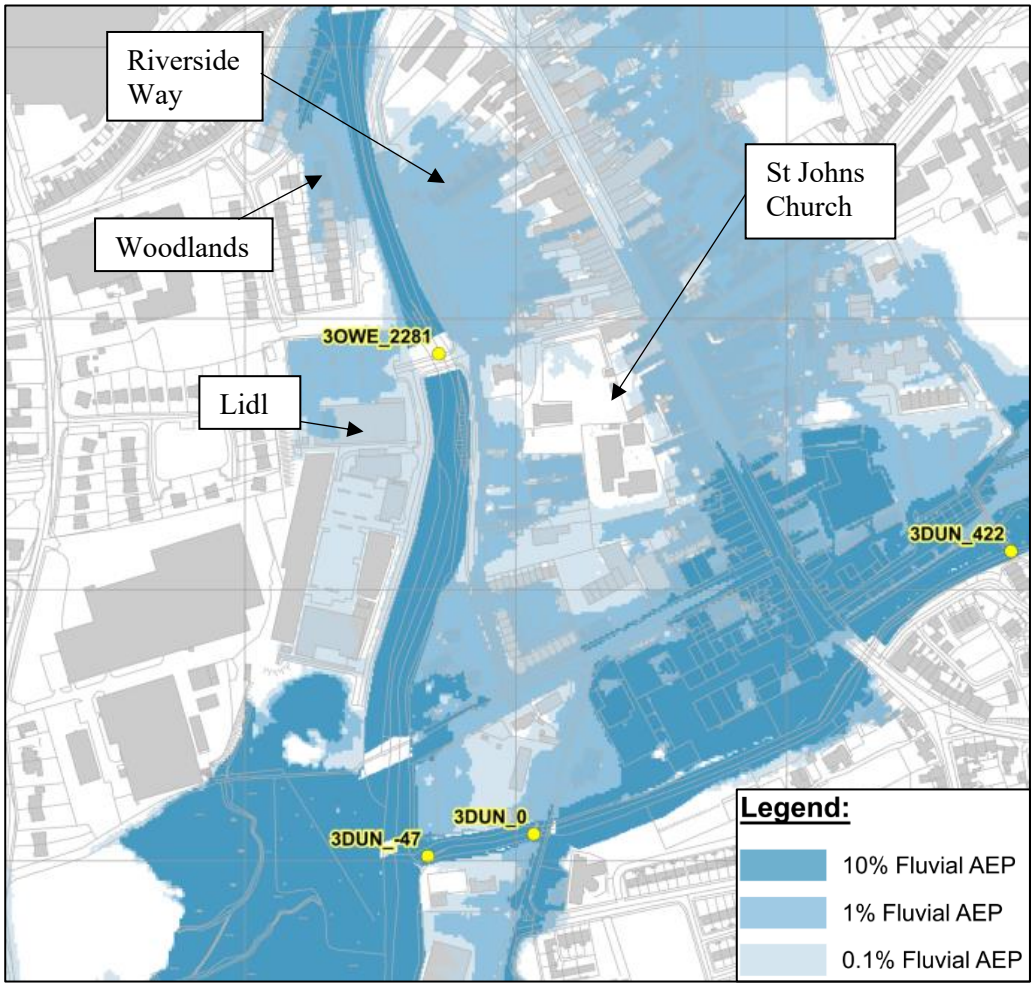


Figure 8-4 Existing Scenario Fluvial Flood Extents- Town Centre

8.3.4 Upstream of the IDL site (Dungourney flood risk)

Water escapes the Dungourney River at a number of locations upstream of the IDL site in the Q5 event, resulting in the inundation of large areas of agricultural land. The IDL site however is not at risk of inundation from upstream of the site, given that it is elevated above existing ground level and defended by a large embankment around the perimeter of the site.[12] Water does not therefore inundate the IDL site but instead collects upstream of it.

Small areas of the IDL site are however at risk during the Q100 event due to water escaping the millrace upstream of a number of undersized structures on the millrace. Flood risk from the millrace is minor.

For the Q100 event, water flows in a westerly direction along the Greenway. However, levels/volumes are not sufficient to overtop the exiting high ground north of the rugby grounds and flood the play pitches. This aligns with observations during the Storm Babet event.

The area of Lauriston Estate is however, at high risk of flooding from groundwater. The reader is referred to the Hydrogeology report for a detailed discussion of groundwater mechanisms in this area.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 extents as well as the groundwater extent is presented in Figure 8-5 for the area upstream of the IDL site.

[12] It is noted that this embankment is not intended to act as a flood defence structure. Further it is noted that there are two minor openings in the embankment which have not been accounted for in the model.

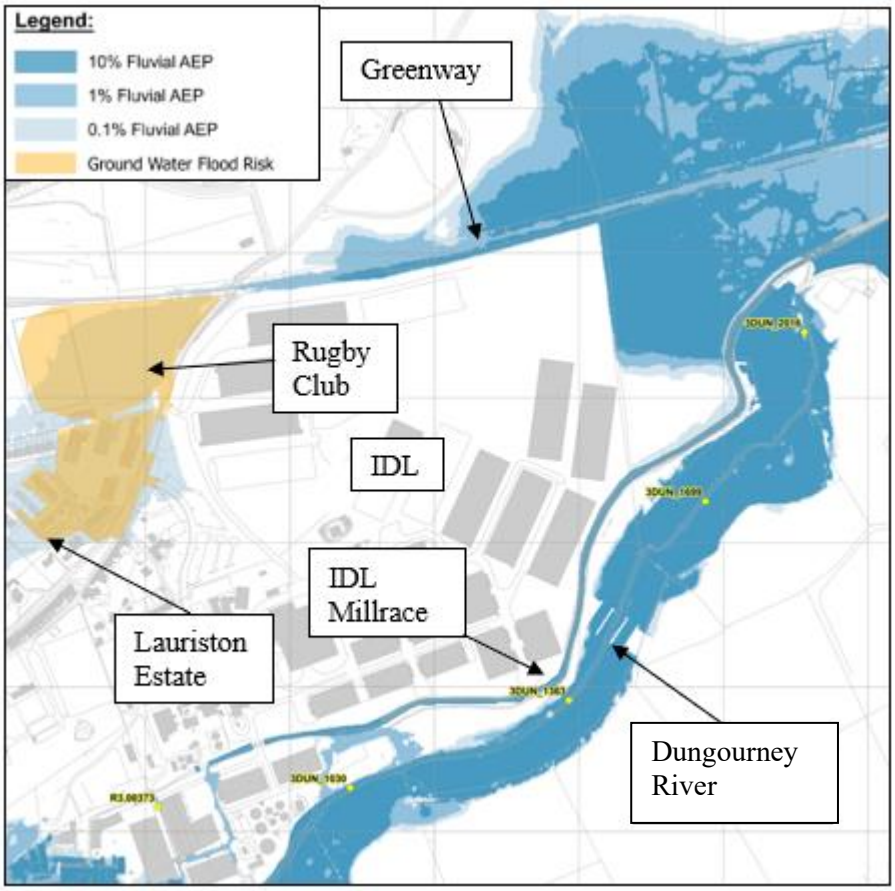


Figure 8-5 Existing Scenario Fluvial Flood Extents- Upstream of the IDL site

8.3.5 The Baby's Walk/ Lower Main Street (Dungourney flood risk)

The main area of the town in the vicinity of The Baby's Walk and Main Street is at risk of flooding from the Dungourney River for both fluvial and tidal events. The threshold of flooding for fluvial flooding is the Q2 event which inundates the area of The Baby's Walk and Cuddigan's Yard. Circa 3 properties are flooded in Cuddigan's Yard. The Q5 event leads to more significant flooding and inundates circa 34 properties at the southern section of Main Street. Extensive areas of the main town centre are flooded for the Q100 event and over 211 properties in the vicinity of Main Street are inundated.

For the large return period events, flood risk in the vicinity of Main Street is due to both the Dungourney river breaching its banks and also from overland flow coming from the Owenacurra River as detailed in 8.3.3.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8.6 for the Baby's Walk/ Lower Main Street area.

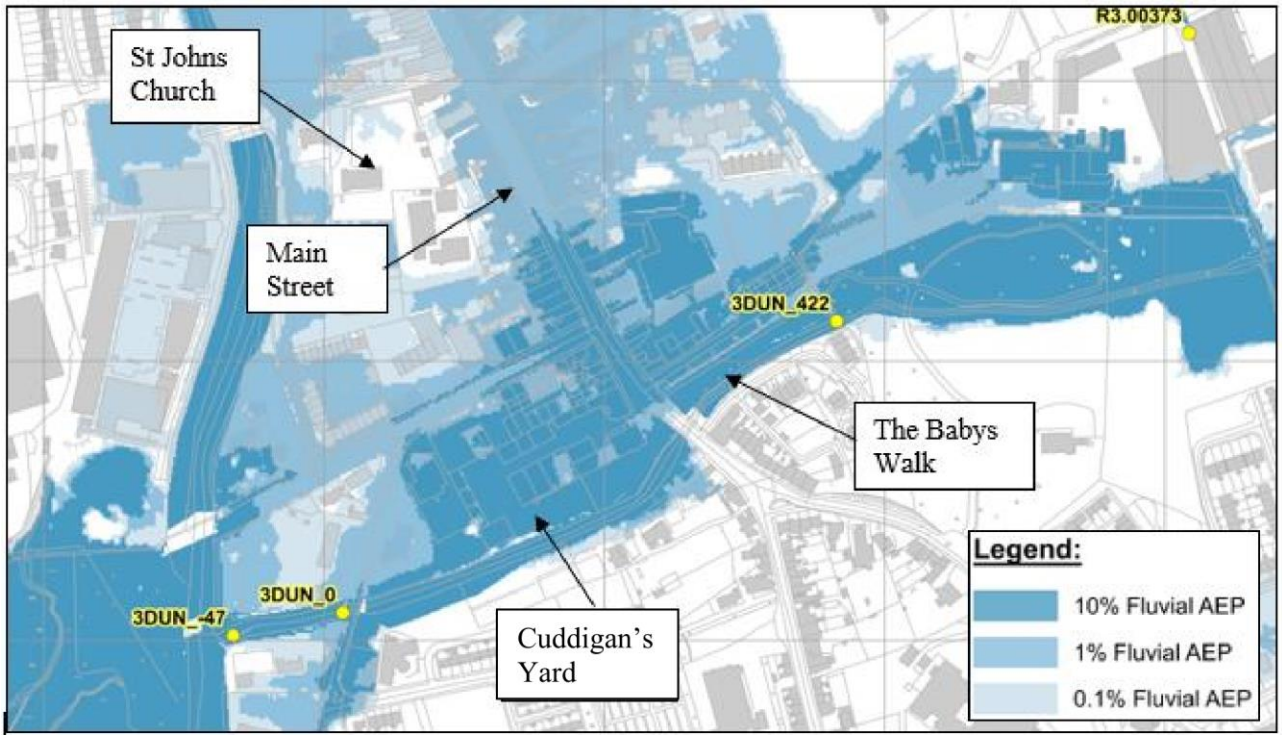


Figure 8-6 Existing Scenario Fluvial Flood Extents- The Baby's Walk/ Lower Main Street

It is noted that as this area of the town is subject to both fluvial and tidal flooding, flood risk is dependent on the fluvial/tidal Joint Probability pairings. Tidal risk for this area is presented later in this report.

8.3.6 Downstream of the N25 road

Flood risk downstream of the N25 is predominately tidal and is addressed later in Section 8.4.

8.3.7 Water Rock Stream

Water levels upstream of the Cave system are dominated by the significant head loss associated with the narrow entrance into the Cave system. Groundwater levels within the cave system can also increase water levels upstream by restricting flow through the system and in effect acting as a blockage to the watercourse.

The Q100 event inundates Water Rock House and overtops the local access road. A number of residential properties adjacent to the road are also inundated. There is a sufficient head of water over the road to cause a significant volume of water to flow east beyond the residential properties and flow towards the Owenacurra. This overland route leads to the inundation of a number of commercial properties in the Northern Point Business Park. Once the water travels further East and reaches the NRR, it collects behind the road embankment and overtops the rail line and from there proceeds to travels south.

While the volume of water flowing south in the Q100 year event is relatively minor, it is quite significant for the Q1000 event and leads to the inundation of a number of commercial properties in the vicinity of the Cork Road.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8-7 for Water Rock upstream.

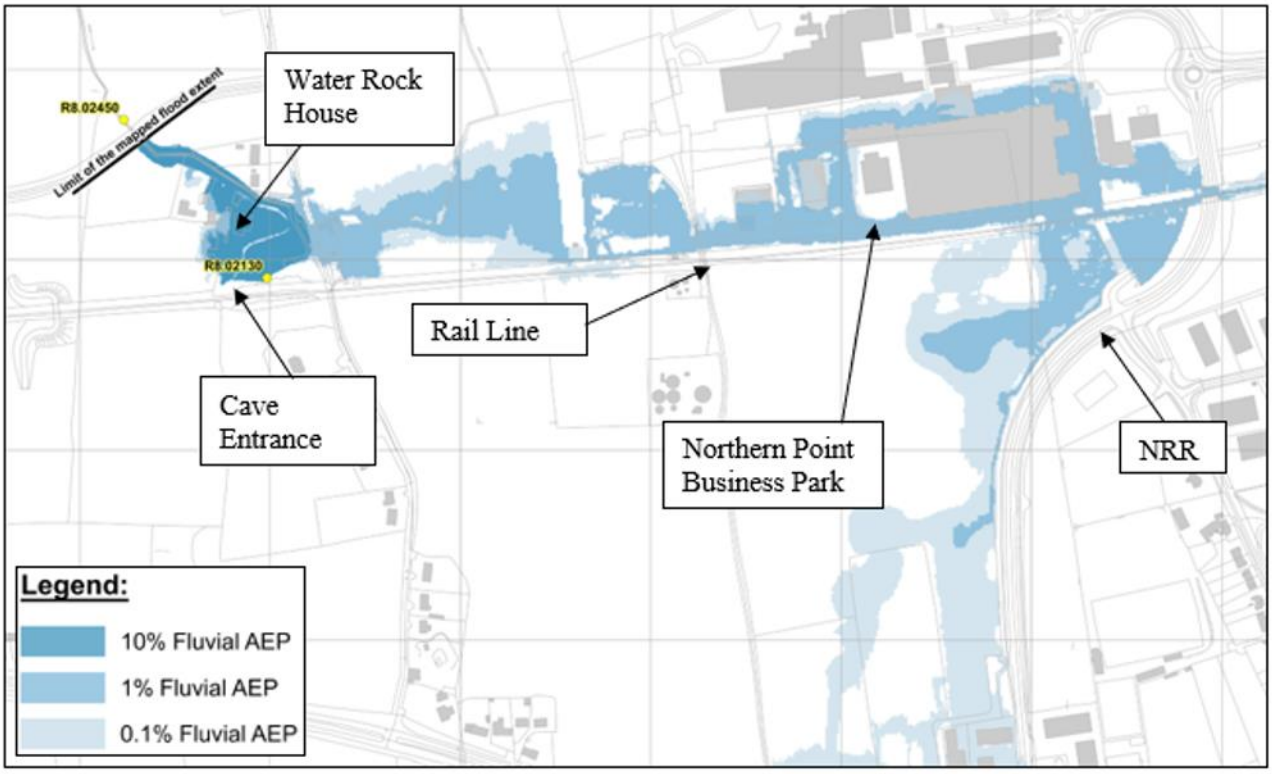


Figure 8-7 Existing Scenario Fluvial Flood Extents- Water Rock upstream

A number of properties (including the WWTP) downstream of the Cave system are also at risk from fluvial flooding. The fluvial flood extents downstream of the Cave system are based on model results which account for the full hydrologically estimated flow from the HEP being capable of reaching downstream of the Cave system i.e. the loss of water from the Water Rock Stream upstream of the Cave system is not accounted for in the specification of the design inflow at the downstream end of the reach. This is a conservative assumption.

There is also a risk of flooding due to the prolonged tidal locking of the downstream culverts adjacent to the WWTP. During periods of tide locking, fluvial flow coming from the cave system get out of bank and floods the floodplain north-west of the WWTP. In extreme events greater than the Q10 event, water exceeds the available volume of this floodplain and overtops the local road, traveling under the N25 and reaching Riversfield Estate.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8-8 for Water Rock downstream.

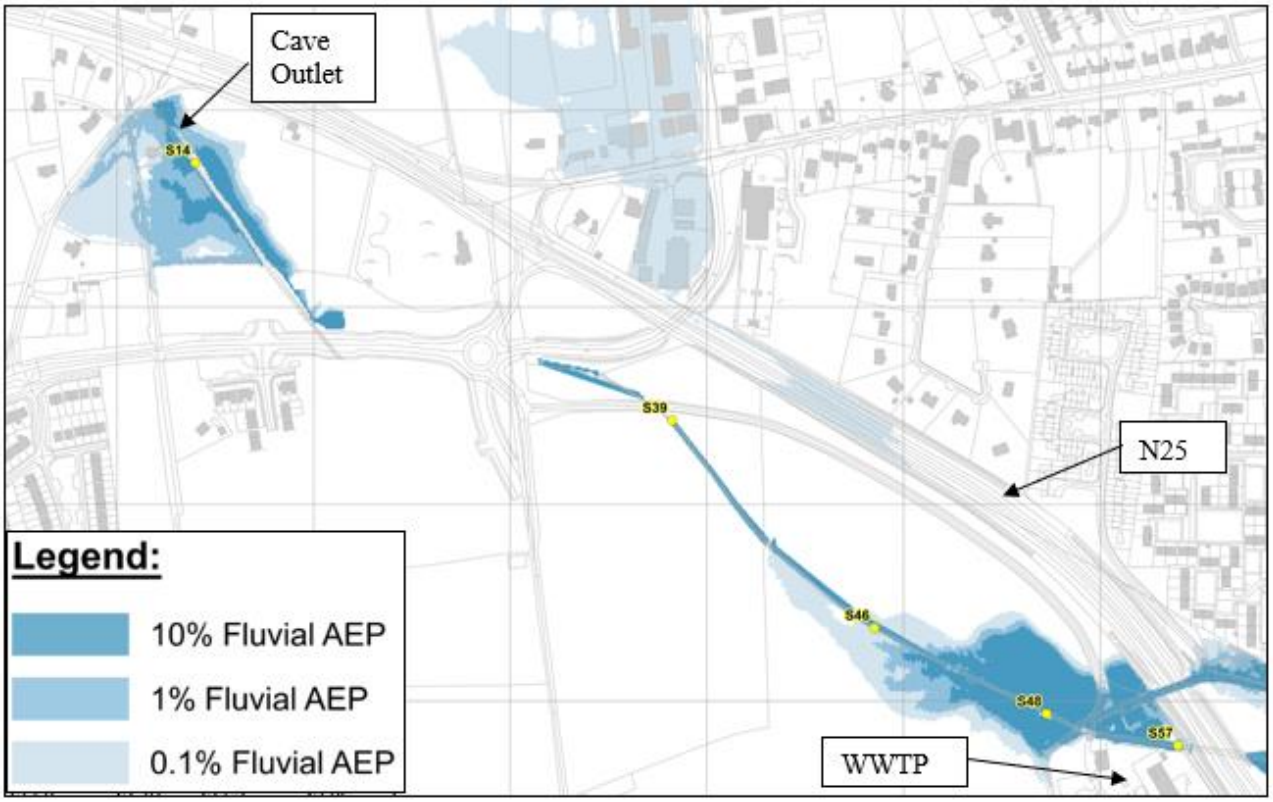


Figure 8-8 Existing Scenario Fluvial Flood Extents- Water Rock downstream

8.3.8 Ballinacurra Stream

Flood risk at the lower end of the Ballinacurra stream is dominated by tide locking of the flap valve at the downstream end of the culvert underneath the R630 (Main Midleton/Whitegate Road). For the period during which the flap valve is tide locked, water from the Ballinacurra cannot enter the estuary and therefore collects within the channel. The threshold of flooding within Ballinacurra Village is the Q2 event where more than 6 properties are at risk. For the Q100 event more than 37 properties are at risk.

An extract of the Existing Scenario Fluvial flood extent map for the Q10, Q100 and Q1000 events is presented in Figure 8-9 for Ballinacurra.

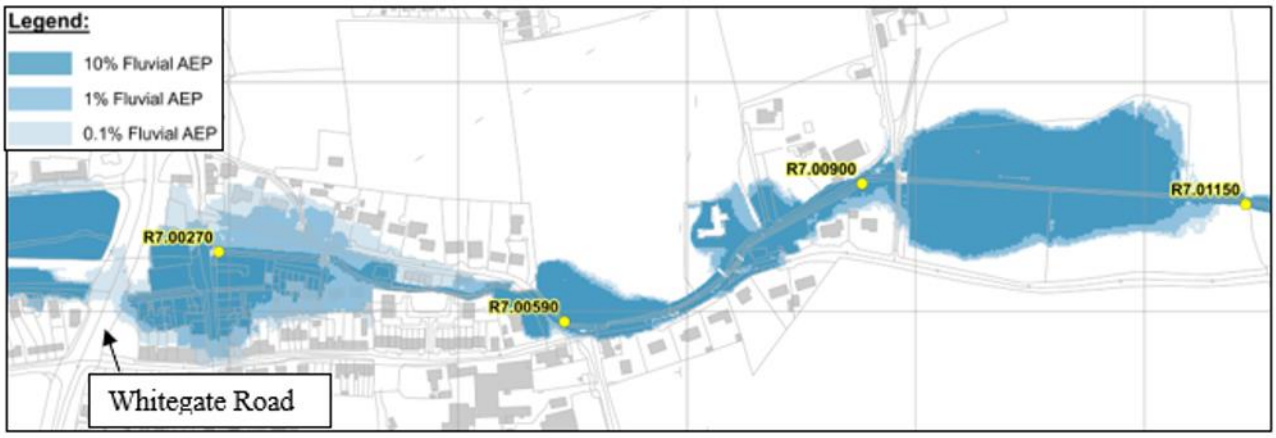


Figure 8-9 Existing Scenario Fluvial Flood Extents-Ballinacurra

8.4 Discussion of the Tidal Flood Risk for the Current Scenario

Areas of the main town centre at the downstream end of Main Street are at risk from tidal flooding. The tidally influenced reach extents as far upstream as the IDL site on the Dungourney River and as far as the Woodlands estate on the Owenacurra River. The threshold of tidal flooding of the commercial properties along Distillery Walk is equivalent to the T5 design event while two properties are shown to be at risk of flooding in the T2 event in Cuddigan's Yard.

An extract of the Existing Scenario Tidal flood extent map for the T10, T200 and T1000 events is presented in Figure 8-10 for the Town Centre.

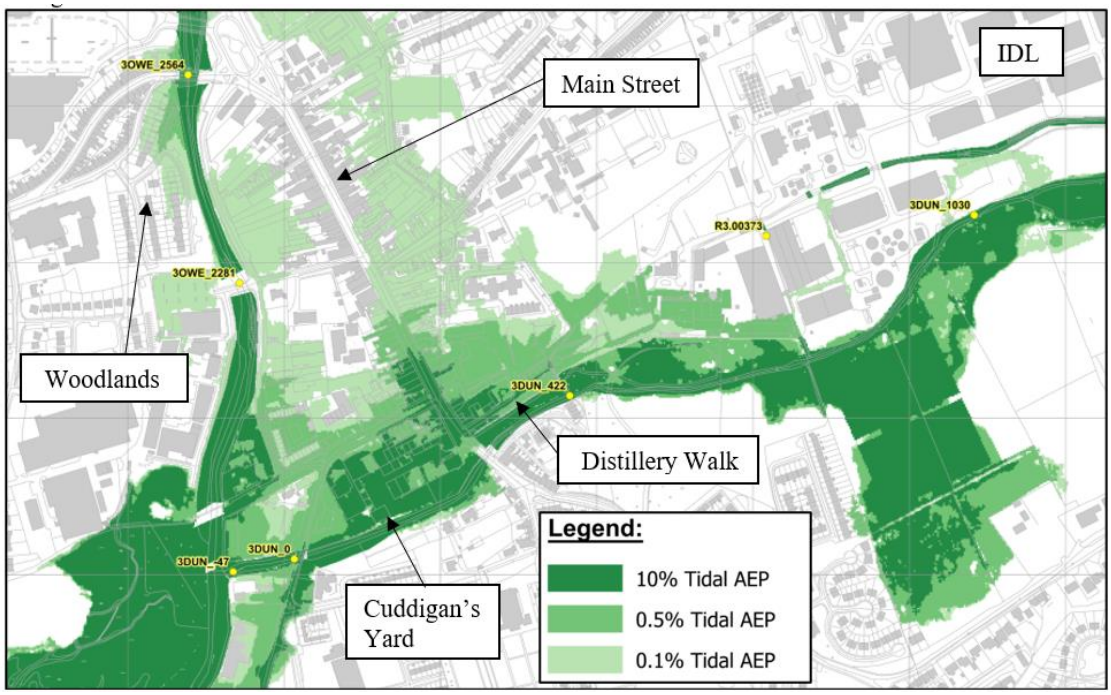


Figure 8-10 Existing Scenario Tidal Flood Extents- Town Centre

The threshold of flooding along Bailick Road is equivalent to the T2 design event and is therefore very low. The threshold of flooding for the properties in this area is greater due to higher floor levels. For the five low lying properties it is equivalent to the T5 event. The majority of properties in this area are first inundated by the T25 event.

An extract of the Existing Scenario Tidal flood extent map for the T10, T200 and T1000 events is presented in Figure 8-11 for the Bailick Road area.

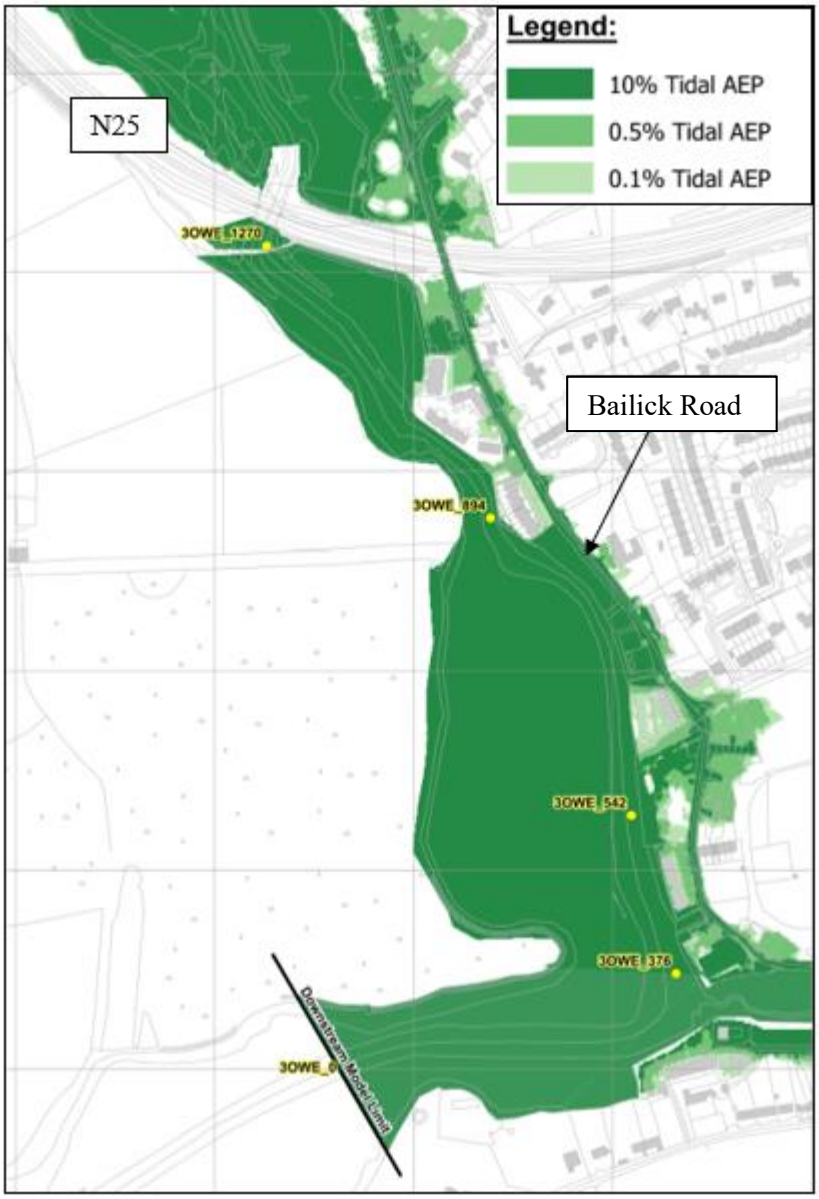


Figure 8-11 Existing Scenario Tidal Flood Extents- Bailick Road

It is assumed in the design model runs that the flap valve at the downstream end of the culvert underneath the main Midleton/Whitegate Road does not fail during tidal flood events. However, as in the fluvially dominated event outlined in Section 8.3.8, there is risk of tide locking which results in flooding of Ballinacurra due to the upstream fluvial flow being unable to discharge to the estuary.

The downstream end of the Water Rock catchment is also at risk of tidal flooding.

9. Climate Change Scenario Model Runs

9.1 Overview

An increase in flood risk associated with climate change was considered as part of the study. Two separate epochs were assessed as outlined in Table 9-1 below.

Table 9-1 Climate Change Uplifts

Scenario	Hydrological Forcing	Tidal water level
Mid-Range Future Scenario (MRFS)	+20% increase in the peak fluvial flow	+0.5m increase in the peak water level
High-End Future Scenario (HEFS)	+30% increase in the peak fluvial flow	+1m increase in the peak water level

9.2 Discussion of Flood Risk for Climate Change scenarios

The fluvial and tidal flood risk for the Mid-Range Future Scenarios (MRFS) and High-End Future Scenarios (HEFS) are discussed in the following sections of the report. Flood extent maps for these scenarios are presented in Appendix A.5 and A.6. It is noted that these future scenario flood extents inform the Climate Change Adaptation Study for the project which is reported on separately as part of Stage 1 of the project.

9.2.1 Tir Cluain/Moore's Bridge (Owenacurra)

MRFS

The mechanisms of flooding that occur in the Current scenario also occur for the MRFS. There is however an increase in the maximum flood extent in Tir Cluain and Willowbank due to the 20% uplift in flows associated with the MRFS. An additional circa 86 residential properties are inundated by the 1% Fluvial AEP undefended MRFS event when compared to the 1% Fluvial AEP undefended current scenario.

There is a greater flood extent on Upper Mill Road in the MRFS compared to the current scenario. This increases flood depths and extents at the junction with the NRR. However,, the contribution is small compared to the flood mechanism upstream of the Northern Relief Road on the left bank.

HEFS

The same mechanisms of flooding occur for the HEFS as for the MRFS. The higher design flow for this event results in a slightly larger maximum flood extent. An additional 11 residential properties are inundated by the 1% Fluvial AEP undefended HEFS event when compared to the 1% Fluvial AEP undefended MRFS scenario. The increase in the maximum water level for the HEFS event compared to the MRFS event in this area ranges from circa 0.01m - 0.18m.

9.2.2 Northern Relief Road/Railway cottages (Owenacurra)

MRFS

The MRFS mechanisms of flooding are the same as for the Current scenario. The greater volume of water associated with the event leads to higher water levels and flood extents. The flood extent around the Railway cottages is largely the same as he current scenario, as the increased flood volume from the MRFS event overtops the railway line and flows down the R626.

The MRFS event leads to a larger flood extent in the Millbrook Estate and a number of additional properties in the south of the estate are inundated in the 1% Fluvial AEP undefended MRFS extent.

Overland flow from the Water Rock Stream to the West inundates an area of land to the south of the Northern Relief Road for the 1% Fluvial AEP MRFS. This site does not flood in the 1% Fluvial AEP undefended current scenario.

HEFS

The same mechanisms of flooding occur for the HEFS as for the MRFS and Current Scenario. The higher design flow for this event results in a slightly larger maximum flood extent and larger flood depths across the area.

9.2.3 Town Centre (Flood risk from Owenacurra)

MRFS

The MRFS fluvial mechanisms of flooding are the same as for the Current scenario. However, the greater volume of water associated with the event leads to higher water levels and larger flood extents. The higher tidal boundary (0.5m higher than the current scenario) leads to a much greater flood extent in the tidally dominated reach. In addition to this, for the MRFS, the volume of water passing along the Greenway is large enough to flood the Rugby pitches. Due to the topography, water then travels south and floods Lauriston Estate and ultimately reaching the town centre where it joins the flood extent from the Owenacurra.

A number of additional properties are flooded in the Woodlands Estate on the right bank immediately downstream of the Cork Road Bridge. Thomas Street and the surrounding area are also inundated the 1% Fluvial AEP undefended MRFS event.

The greater flow rate associated with the MRFS coupled with the increased backwatering from the MRFS tidal level increases the flood extent upstream of Lidl Bridge. The MRFS flood extent is consequently greater than the current scenario. Downstream of Lidl Bridge there are three relief points where the flow coming from upstream of Lidl Bridge returns to the Owenacurra over the right bank. For the 0.5% Tidal AEP event, the relief points turn into flood paths from the Owenacurra to the commercial buildings. The mechanism remains the same for the current and MRFS along the left bank.

The maximum MRFS design flood level transitions from the 1% Fluvial AEP event to the 0.5% Tidal AEP event just downstream of the Lidl Bridge i.e. this is the point at which the flood risk transitions from being fluvially dominant to tidally dominant. It is evident from the results, that the 0.5m increase in the maximum tidal level has a significant impact on the 0.5% Tidal AEP design flood event maximum extent and depths in this area and causes a number of additional residential and commercial properties to be inundated. The entire Riverside Way commercial development (which includes the Lidl, McDonalds, Aldi etc) is inundated in this event due to the higher tidal boundary. Additional commercial properties are also inundated on the opposite bank.

The MRFS also inundates the site of the WWTP's pumping station as well as Chadwick's commercial property along the Bailick Road. Circa 11 residential properties adjacent to the Choctaw Park are also inundated in this event.

HEFS

The HEFS mechanisms of flooding are the same as for the MRFS and Current scenario. The fluvially dominated area flood extent for the HEFS in marginally larger when compared to MRFS through the town centre.

Due to the local topography the higher water levels associated with the HEFS does not however result in any significant increase in the maximum extent of the HEFS when compared with the MRFS at the tidally dominated area. The flood depths are circa 0.5m higher in the tidally dominated reach compared to the MRFS, this is consistent with the change in tidal boundary being applied.

9.2.4 Upstream of the IDL site (Dungourney flood risk)

MRFS

The MRFS mechanisms of flooding are the same as for the Current scenario except that that the greater volume of water associated with the event leads to higher water levels and flood extents.

HEFS

The HEFS mechanisms of flooding are the same as for the MRFS and Current scenario. As the HEFS however includes for a 1m increase in the peak tidal water level, the area inundated is significantly increased.

9.2.5 The Baby's Walk/Lower Main Street (Dungourney flood risk)

MRFS

Due to the higher downstream boundary associated with the MRFS, the fluvial/tidally dominant transition point is located just upstream of the Dungourney footbridge in front of the CCC offices.

Design water levels around The Baby's Walk and Lower Main Street are driven by the 0.5% Tidal AEP Event and the maximum water level in The Baby's Walk and Lower Main Street is circa 0.4m higher than the current scenario 0.5% Tidal AEP maximum water level.

This increased water level results in a larger maximum flood extent and as a consequence, an additional 10-15 residential properties are inundated by the 0.5% Tidal AEP flood extent.

HEFS

The HEFS mechanisms of flooding are the same as for the MRFS and Current scenario. As the HEFS however includes for a 1m increase in the peak tidal water level, the length of the tidally dominated reach is increased. Due to the local topography the higher water levels associated with the HEFS does not however result in any significant increase in the maximum extents of the HEFS when compared with the MRFS for the right bank of the Dungourney river. Along the left bank of the river, there is a new flood path at An Charraig road where 20 properties are now within the flood extents. The flood depths are circa 0.5m higher in the tidally dominated reach.

9.2.6 Downstream of the N25 road (Tidal dominated flood risk)

MRFS/HEFS

Flood risk downstream of the N25 is significantly increased in both the MRFS and HEFS due to the increase in the design tidal water level. An extra circa 79 properties are inundated in the MRFS when compared with the current scenario. An extra circa 94 properties are inundated in the HEFS when compared with the current scenario.

9.2.7 Water Rock (Oatencake) Stream

MRFS

The MRFS extent is similar to the current scenario. The flood depths are however greater due to the greater volumes of water.

HEFS

The WWTP is flooded in the HEFS T2 event due to the increases in tidal water level of 1m. Riversfield's flood extent is considerably larger, putting circa 58 more properties at risk in the Q100 HEFS compared to the Q100 current scenario.

9.2.8 Ballinacurra Stream

MRFS/HEFS

The MRFS and HEFS mechanisms of flooding are the same as for the Current scenario except that the greater volume of water associated with the event and the longer time during which the flap value is tide locked, leads to higher water levels and flood extents upstream of the main Midleton/Whitegate Road. The Midleton/Whitegate Road is also overtopped by the tide in the HEFS event due to the increased water level associated with this event.

10. Pluvial and Groundwater flood risk mapping

10.1 Introduction

Both Pluvial and Groundwater flood risk maps have been produced as part of the study. The methodology adopted to produce the maps is detailed in this section of the report.

10.2 Pluvial Flood risk to Lower Main Street/The Baby's Walk

As noted in Section 7.2.6.

10.3 Pluvial Flood Risk Map for the Scheme Area

The Pluvial flood risk maps were produced using the results of the Pluvial model developed as part of the study. The development of the model is described in the following sections.

10.3.1 Pluvial Model – Hydrological Estimation

The Flood Studies Update (FSU) Depth Duration Frequency (DDF) dataset was used to determine design rainfalls depths for the Pluvial modelling. The Flood Study Report FSSR16 Rainfall-runoff method was used to estimate the critical storm duration for the main area of the town.

A hyetograph for the 100yr rainfall event was developed using FSU DDF data in Microdrainage. An allowance for the capacity of the existing drainage network in the town was made by subtracting an assumed equivalent rainfall capacity of 2mm/hour from the 100yr hyetograph. The resulting hyetograph therefore represents the rain falling on the catchment which might be expected to generate overland flow. Figure 10-1 presents the finalised 100yr rainfall event hyetograph that was used as the boundary condition to the pluvial model.

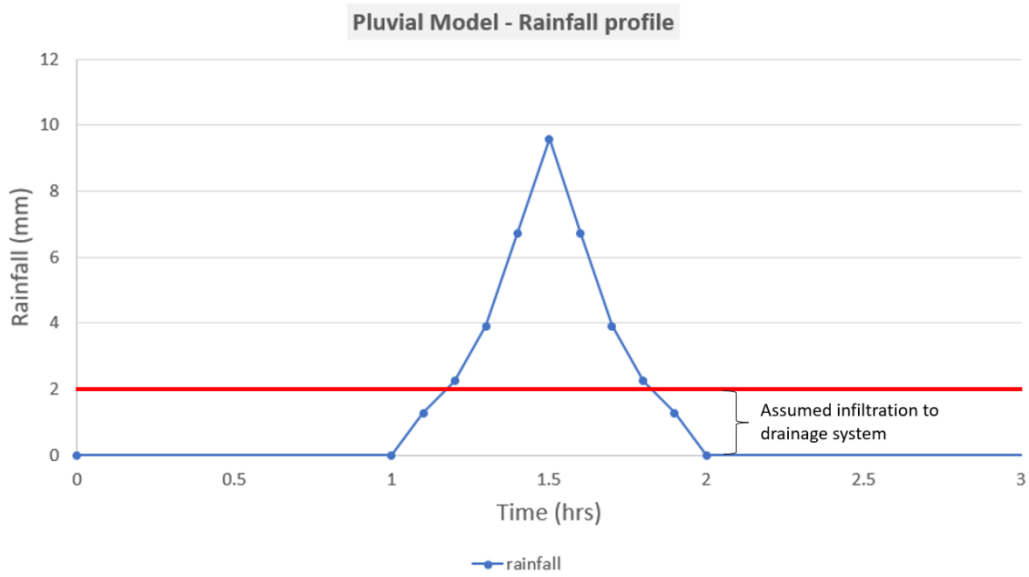


Figure 10-1 Pluvial model hyetograph

10.3.2 Pluvial Model – Hydraulic Modelling

The design 100yr rainfall event hyetograph was applied to the 2D component of the hydraulic model as a time-varying area boundary. This allowed for the rainfall to be directly applied to each of the individual grid cells of the model. The pluvial model then simulates the resulting surface water runoff and overland pluvial flow routes. The areas at risk from pluvial flooding can then be identified. All the walls and floodplain features identified as part of the Fluvial Modelling were included in the pluvial model as it has been assumed that none of these would fail in the design pluvial event.

10.3.3 Pluvial Model – Results

A sample of the pluvial flood risk map is presented in Figure 10-2 and a larger map is presented in Appendix A.3. It can be seen from the results that there are a number of areas in Midleton at risk of pluvial flooding. These include:

- Thomas Street;
- Lauriston Housing Estate;
- Rugby Club;
- Railway Cottages;
- The Baby's Walk/Bottom of Main Street;
- Europa Business Park.

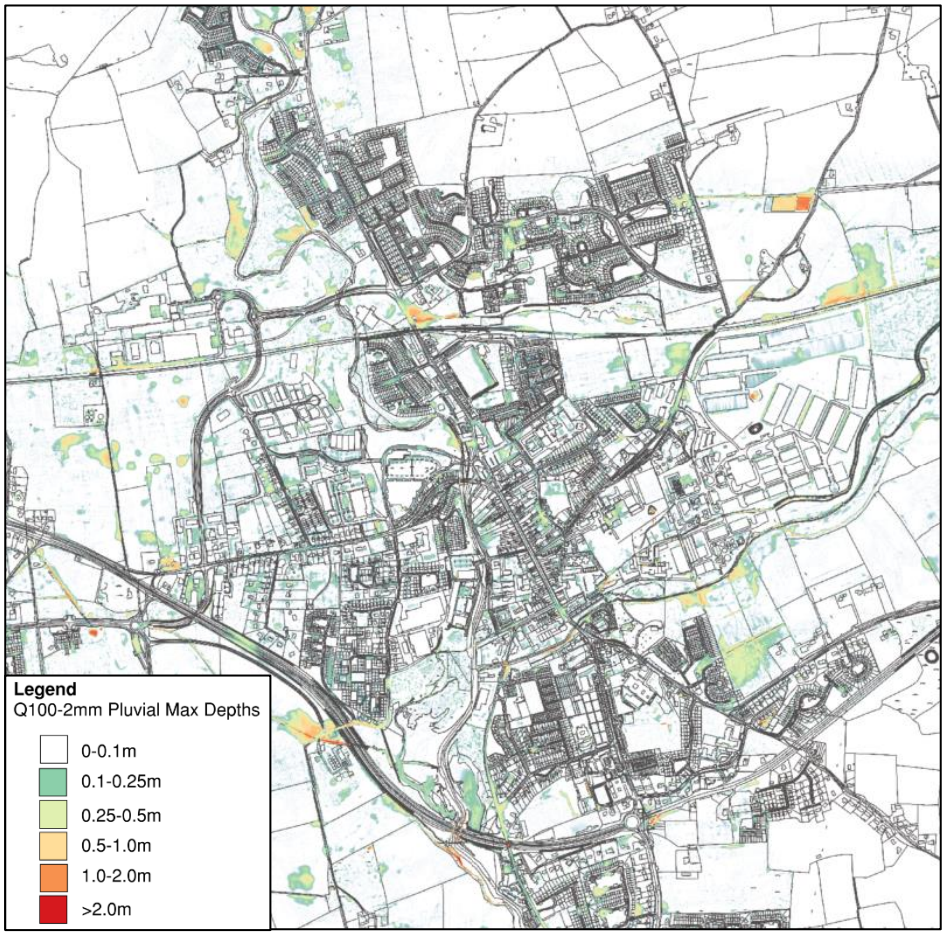


Figure 10-2 Pluvial Flood Risk Map

Measures to address this risk of pluvial flooding are presented in the Options report.

10.4 Groundwater Flood Risk Maps

As noted in Section 1 of the report, we have adopted the December 2015 event as a proxy for the groundwater design event. The groundwater flood risk map is therefore based on the areas which were inundated during the December 2015. The two main groundwater flood risk areas presented on the map are:

- Area in the vicinity of the Rugby Club and Lauriston Estate;
- Area in the vicinity of Water Rock House.

During significant groundwater events, the Cave System reaches or exceeds its conveyance capacity, restricting inflow from the Water Rock Stream which leads to an inundation of flood water in the vicinity of Water Rock House and causes very significant backwatering upstream of the Cave System. When the water level exceeds the level of the local access road, water can flow overland in an easterly direction.

As the source of this overland flow route originates from the Water Rock Stream it is therefore classified as fluvial flooding on the flood maps. However, the flooding mechanism is driven by blockage of the Cave System during a groundwater flood event. As such, the route is fluvial in source but groundwater-driven in the vicinity of the Cave System.

11. Conclusions

A dynamic 1D/2D hydraulic model of the relevant watercourses associated floodplains in Midleton was developed as part of the study in order to assess flood risk across the study area. The model simulated a range of combined fluvial/tidal design flood events for the current scenario for both fluvially dominant and tidally dominant scenarios. Two future scenarios (MRFS and HEFS) were also considered.

The findings of the hydrological assessment undertaken as part of the study were used to define the fluvial inflows into the models. Tidal water levels for the downstream boundary of the model are based on design tidal levels estimated by the Lee CFRAM Study at Roches Point propagated into the Owenacurra estuary using a two-dimensional model of Cork Harbour.

The 1D/2D hydraulic model was calibrated/validated against the significant flood event that occurred in December 2015 and also against in bank events from April 2018 and December 2018. Overall, a very good match was achieved between the modelled and measured results. The performance of the model at the peak of both of the 2018 events falls within the OPW's specified tolerance of +/-100mm and SEPA's 'high confidence' tolerance of +/- 150mm confidence. This tolerance is also achieved at a number of locations for the 2015 event. There are however a few exceptions such as at downstream of Moore's Bridge where the model underestimates water levels by circa 0.5m over a short length of the Owenacurra. This underestimation however is likely to be due to a blockage in the channel which occurred during the event, and which is not represented in the model.

A sensitivity analysis on the calibrated and validated model was also undertaken as part of the study.

Generally, it was found that the model is not particularly sensitive to changes in the model parameters. The only notable sensitivity relates to the flood risk to Thomas Street from the Owenacurra which was shown to be at risk in the Q100 event when a 4m grid resolution model was tested.

The model was then validated against the 2023 October (Storm Babet) event which was the largest fluvial flood event on record in Midleton. The model results compared well to the observed flood extents and recorded wrack marks. When the calibration/validation for the four events is considered in the round, it is evident that the model is suitable for use in order to simulate design flood events across the scheme area.

Fluvial and tidal flood maps were produced from the model and highlight the flood risk areas in Midleton/ Ballinacurra and Water Rock. Large areas of Midleton are at risk from both fluvial and tidal flooding. Approximately 510 residential and 190 commercial properties are within the modelled Q100/T200 flood extents.

A table of the number of properties inundated for each of the climate epochs is presented below.

Table 11-1 No of properties inundated across the Epochs for the entire Scheme area

Epoch	AEP	Residential	Commercial	Total
Current	10% (F & T)	84	66	150
Current	1%F & 0.5%T	516	192	708
Current	0.1% (F & T)	862	279	1141
MRFS	10% (F & T)	220	105	325
MRFS	1%F & 0.5%T	781	250	1031
MRFS	0.1% (F & T)	1044	323	1367
HEFS	10% (F & T)	503	134	637
HEFS	1%F & 0.5%T	1072	299	1371
HEFS	0.1% (F & T)	1135	329	1464

Pluvial flood risk across the scheme area was also assessed as part of the study. Much of the existing drainage network in the town centre is undersized and consequently, some low-lying areas of the town are at risk of surface water flooding for low period events. Additionally, there are also a number of areas within the scheme area risk of pluvial flooding.

There is a risk of groundwater flooding in the area in the vicinity of the Rugby Club/ Lauriston Estate and in the vicinity of Water Rock House.

The findings of the hydraulic modelling and assessment will be brought forward and considered as part of the optioneering for the scheme.