ARUP

Cork County Council

Midleton Flood Relief Scheme

Hydraulics Report

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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
FSR	Flood Studies Report
LiDAR	Light Detection and Ranging
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 will be detailed in the subsequent 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 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 the hydraulic model against historic flood events (December 2015, April 2018 and December 2018);
- 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 scenario;
- 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 Stream.

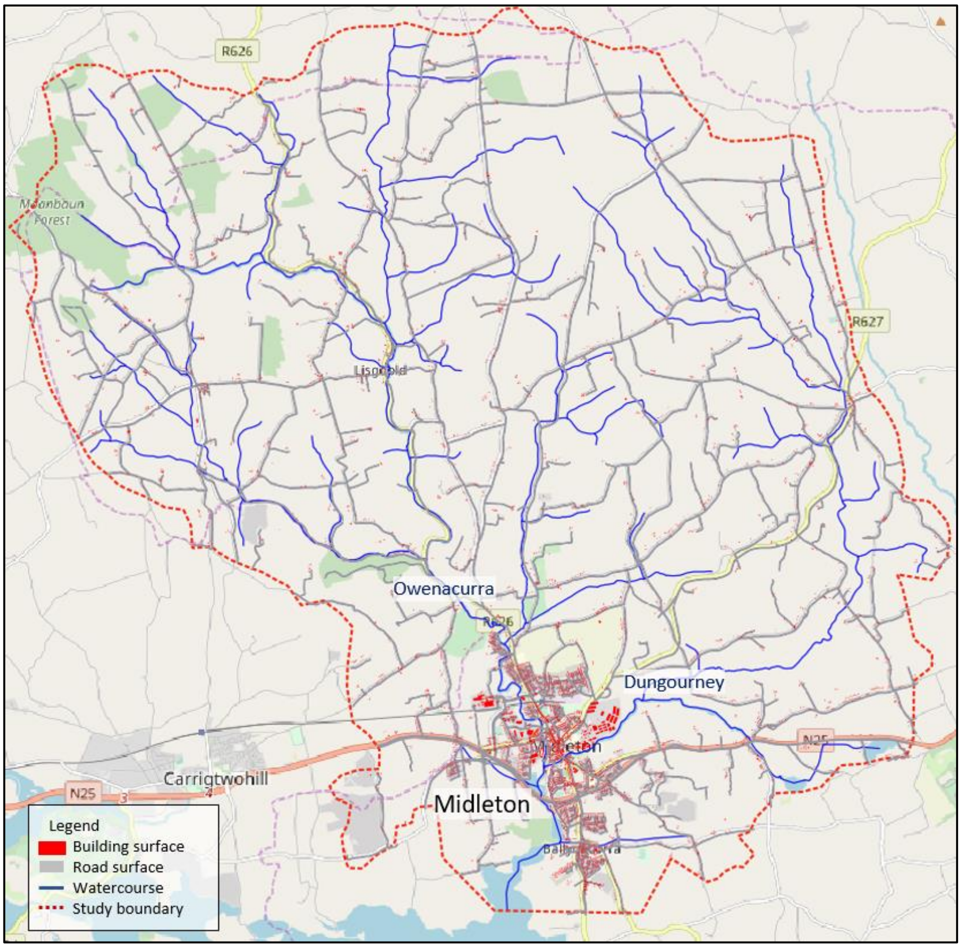

Figure 1.1 Study Area (© Open Street Map)

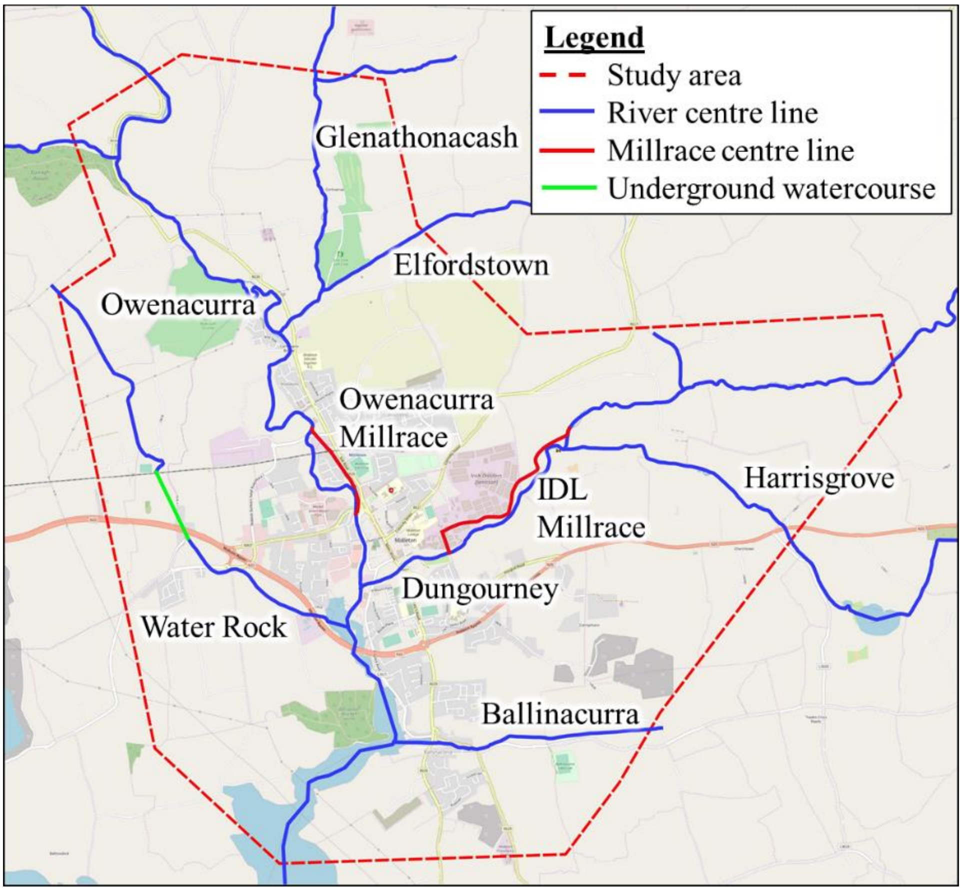

Figure 1.2 Scheme Area (© Open Street Map)

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 the table. Figure 1.2 highlights 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	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 have confirmed with CCC/OPW that the target SoP is to include an allowance for freeboard which will be determined as part of the Optioneering.

The SoP against both pluvial flooding and groundwater flooding is not specified in the brief. For pluvial flooding Arup have therefore adopted the same SoP as for fluvial flooding i.e. the 1% AEP plus freeboard SoP. 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 very extreme, we have adopted it 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. Section 3 outlines how both the hydrological estimation and hydrogeological assessment have been incorporated into the hydraulic modelling.

The development of the various elements of the model is described in Section 4 of the report while Section 5 presents the calibration of the model. An overview of the results design runs is presented in Section 6. 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 7 presents the increase in flood risk associated with climate change and discusses the mechanisms of flooding in each area associated with climate change scenarios.

Section 8 of the report presents the finding of the Sensitivity Analysis runs. The methodology used to derive both the pluvial and groundwater flood maps are presented in Section 8.6.

The overall conclusions of the hydraulic modelling are presented in Section 10.

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:

- Lee CFRAM survey data;
- Midleton FRS Infill and validation survey;
- Detailed culvert survey.

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

2.2.1 Lee CFRAM 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, 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.

2.2.2 Midleton FRS Infill and validation survey

As part of the review and quality assurance of the Midleton Lee CFRAM 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 CFRAM 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.

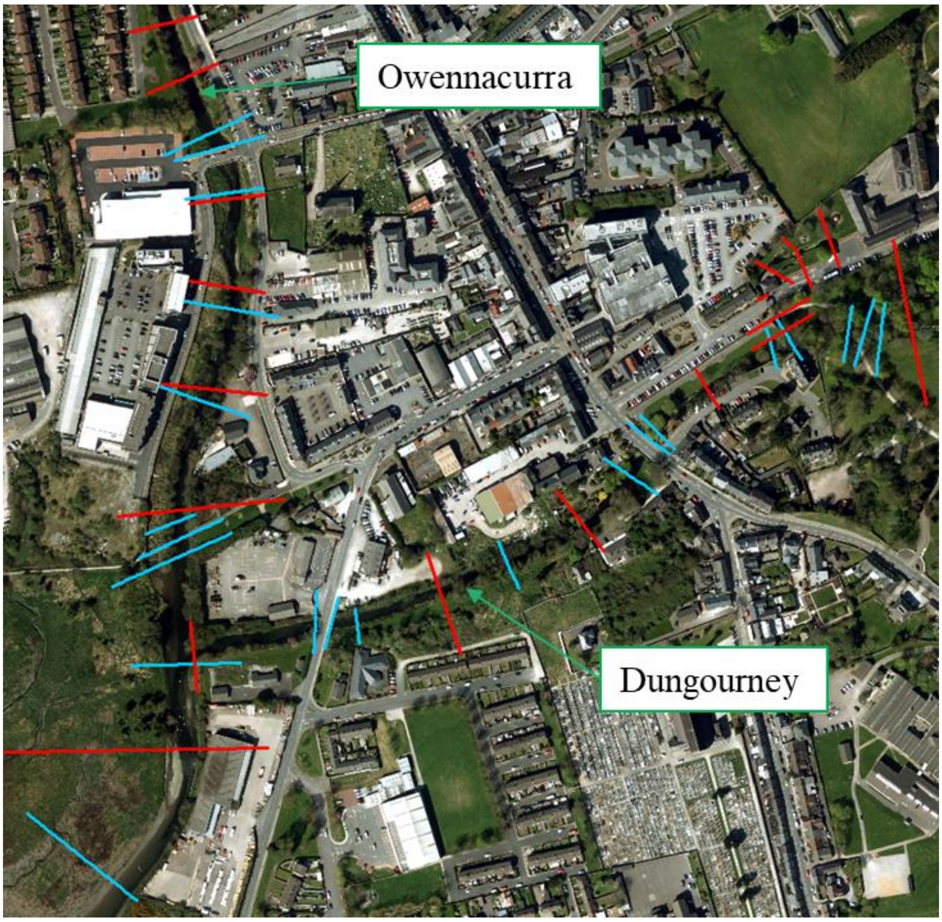

Figure 2.1 Midleton FRS Hydraulic model cross sections

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.

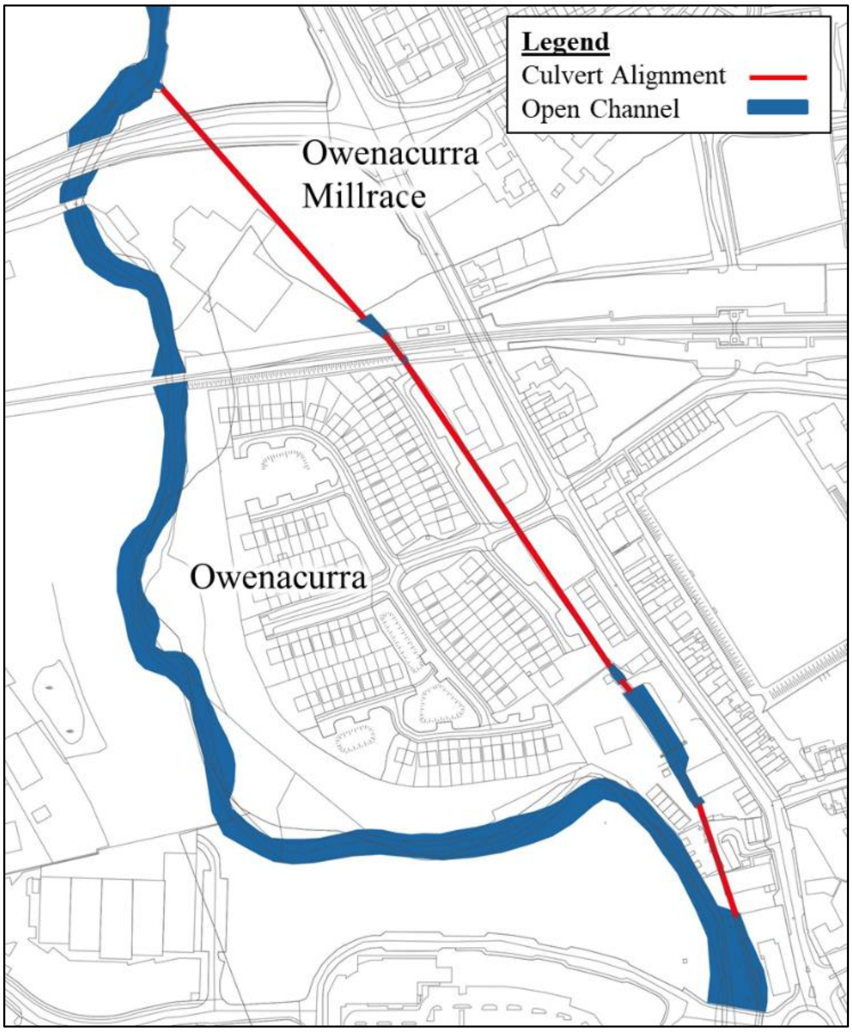

Figure 2.2 Owenacurra Millrace alignment

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 undertaken 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.

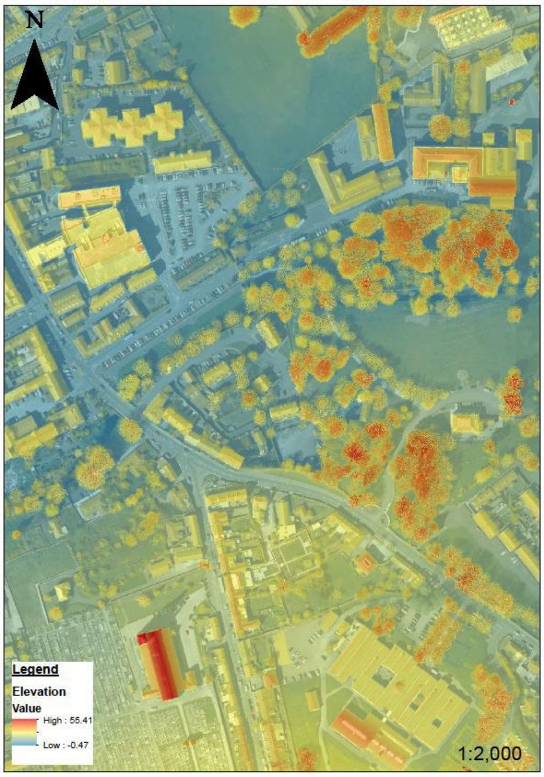

Figure 2.3 Sample Lidar data from the Midleton dataset

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[1].

Various data is therefore available from the event, including;

- 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 5 of this report.

2.5 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);

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.

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

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 marked up 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.

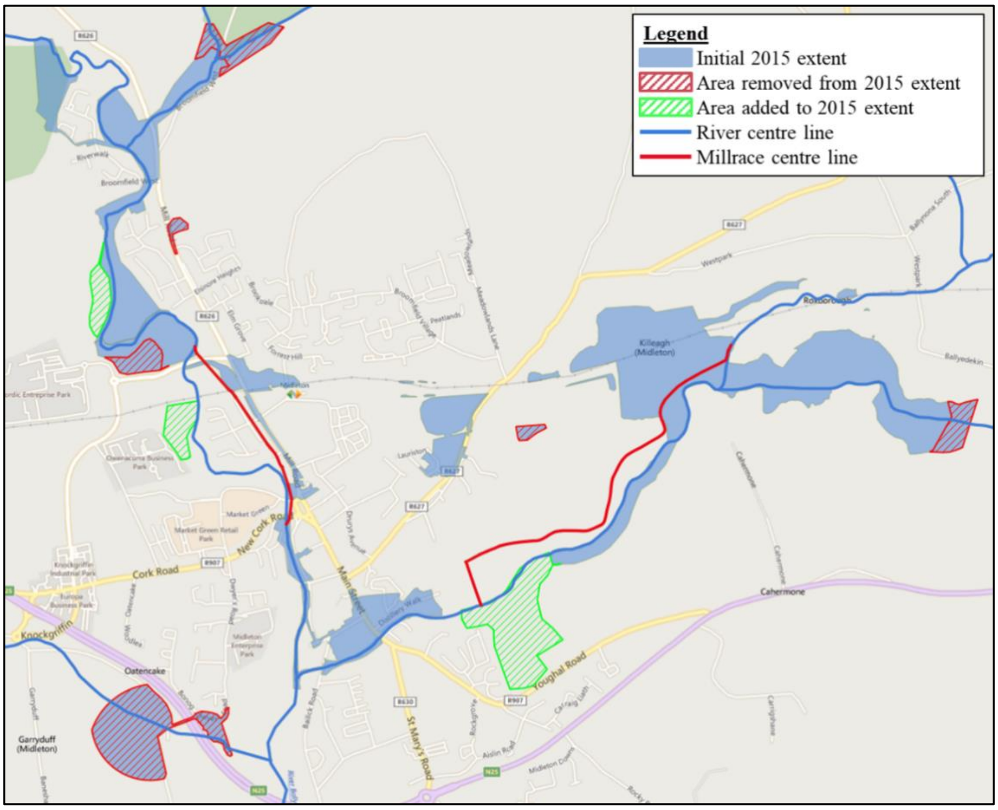

Figure 2.4 December 2015 estimated extent: initial and revised (© Bing Roads Map)

Water Rock House and its surrounding area was extensively flooded during the December 2015 event. An estimate of the flood extent in this area was not provided to Arup by CCC and is therefore not shown on Figure 2.4.[2] The L3619 road (Ballyvodock Road) in the Water Rock catchment was also inundated during the event but is also excluded from the flood extent map.

2.6 Long term time series of water levels in the Owenacurra and Dungourney Rivers

As part of the Midleton FRS project, Arup undertook a series of detailed hydrogeological assessments within the study area. This involved the collection of extensive site investigation data which included:

- Geophysical investigation compromising of seismic surveys, electrical resistivity surveys;
- Intrusive ground investigation including the drilling of cable percussion and rotary boreholes;
- Permeability testing of the sand and gravel aquifer;
- Long term monitoring of river and groundwater levels at a number of locations as illustrated in Figure 2.5.

[2] It is noted that the area in the vicinity of O'Dywer's Road/An Bonnog is not included in the extent as anecdotal data suggests that that area was not inundated during the event.

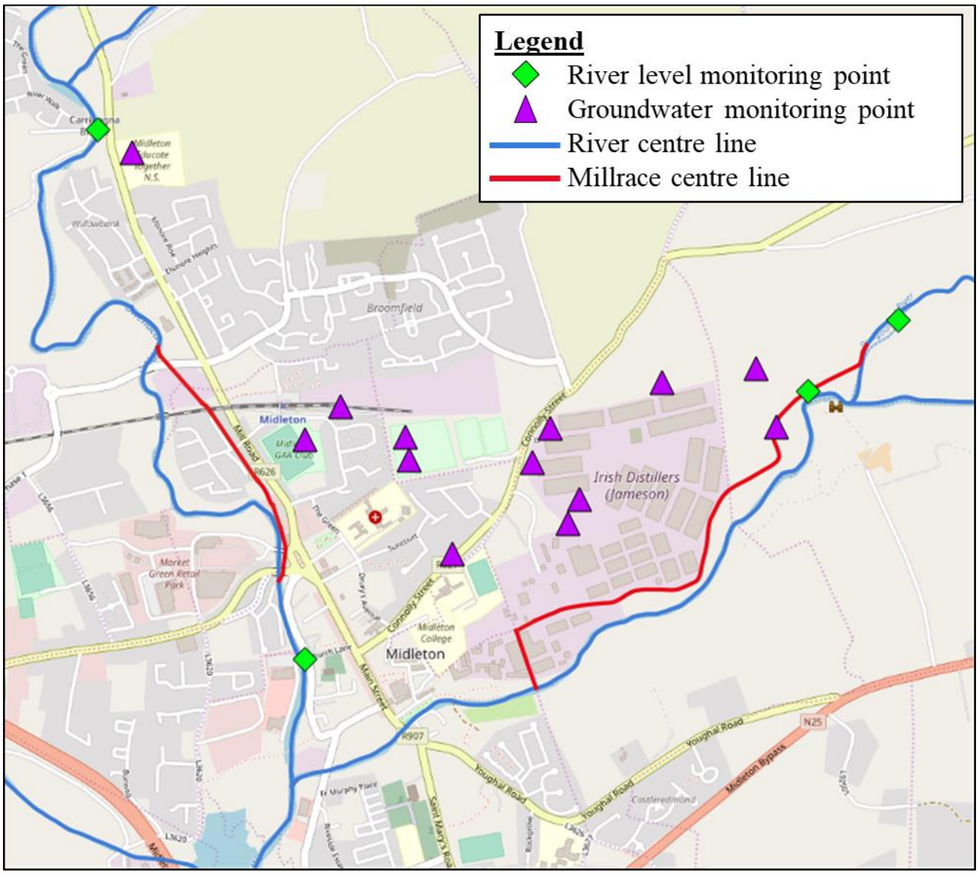

Figure 2.5 Locations of groundwater and river monitoring points (© Open Street Map)

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. 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 5 of the report.

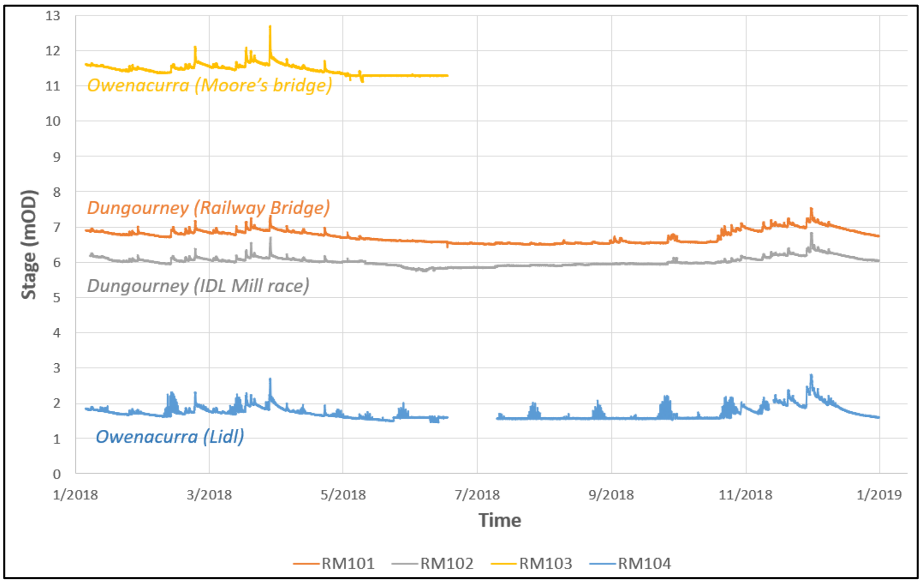

Figure 2.6 River monitoring data

3. Hydrological estimation and Hydrogeological assessment

3.1 Hydrological Estimation Undertaken as Part of the Study

A detailed hydrological analysis of the various contributing catchments has been undertaken as part of the study. 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 been used as input 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 into the hydraulic models.

3.2 Overview of the Hydrological Estimation Undertaken as Part of the Study

3.2.1 Design flow estimation

A detailed hydrological analysis has been 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 various Hydrological Estimation Points (HEP) in the study area (Figure 3.1).

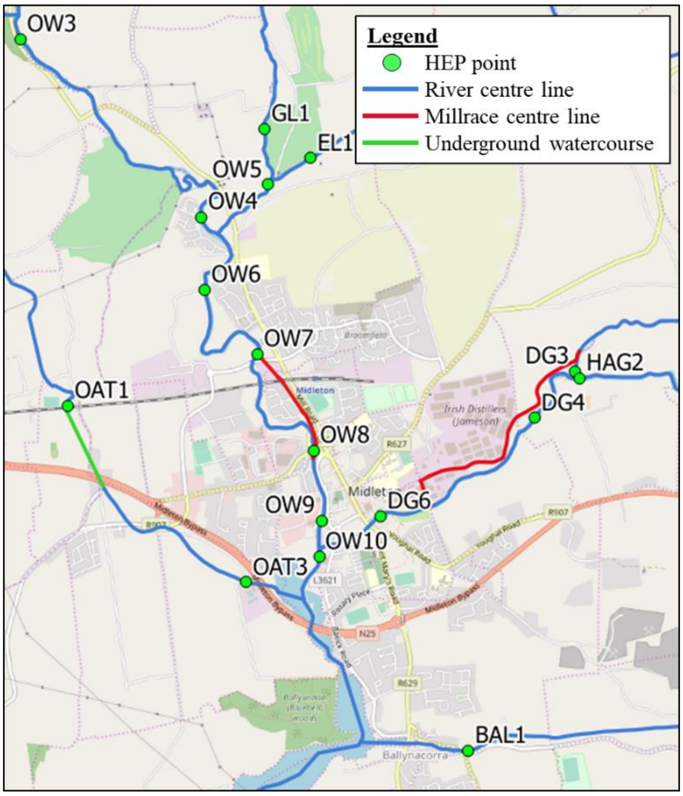

Figure 3.1 HEP points (© Open Street Map)

A set of index flood flow (Qmed) estimates were first 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 which included 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 24.46m^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. The adopted index flows are presented in the following Table 3-1.

Table 3-1 Index Flows

HEP Location	Index Flow (m^3/s)	HEP Location	Index Flow (m^3/s)
BAL1*	0.99	OW3	25.74
DG3	13.36	OW4	25.93
DG4	14.85	OW5*	7.54
DG6	15.48	OW6	31.63
EL1*	3.10	OW7	31.73
GL1*	4.89	OW8	32.35
HAG2*	3.61	OW9	32.72
OAT1*	1.88	OW10	45.78
OAT3*	3.56	* HEP located on small catchment	

A flood frequency analysis was carried out. This established a study growth curve and in turn a set of design peak flows. The adopted growth curve was produced using the Single Site Analysis at Ballyedmond 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 3-2. The design flows for the HEPs are presented in Table 3-3.

Table 3-2 Study Growth Curve

Return period (years	Study Growth Curve
2	0.97
5	1.29
10	1.50
25	1.76
50	2.30
100	2.59
200	3.01
1000	3.96

Table 3-3 Design Flows in m³/s

HEP Location	Return Period (1in _ years)							
	2	5	10	25	50	100	200	1000
BAL1*	0.96	1.28	1.49	1.74	2.28	2.56	2.98	3.92
DG3	12.96	17.24	20.04	23.52	30.73	34.60	40.22	52.91
DG4	14.40	19.15	22.27	26.13	34.15	38.45	44.69	58.79
DG6	15.01	19.97	23.22	27.24	35.60	40.09	46.59	61.30
EL1*	3.01	4.00	4.65	5.46	7.13	8.03	9.34	12.28
GL1*	4.74	6.30	7.33	8.60	11.23	12.65	14.70	19.34
HAG2*	3.51	4.66	5.42	6.36	8.31	9.36	10.88	14.31
OAT1*	1.82	2.43	2.82	3.31	4.33	4.87	5.66	7.45
OAT3*	3.46	4.60	5.35	6.27	8.20	9.23	10.73	14.11
OW3	24.97	33.21	38.61	45.31	59.21	66.67	77.48	101.94
OW4	25.15	33.45	38.89	45.63	59.64	67.15	78.04	102.68
OW5*	7.31	9.73	11.31	13.27	17.34	19.53	22.70	29.86
OW6	30.68	40.80	47.45	55.67	72.75	81.92	95.21	125.26
OW7	30.78	40.94	47.60	55.85	72.99	82.19	95.52	125.67
OW8	31.38	41.73	48.53	56.94	74.41	83.79	97.38	128.11
OW9	31.74	42.21	49.08	57.59	75.26	84.75	98.49	129.58
OW10	44.41	59.06	68.67	80.57	105.29	118.57	137.80	181.29
* HEP located on small catchment								

3.2.2 Hydrograph Shape Analysis

Two different methods have been used to estimate the design hydrograph shape:

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 has been adopted as it provides 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.

The reader is referred to the hydrology report for a detailed analysis of the hydrograph shape analysis.[3]

3.2.3 Small Catchment Hydrology Sensitivity

As part of their review of the draft hydrology report for the study, OPW requested Arup to undertake a sensitivity analysis on the small catchment hydrology by considering an alternative and more conservative hydrological estimation method to the IH124. Arup therefore 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 8 of this report.

3.3 Integrating the Design Flows into the Hydraulic Model

3.3.1 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 reach 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 3.2. It can be seen from the figure that the Q100 design flow for OW4 is 67.15m^3/s. This flow estimate was used as the Q100 inflow for HEP OW3 which is located upstream.

Figure 3.2 Inclusion of HEPs in the hydraulic model (© Open Street Map)

3.3.2 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 overestimation of flows downstream of confluences and consequently an overestimate of design water levels in a reach. As can be seen from

[3] Midleton FRS Hydrology Report, Arup 2022

Table 3-3, the design flows in the study area from one or more sub catchments are generally within 5% of the design flow for the whole catchment.

To further illustrate this, Figure 3.3 presents the 1% AEP design flows at the Owenacurra/Dungourney confluence. It can be seen from the figure that the Q100 downstream of the confluence is 118.57m^3/s while the sum of flows upstream of the confluence is 124.84m^3/s (84.75m^3/s + 40.09m3/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 will therefore be overestimated by 4.8%.

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 4.8%. This assumption will be revaluated as part of the optioneering to ensure its conservatism does not result in flood relief measures that fail to meet any economic, social or environmental criteria.

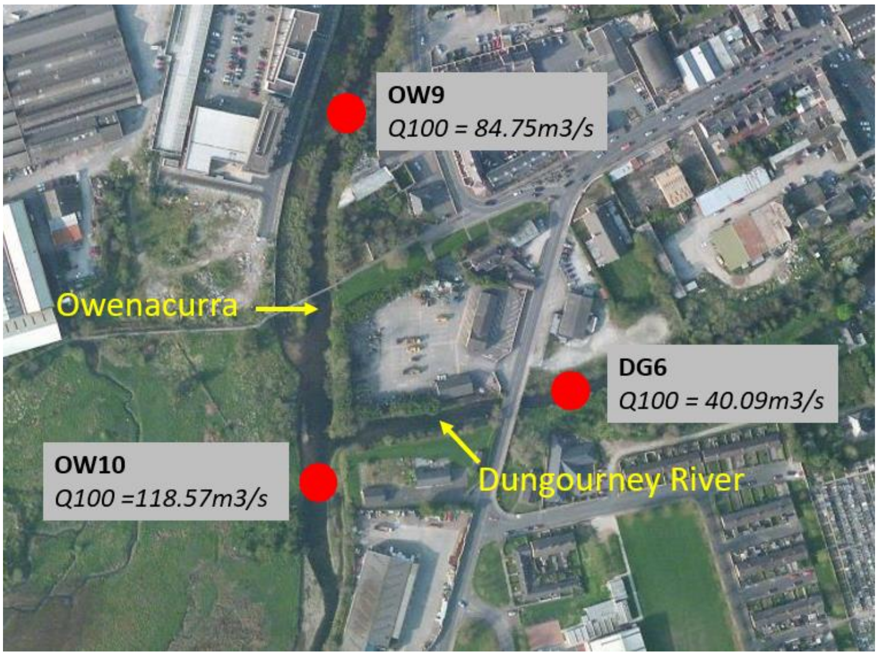

Figure 3.3 Design Q100 flows at the Dungourney/Owenacurra confluence

3.3.3 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 the run.

3.3.4 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.

3.3.5 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 has utilised recorded tidal water levels the Port of Cork Gauge at Cobh from the event as the downstream boundary. 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 – 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;

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 used to calculate the design water levels in the Owenacurra estuary.

The peak tidal water levels for the Owenacurra estuary are presented in Table 3-4. The reader is referred to the project hydrology report for further detail on the design tidal water levels.

Table 3-4 Peak Tidal Water Levels

Design Event (AEP)	Peak Tidal Levels in Owenacurra Estuary (mOD Malin)
50%	2.37
20%	2.48
10%	2.55
4%	2.64
2%	2.71
1%	2.78
0.5%	2.84
0.1%	3.00

3.3.6 Urban Drainage Network

Arup reviewed all the available existing data and reports on the Urban Drainage Network in Midleton which included a detailed report by Byrne Looby from 2015.[4] As part of their study, BL 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[5] was incorporated into the model in order to infill the data gaps as previously identified by Byrne Looby. 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.

3.3.7 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.

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

[5] 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.

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 in the work in detail.

The fluvial/tidal scenarios for the existing scenario in Midleton are tabulated in Table 3-5 (Fluvial dominated) and

Table 3-6 (Tidal dominated) below. The scenarios highlighted in red represent the required standard of protection the scheme, i.e. the preferred flood relief option for Midleton will be required to defend up to and included these fluvial/tidal events.

Table 3-5 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 3-6 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

3.4 Hydrogeological assessment

As part of the Midleton FRS project, Arup was commissioned by Cork County Council to undertake 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 3.4 below.

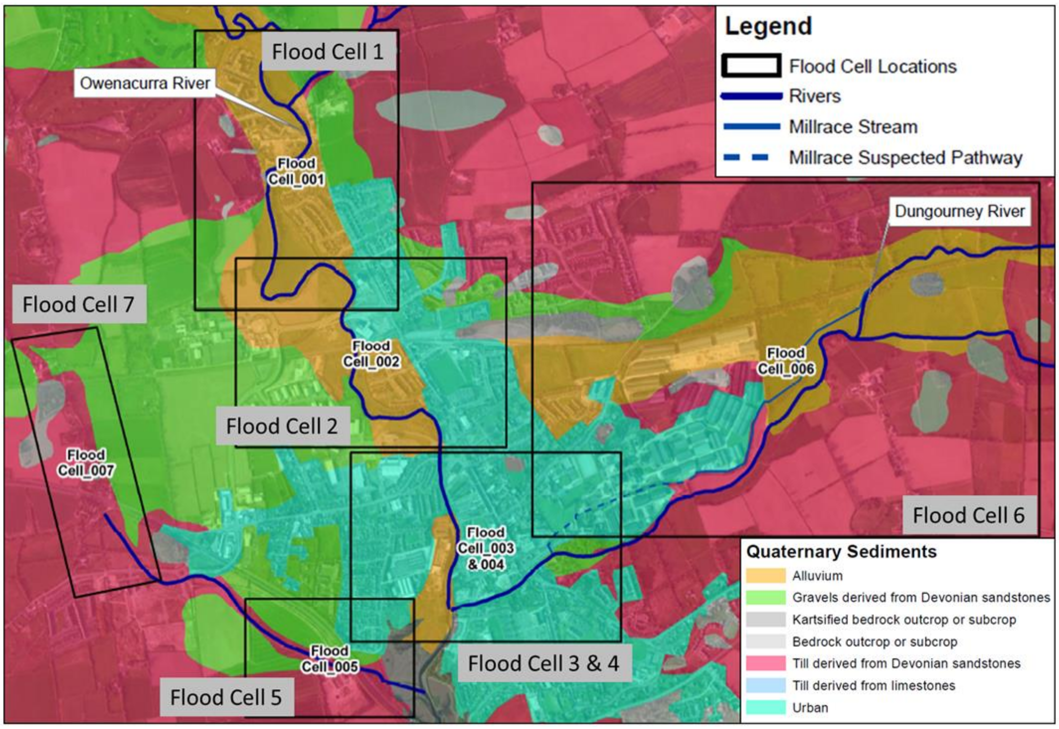

Figure 3.4 Flood cells

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

3.4.1 Flood Cell 2, 3 and 4

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

3.4.2 Flood Cell 1

The results of this assessment concluded that the groundwater contribution to the Owenacurra River is negligible. Additional inflows to the river from groundwater sources is therefore not required.

3.4.3 Flood Cell 5 and 7

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

3.4.4 Flood Cell 6

Analysis of the river water level and groundwater monitoring data collected in Flood Cell 6 indicate that the Dungourney River loses water to the gravel and limestone aquifers in this area. The follow key points highlight the groundwater-surface water interactions:

The hydrogeological conceptual model highlights that river is in hydraulic connection with the gravel aquifer and discharges water into the aquifer during peak flow conditions. The limestone aquifer is semi-confined with groundwater levels that are lower than in the overlying gravel aquifer and the river.

The groundwater elevation in both the gravels and limestone aquifers are consistently below that of the river water level, indicating that the river is losing flow to the gravel aquifer, rather than the reverse.

There is a time lag from when the maximum water level occurs in the river, followed by the maximum that occurs in the gravels or limestone demonstrating the aquifer responding recharge which may be from the river.

The response in groundwater level in the gravel aquifer dampens with distance from the river and also vertically which demonstrates the influence of aquifer storage effects as water travels from the river through the aquifer.

These factors are of importance as they indicate that the groundwater is not a contributing factor to high water levels seen in the river, but rather the Dungourney River is losing water to the aquifer, even during high rainfall events.

The groundwater is flowing from east to west through the Lauriston Mews/Midleton Rugby Club study area, as is demonstrated by the ripple effect of peak water levels emanating from the Dungourney River towards the Owenacurra River and the gradient observed across groundwater level monitoring wells.

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.

4. Model Development

4.1 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.

4.2 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 CFRAM 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 CFRAM 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 CFRAM model was represented by reservoir units and parallel channels all of which has 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 CFRAM model have been included as part of 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 CFRAM model. Additionally, Arup also identified a number of areas where modifications to the river channels (and floodplain) have occurred since the Lee CFRAM 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 CFRAM model were altered in the Midleton FRS model. These include channel roughness and structure coefficients which are described in Section 4.4 of this report.

4.3 Model Extents

4.3.1 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 4.1. The 2D model domain is represented with the green shading in the figure and it can be seen that it covers the main urban area of Midleton. 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.

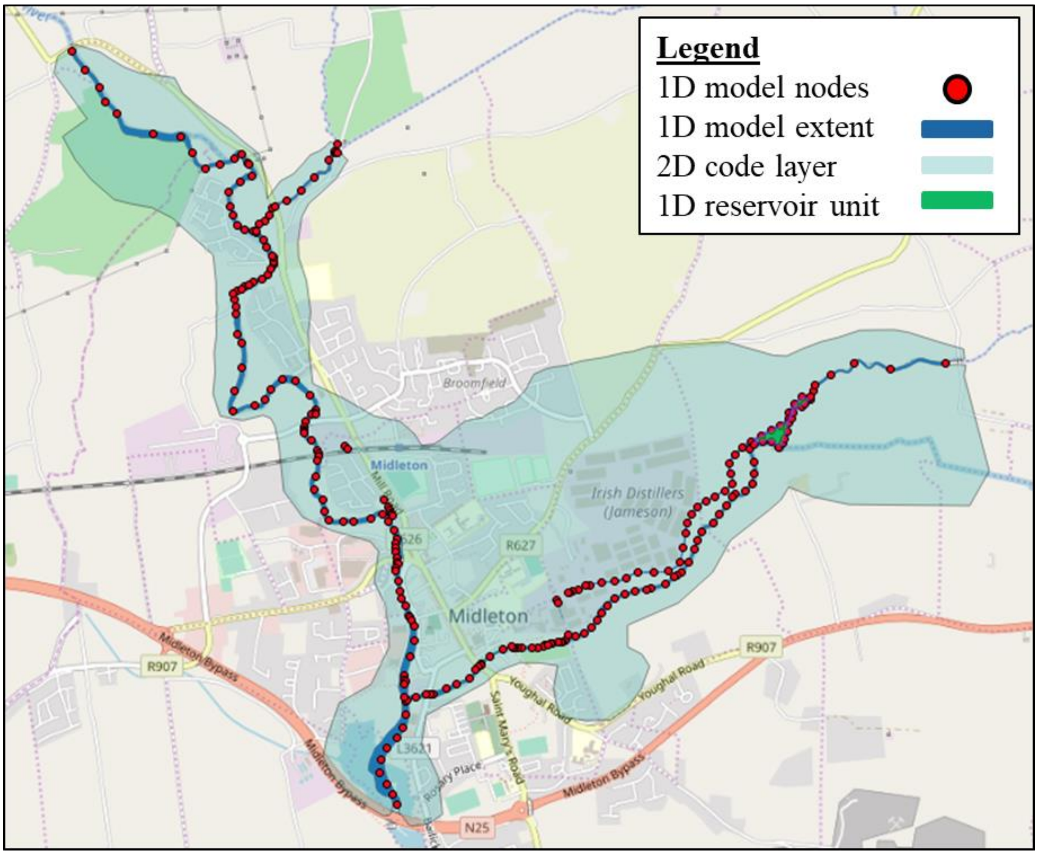

Figure 4.1 Schematic of the Midleton FRS model (© Open Street Map)

Separate hydraulic models of the Ballinacurra and Water Rock watercourses have also been development as part of the study. The Water Rock Model is a 1D/2D model while the Ballinacurra model is a standalone 1D only model. Schematics of these models are presented in the following two figures. A full detailed schematic of each of the hydraulic models is included in the appendices.

Figure 4.2 Ballinacurra Hydraulic model (© Open Street Map)

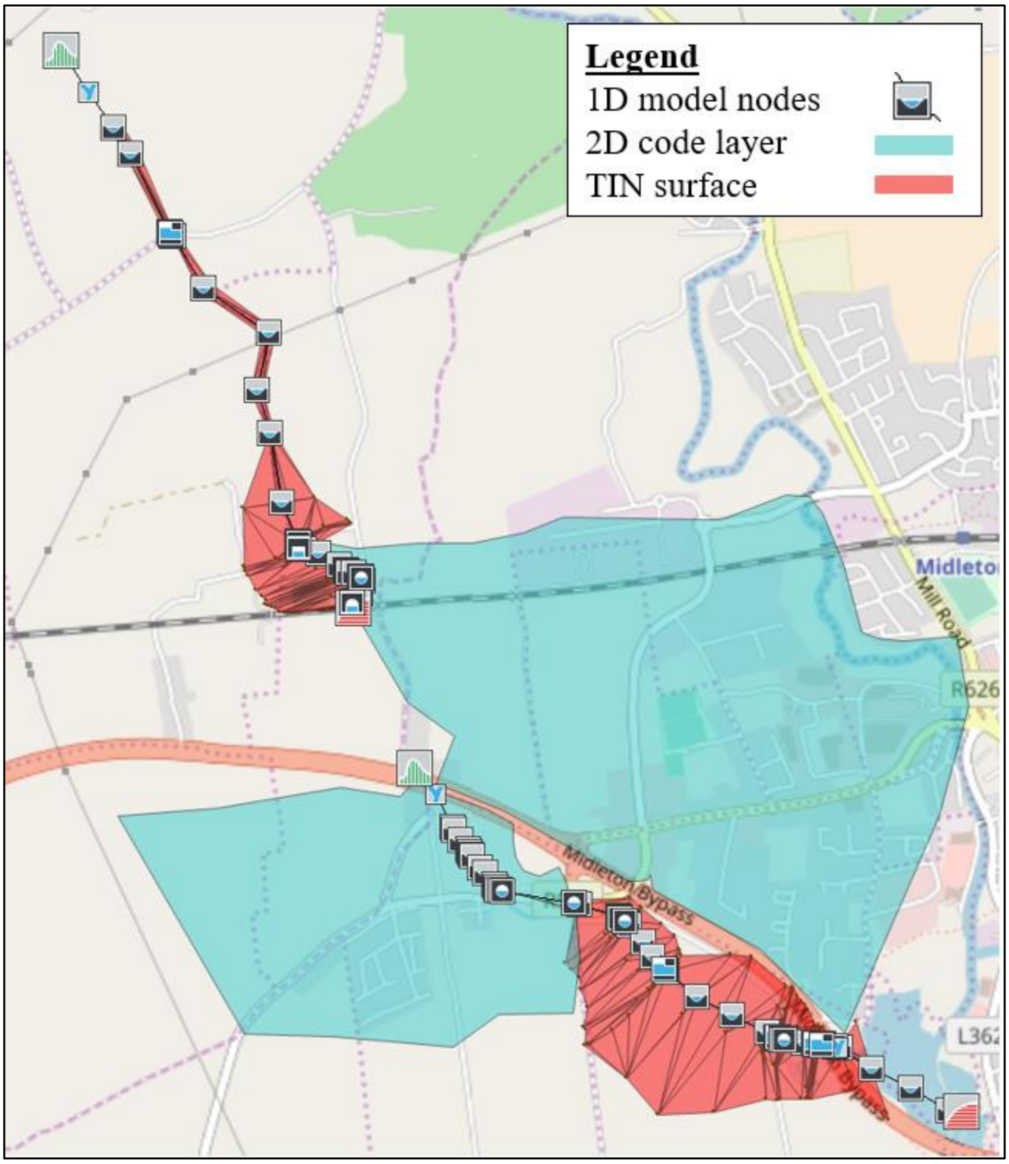

Figure 4.3 Water Rock Hydraulic model (© Open Street Map)

Figure 4.4 presents a schematic of the 1D components of each of the models.

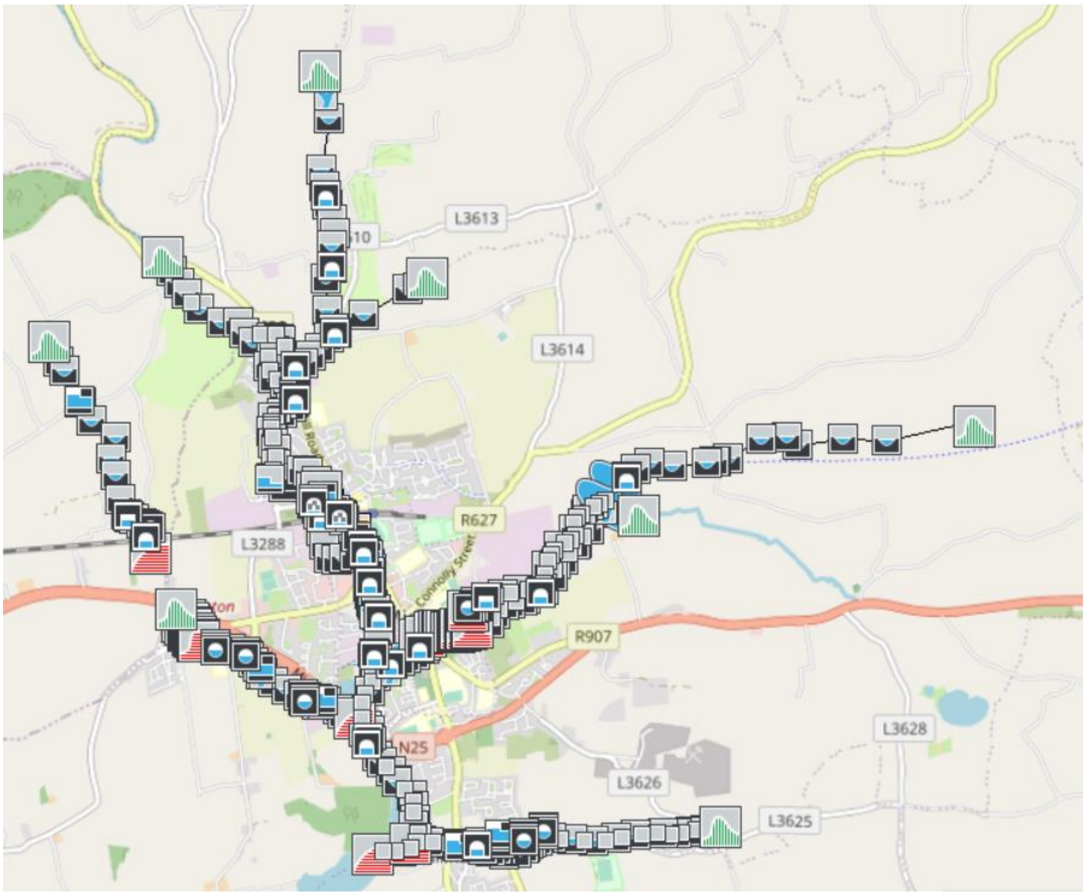

Figure 4.4 1D elements of all the hydraulic models

4.4 Model Parameters

4.4.1 Labelling System

The model nodes derived from the infill survey followed the same labelling format as used for the Lee CFRAM 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. This approach also ensures that a direct reference can be made to both the CFRAM 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).

4.4.2 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.

4.4.3 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 of the 1D model were selected based on a detailed analysis of a number of datasets as follows:

- Model calibration (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 4-1 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.

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

Table 4-2 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 8 of the report.

4.4.4 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 relative size. Overtopping of the bridges has been accounted for in the 2D domain with the exception of the Moore's Bridge which has been modelled using a spill in the 1D model.

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 8 of the report.

4.4.5 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 a 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 represented.

The approach is deemed accurate and appropriate given that a significant number of buildings were inundated during the 2015 event. It is noted that the 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 the walls and embankments as indicated in Figure 4.5 have been included in the model as there is no record of any of the structures having collapsed during the event.

For the design runs up to and including the Q50 event, it has been assumed that these walls and embankments will not fail. For the Q100 and Q1000 events however the walls and embankments have been excluded from the model as their structural integrity cannot be guaranteed during the design event given that none of them were designed as formal flood defences structures.

It is noted that the heights of each wall have been estimated from site visits.

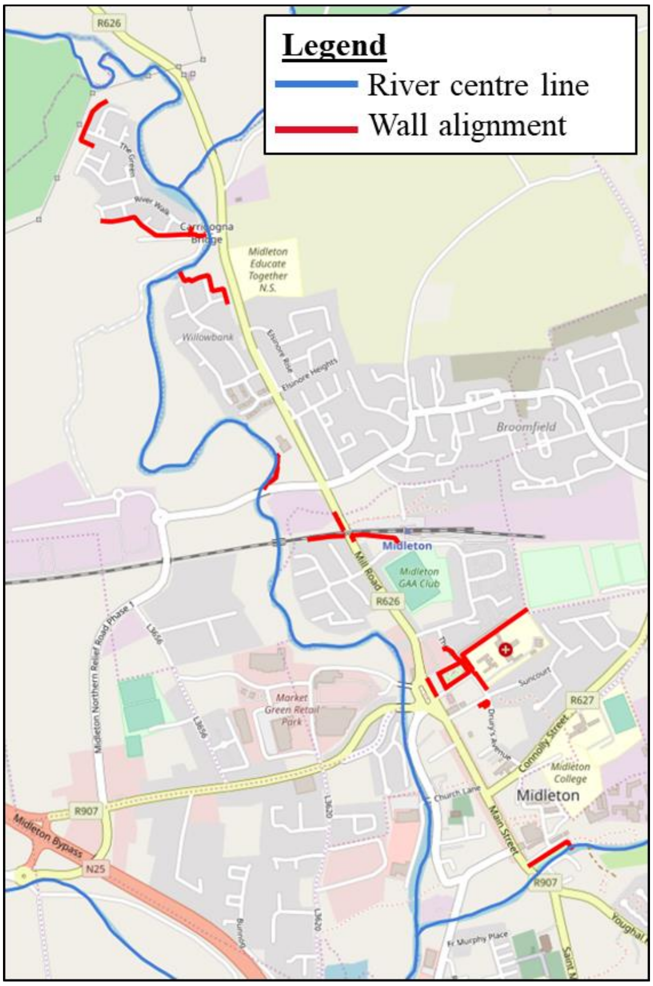

Figure 4.5 Walls and embankment Z lines (© Open Street Map)

4.4.6 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 which in turn impacts on water levels in the Water Rock stream upstream of the Cave. Other sources of uncertainty include:

- The irregular and unknown geometry of the entrance to the cave (shown in Figure 4.6);
- The unknown conveyance capacity of the sink hole at the entrance to the Cave which allows water to ingress into the cave.

Figure 4.6 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 5 for details on the calibration.

4.5 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 credible 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.

4.6 Hydraulic Modelling of the Options

The Midleton FRS model will be modified to model the various flood relief options considered as part of the development of the scheme. This element of work is described in the Options Report.

5. Model Calibration

5.1 Introduction

5.1.1 Overview

The Midleton hydraulic model has been calibrated and validated against three separate flood events:

- December 2018 event – model calibrated against measured water level timeseries on the Owenacurra river;
- April 2018 flood event – model validated against water level timeseries on the Owenacurra river;
- December 2015 flood event – model calibrated against flood event data collected by various authorities during the December 2015 flood event;

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.

5.1.2 Blockage assumptions in calibration model

It was assumed initially that all bridge and culvert units were unblocked as part of the calibration hydraulic model run. As described in the following sections however, this assumption was revised for both Moore's Bridge and the culverts along the Owenacurra Millrace.

5.1.3 Model tolerance

The Midleton FRS brief states that the consultant for the project "shall clearly demonstrate the calibrated models level of accuracy against historical events". The brief does not however specify any quantitative tolerance of model performance.

The project briefs of more recent flood relief scheme tenders issued by the OPW in Ireland however do specify tolerances as regards the required accuracy of calibrated model. They state 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.

5.2 Calibration Hydrology

5.2.1 2018 flood events

The Ballyedmond gauge was operational throughout 2018. Recorded water levels and flows (derived from the revised rating curve) 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 the calibration hydraulic models. Inflows for all the other 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 5-1). The uncertainty associated with this approach is noted given the spatial and temporal variation in rainfall and groundwater baseflow across the various catchment. Given the relative size and proximity of the various catchments to each other however and in the absence of any other data for any of the other tributaries for this period, this method is deemed to be appropriate.

Table 5-1 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

5.2.2 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. We therefore cannot state with high confidence what the peak flow on any of the watercourses are likely to have been for the event.

We have addressed this uncertainty in the study 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 5-2). 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.

Table 5-2 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

5.2.3 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 5-3. In order to assess the approximate return period of each event, the design flows for HEP OW3 (also located at the upstream end of the scheme area) are presented in Figure 5.1. By comparing the peak calibration flows against the 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 approximately to a circa Q45 year event.

Table 5-3 Magnitude of calibration events

Event	Calibration Data	Type	Estimated Peak Flow(m^3/s)	Return Period (1 in _ years)
December 2015	Wrack Marks / Observations	Calibration	43-53	~25–45
December 2018	Water Levels recordings	Calibration	27	~3
April 2018	Water Levels recordings	Validation	27	~3

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

Figure 5.1 Design flows for OW3

5.3 Calibration data for the 2018 flood events

The calibration/validation events of April 2018 and December 2018 are highlighted with dashed lines in Figure 5.2. A strong correlation between the water levels at each of the four locations is evident in the data across the full record period.

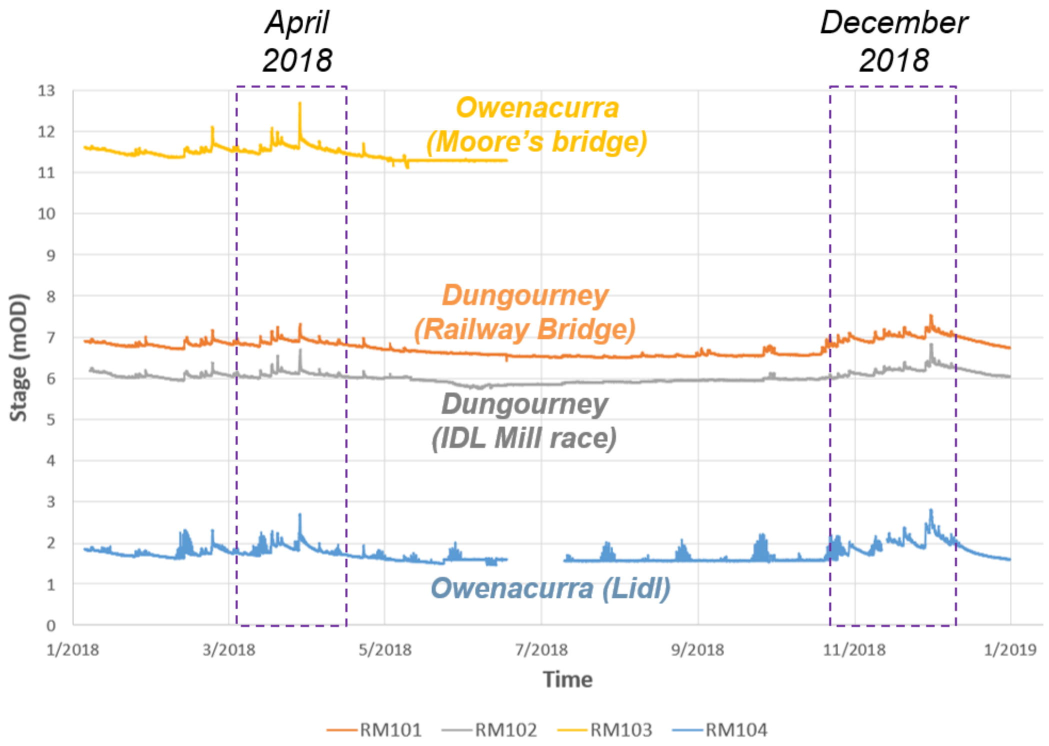

Figure 5.2 Recorded water level timeseries

5.4 2018 calibration/validation - Owenacurra River

5.4.1 December 2018 calibration at Lidl bridge

The water level calibration for the December 2018 event at the Lidl Bridge gauge is presented in Figure 5.2. 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 the event which as noted previously approximates to a 3-year return period 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 5.2 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.

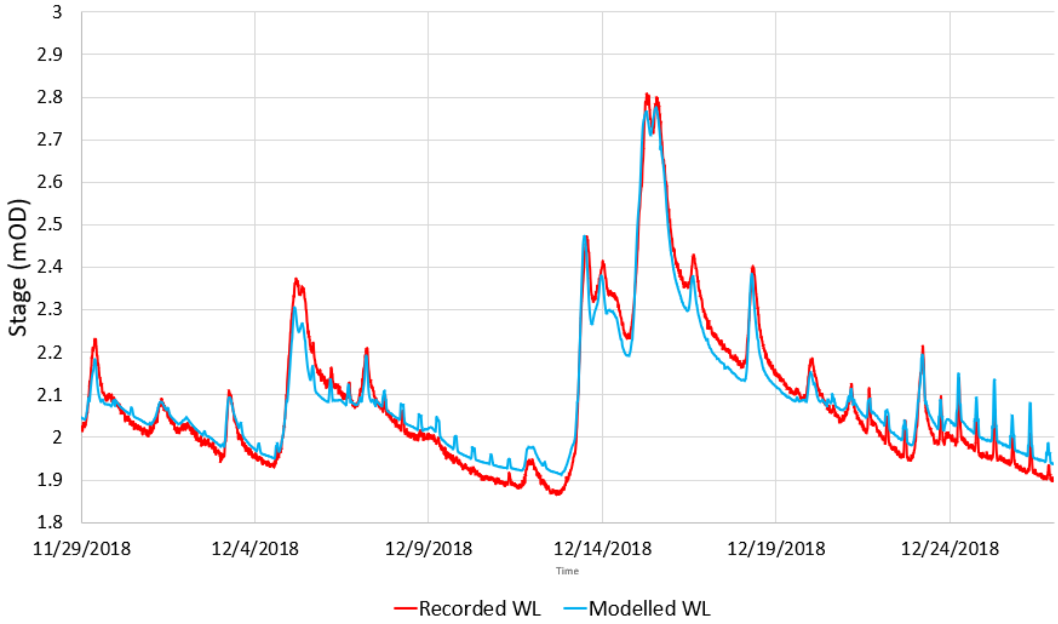

Figure 5.3 Lidl Bridge Water Level Calibration

We note that the recorded peak water level (circa 2.8mOD) is in-bank at this location. This is indicated in Figure 5.4 which plots the cross section at the Lidl bridge with the 2.8mOD water level indicated with the dashed red line.

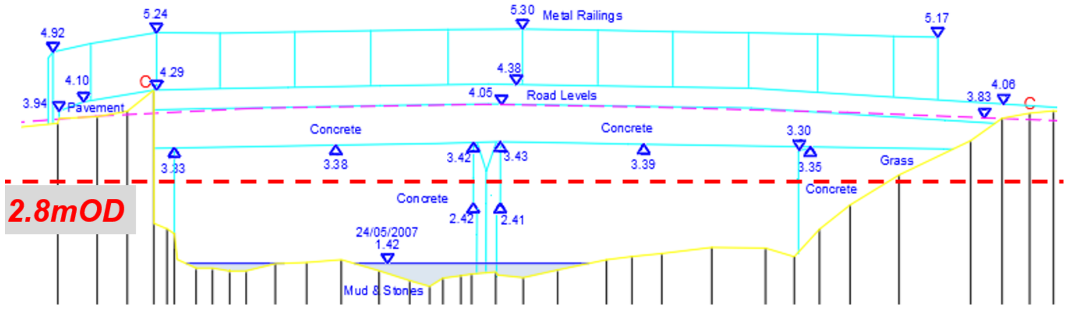

Figure 5.4 Cross section at Lidl bridge

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.

5.4.2 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 was run using the April 2018 flood event data. The water level validation plot is presented in Figure 5.5. 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.

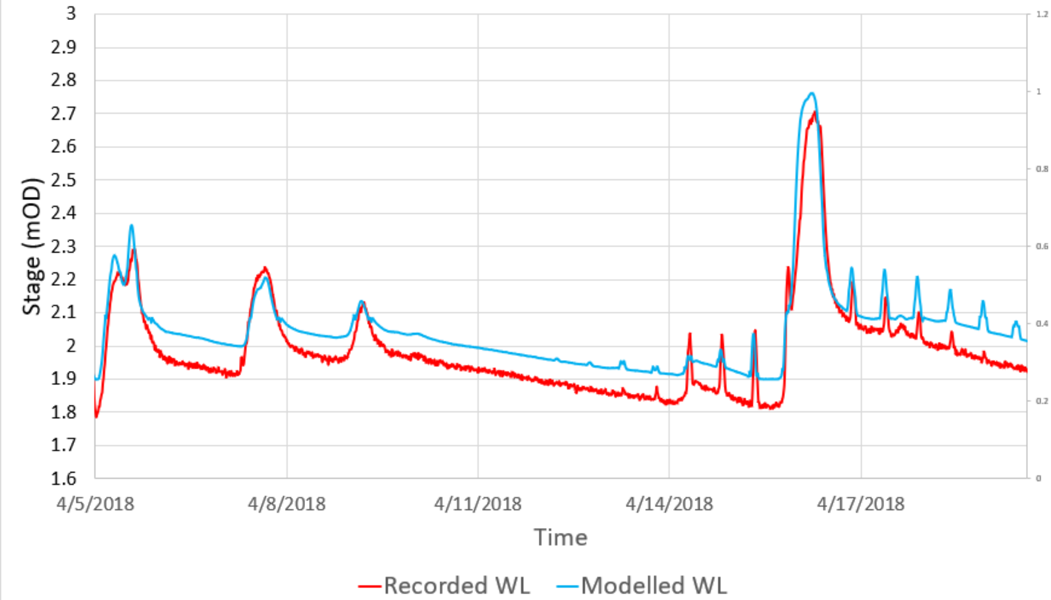

Figure 5.5 Lidl Bridge Water Level Validation - April 2018

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 5.5) 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.

5.4.3 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 allow for the accuracy of the model at this location to be demonstrated.

The model calibration at Moore's Bridge is presented in Figure 5.6. It can be seen from the figure 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.

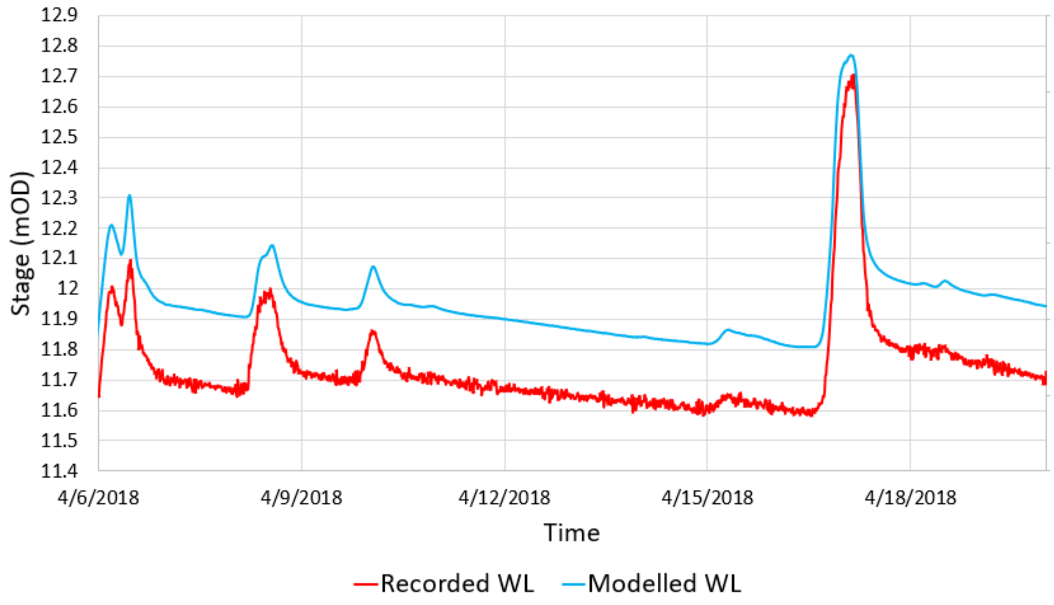

Figure 5.6 Moore's Bridge Water Level Calibration

The model overestimates low water levels by circa 200mm and there are a number of likely reasons as to why:

- The methodology for deriving inflows on the tributaries is approximate and may lead to uncertainties in the flows and 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 differences 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 all the primary courses.

It is noted that the parameters of the model could have been adjusted to improve the calibration at low flows i.e. the Manning's n value and head loss coefficients could have been adjusted to achieve a better match at low flows. This however would have reduced the accuracy of the model at high flows i.e. the model would not have achieved as good a calibration for the periods of high flows. This would therefore have reduced confidence in the model as a tool to assess flood risk in Midleton and was not explored. 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 5.7 presents the cross section at Moore's bridge with the peak observed water level from the April 2018 event superimposed on the plot. It can be seen that the peak water level was in-bank at this location during the event.

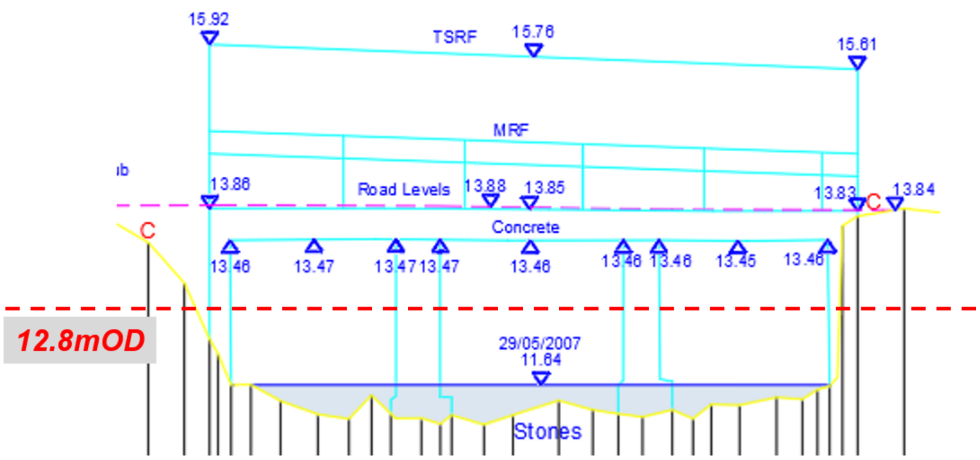

Figure 5.7 Moore's Bridge Cross Section

A statistical analysis for the validation simulation 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 that 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.

5.5 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 which has been used to calibrate the model.

An overview of the model calibration for the whole of the scheme area (assuming no blockages of any of the structures) is presented in Section 5.5.1. The model calibration for individual areas of the town are then assessed in detail in Section 5.5.2 to Section 5.5.8 which consider both unblocked and blocked scenarios.

As noted in Section 5.2.2, in order to address the uncertainty over inflows for the event[6] 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.

5.5.1 Overview of December 2015 calibration

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 ran 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 5.8 for the unblocked and low-end inflow scenario. It can be seen from the figure 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 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.

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

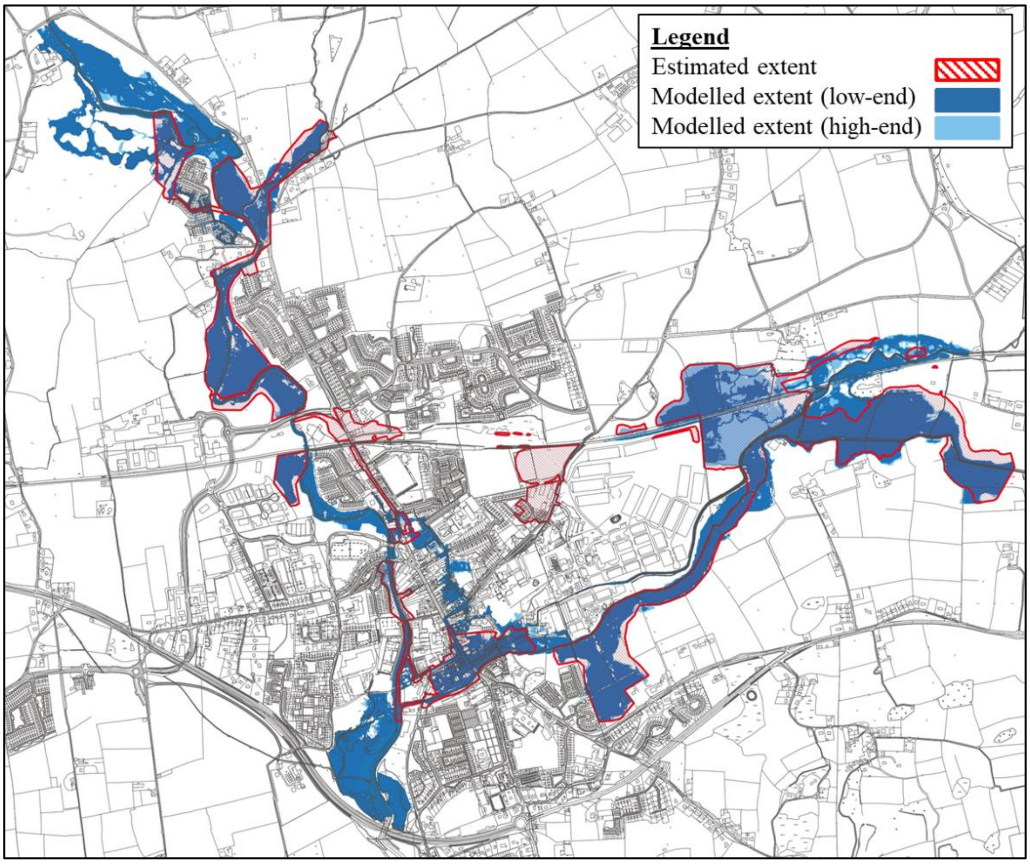

Figure 5.8 December 2015 flood event– maximum extents for entire area

Maximum water levels from the hydraulic model are compared against observed peak flood levels at a number of locations in the table below. The locations of these points are mapped out spatially in the zoomed in figures 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 5-4: 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

Location	Observed (mOD)	Modelled (mOD)	Comment	Model tolerance
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

5.5.2 Calibration at Tir Cluain/Moore's Bridge

Figure 5.9 presents the modelled and estimated maximum flood extent for the area of Tir Cluain and Moore's Bridge which we note are identical to the results presented in Figure 5.8 but are instead zoomed in on this area. It can be seen from the figure that the model captures the primary mechanism by which Tir Cluain was flooded during the event: water entering the estate from the Northwest from Water Rock golf course through the perimeter fence. The entry point predicated by the model is however is further to the north than where presented in the estimated 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 included in the model.

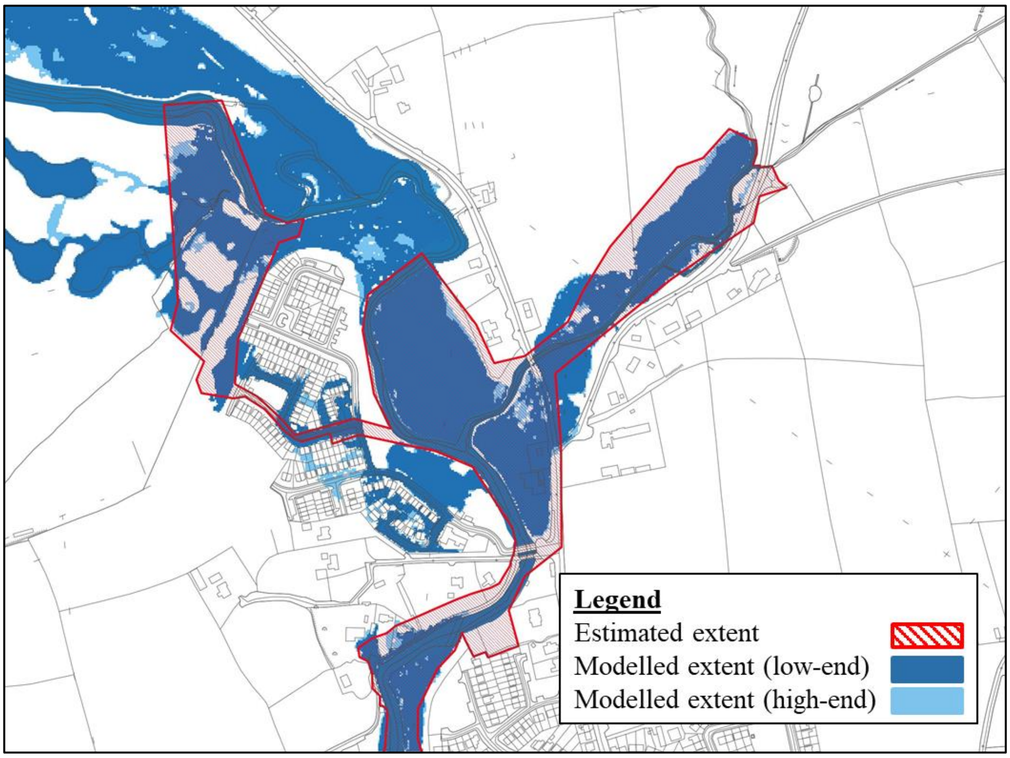

Figure 5.9 December 2015 Maximum flood extent calibration at Tir Cluain/Moore's Bridge

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 20mm higher than the low-end estimate i.e. maximum flood depths are circa 270mm to 380mm.

It can also be seen from Figure 27 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 5.9 also 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 quantified the underprediction by comparing the results against the estimated peak water levels which were derived from a post flood event survey of wrack marks along the reach. Figure 5.10 presents the comparison. The chainage on the graph covers 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. The Energy grade line is not presented on the plot.[7]

It can be seen from the plot 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.

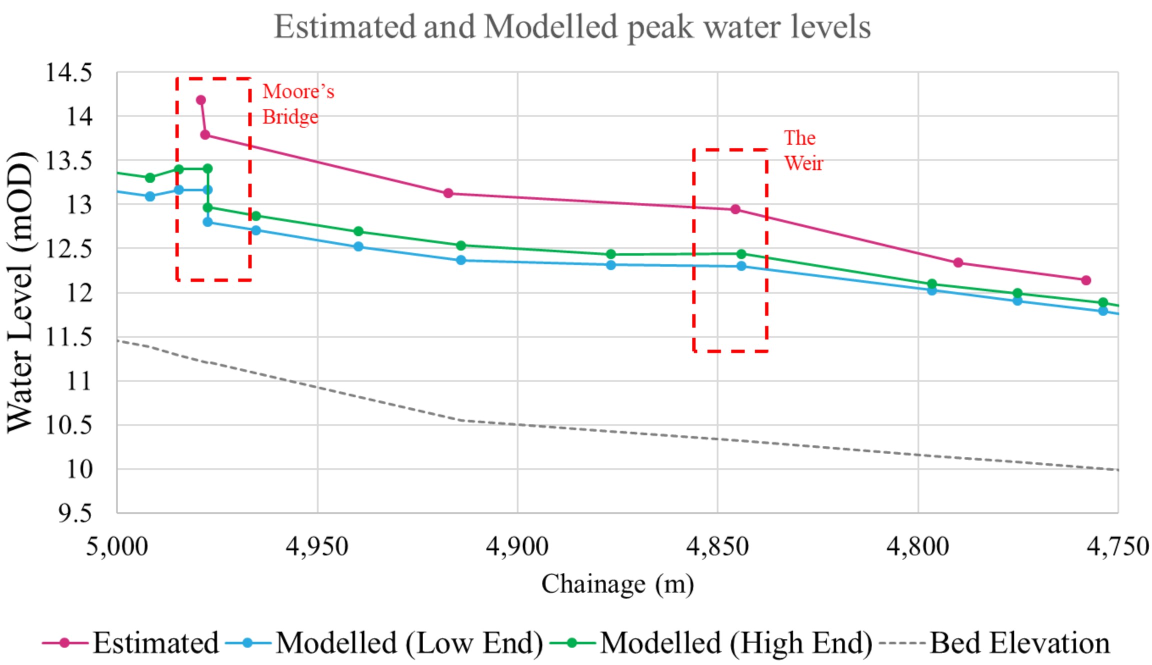

Figure 5.10 Peak water levels downstream of Moore's Bridge: modelled and recorded

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 hydraulic model. These are listed as:

- Changes in the channel geometry;
- Changes in the vegetation;

[7]The energy grade line through the reach will however be considered as part of the optioneering for the reach

- 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 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 considered 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 5.11 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 5.11 Blockage at Moore's Bridge

Figure 5.12 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 5.12 Tree in the river downstream of Moore's Bridge

Figure 5.13 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 5.13 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 one 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 8. 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 5.14 presents an image of the bend in the river taken on one of our recent site visits. 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 as 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 5.14 Ineffective flow areas downstream of Moore's Bridge

5.5.3 Calibration at Railway crossing/Northern Relief Road

Figure 5.15 presents the modelled and recorded maximum flood extent for the area in the vicinity of the Northern Relief Road. It can be seen from the figure 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 hut 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 hut (8.59mOD) is set 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 very 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.

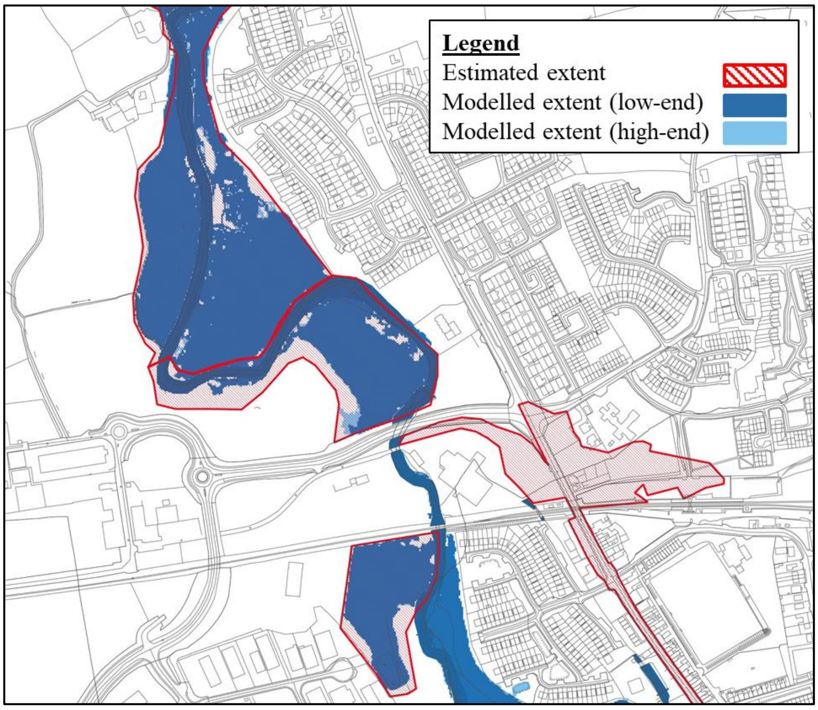

Figure 5.15 December 2015 Maximum flood extent calibration at Railway crossing/Northern Relief Road

From Figure 5.15 it can be seen that 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 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 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 5.16. 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 5.16 Mechanisms of flooding North of the Railway Line

Through inspecting photographs of both the old access road bridge and the railway culvert (Figure 5.16) 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 5.17 presents a photograph at the location marked as point A in Figure 5.16. It can be seen that there is significant vegetation at this location which can block the flow.

Figure 5.17 Downstream face of culvert on Millrace (Refer to Point A on Figure 5.16)

- 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 5.18) in order to illustrate the extent of the siltation.

Figure 5.18 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 5.19 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.

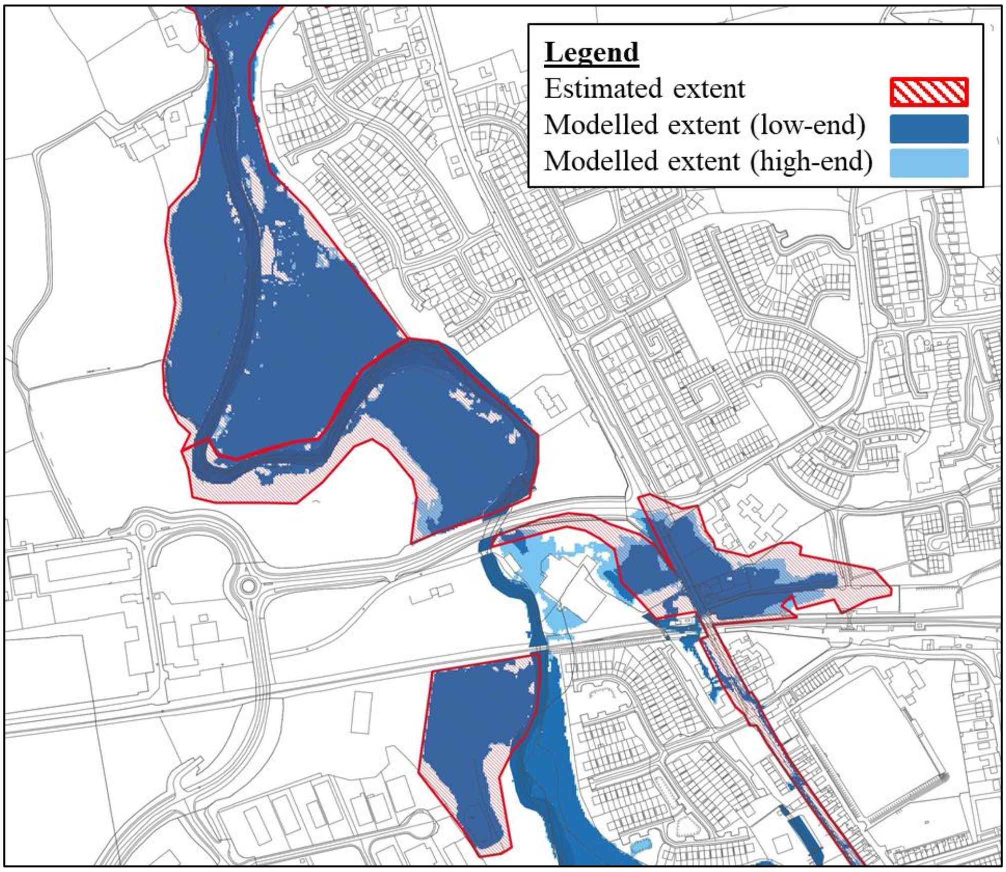

Figure 5.19 40% blockage at the old access road bridge and 95% blockage at the millrace culvert

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.

5.5.4 Calibration at downstream end of the culvert millrace

Figure 5.20 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 5.21. 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 and neither is there evidence to suggest that the area of the town inundated from overtopping at this location was flooded during the event. It is therefore likely that the model overestimates water levels at the downstream end of the millrace and peak water levels remain in bank.

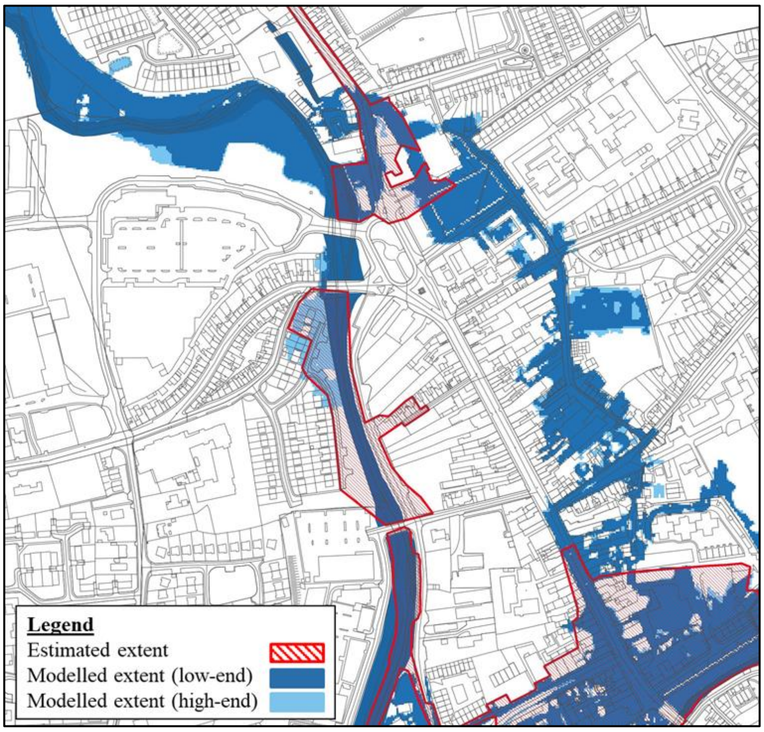

Figure 5.20 Water escaping the downstream end of the millrace

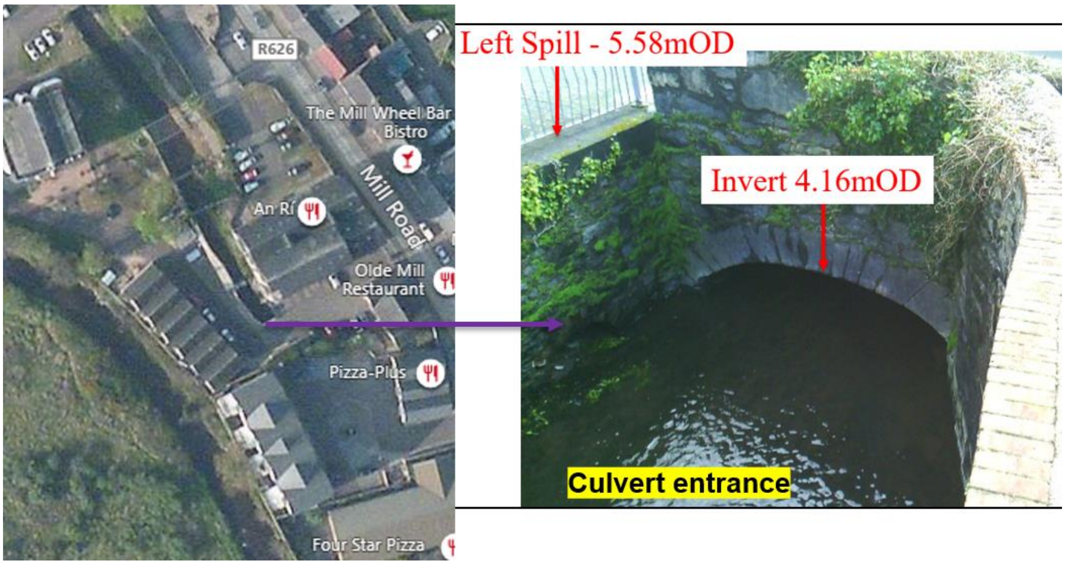

Figure 5.21 Downstream end of the millrace

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 may have 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 discussed sluice gate at the entrance to the millrace. It is our understanding that this structure is no longer in use. However, it may have been 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 all along the mill race culvert, we have utilised the 95% blockage of the railway culvert upstream scenario (which was 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 case, the flow rate through the downstream end of the culvert is reduced and hence peak water levels are also reduced. Table 5-5 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 5-5 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

5.5.5 Calibration in the town centre (Owenacurra dominated)

Figure 5.22 presents the modelled and estimated maximum flood extent for the main area of the town impacted by the Owenacurra for the millrace blocked scenario. It can be seen from the figure that the flood extent along the main channel of the Owenacurra is well matched to the recorded extent for the high-end flow scenario.

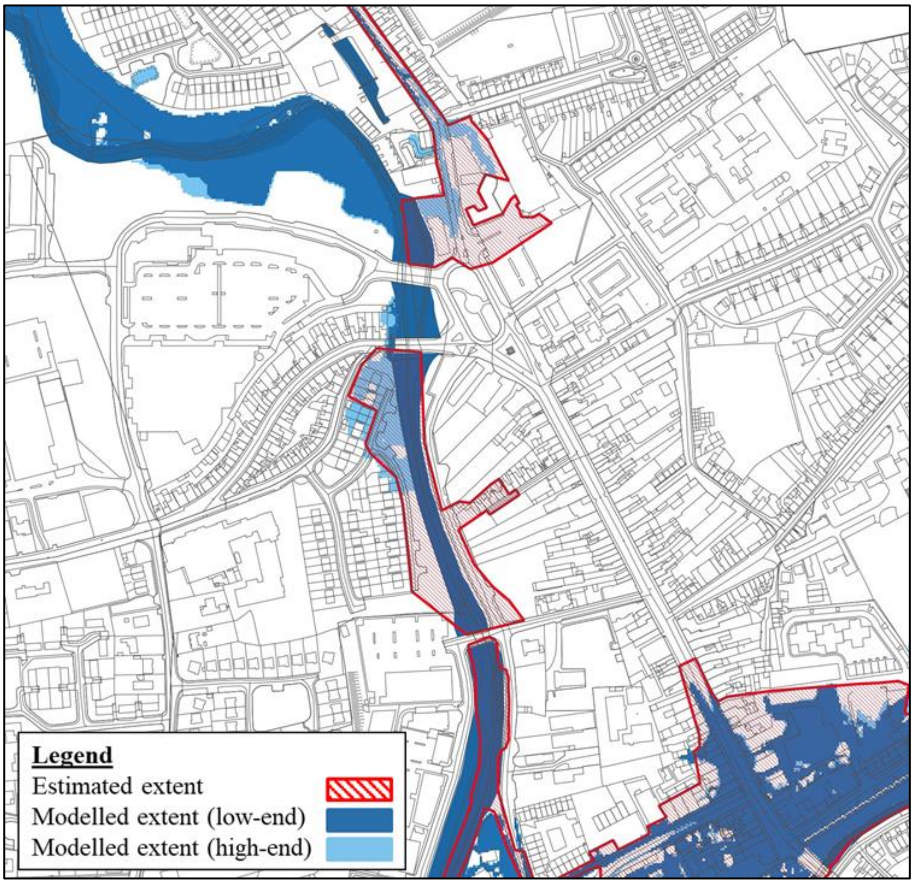

Figure 5.22 Flood extent calibration in the town (from Owenacurra)

From Figure 5.22 it can be seen that Thomas Street was flooded during the December 2015 flood event by overland flow from the left bank of the Owenacurra. This mechanism of flooding may not however have occurred for a number of reasons:

- 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 5.23) 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).
- The threshold of flooding of the Owenacurra in the immediate vicinity of Thomas Street is circa $73m^3/s$ which is greater than the 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 would have lead 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.

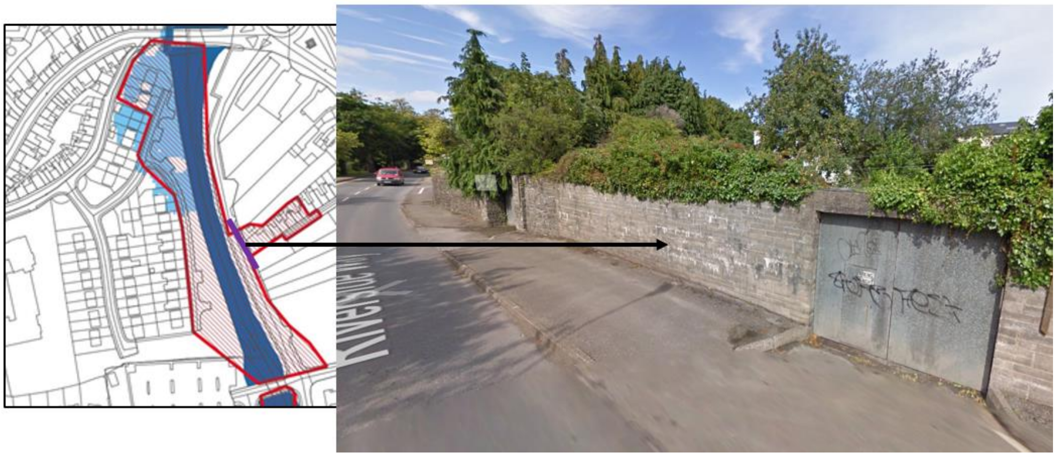

Figure 5.23 Boundary wall between Thomas Street and the Owenacurra

5.5.6 Calibration at the upstream end of the Dungourney River

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 5.24). 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 for the peak of the event.

Figure 5.24 Photo of trench box installed by CCC to limit the inflow to the millrace

Figure 5.25 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.

It can be seen from the figure 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 underestimated in flood extent map.

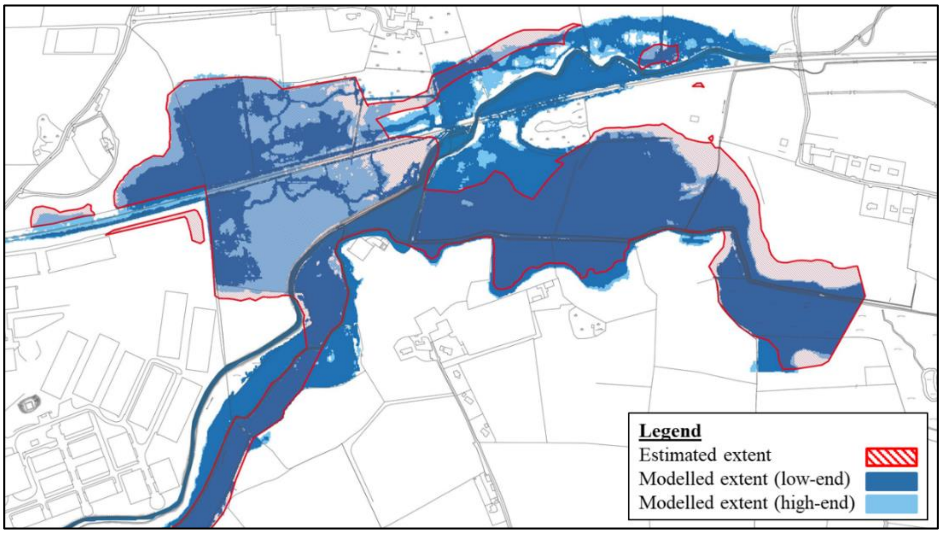

Figure 5.25 Calibration extent – upper Dungourney (with trench box)

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

Figure 5.26 presents the modelled and estimated maximum flood extent for the lower of the Dungourney river. It can be seen from the figure that the modelled extent is well matched to the estimated extent in the lower reach.

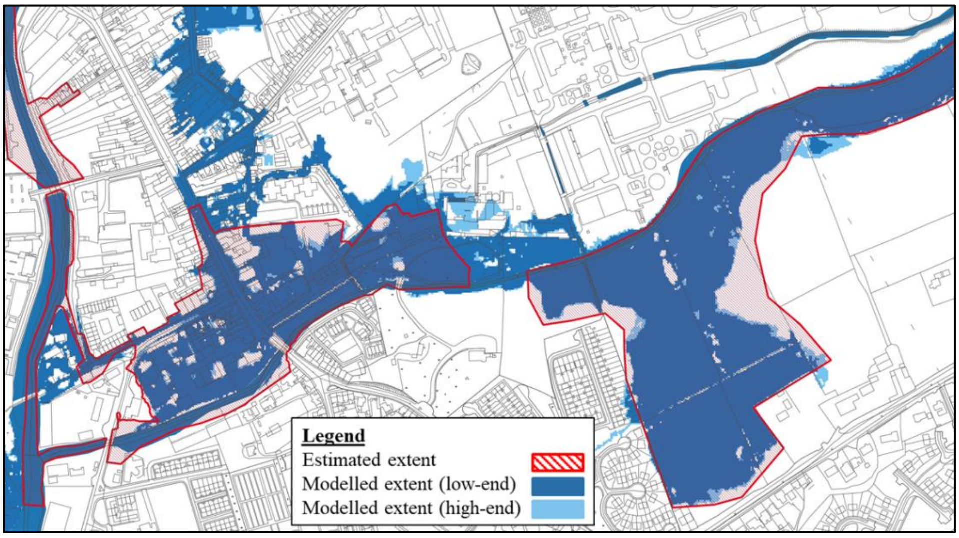

Figure 5.26 Calibration extent – Lower Dungourney (with trench box)

5.5.8 Assessment of the impact of the Trench box

In order to assess the impact of deploying the trench box in Midleton 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 5.27 presents the findings of the analysis. It can be seen that with the trench box removed a greater volume of water enters the IDL Millrace at the upstream end and flows downstream. Parts of the IDL site are as a consequence 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 flows south towards the Dungourney as existing ground levels fall in that general direction. Flood depths across the IDL site however are low.

With the trench box in place a greater volume of water enters the Dungourney River. It can also be seen from Table 5-6 and Table 5-7 that the increase in peak water level associated with having the trench box in place is very minor (circa 0.02m) and does not result in a greater flood extent. 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.

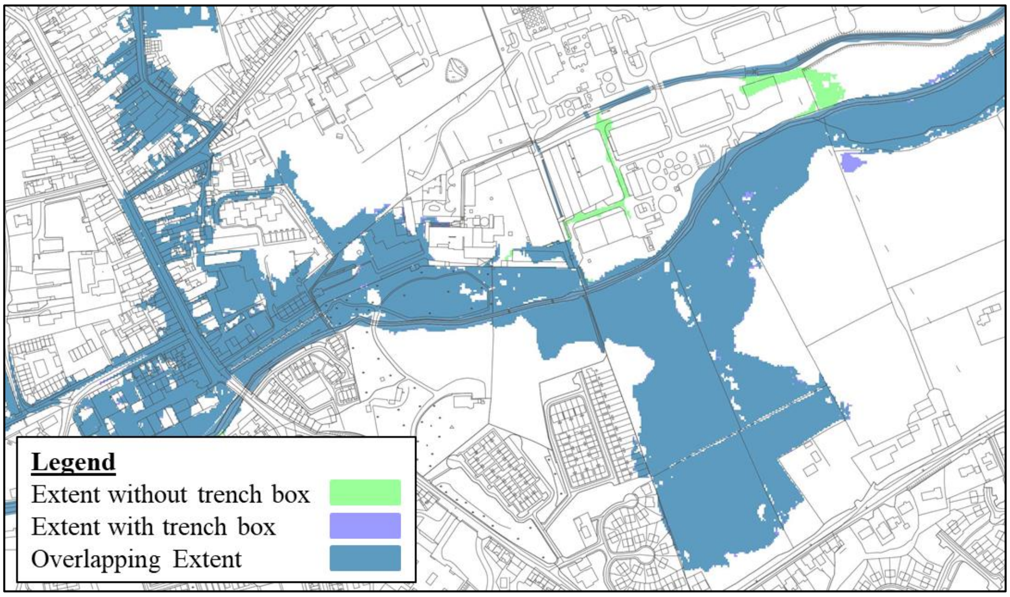

Figure 5.27 Calibration extent for low flow estimate – with and without trench box

Table 5-6 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

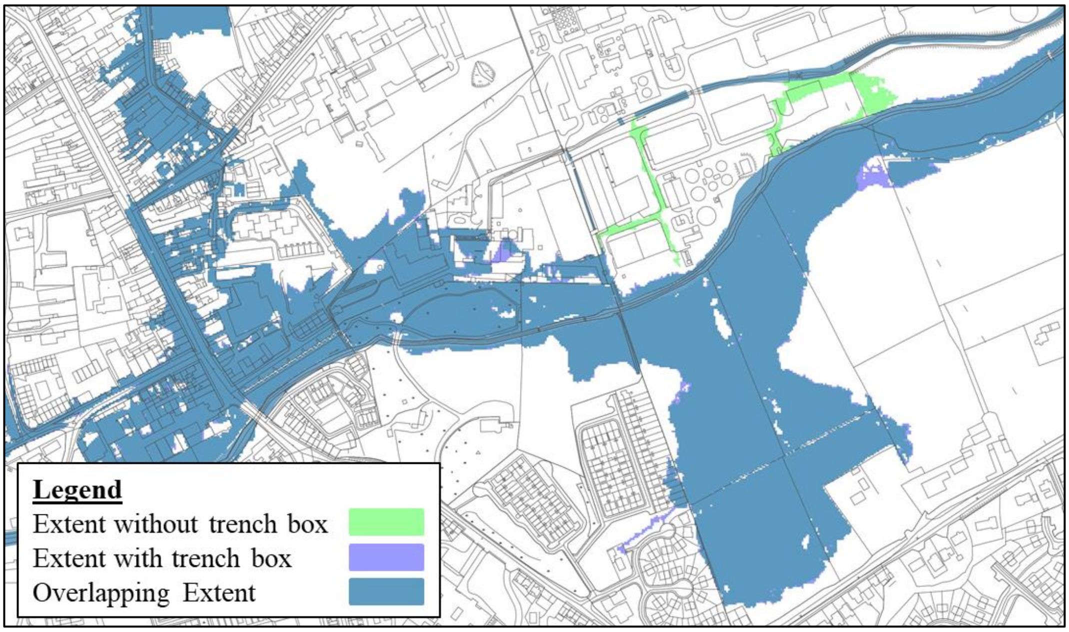

Figure 5.28 Calibration extent for high flow estimate – with and without trench box

Table 5-7 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

5.5.9 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.

5.6 Ballinacurra model Calibration

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 of model parameters as detailed in the literature. A sensitivity on the key model parameters of the model will be undertaken as part of the optioneering.

5.7 Water Rock model Calibration

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. The estimated peak water level at Water Rock House during the event was 11.0mOD which was estimated based on observed levels during the event (Figure 5.29). 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 5.29 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 and 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] It was suggested to Arup in the course of this study 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 during the has been collected by Arup.

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. The inflows to the downstream section cannot therefore be determined with any degree of confidence and any attempt to do so would very likely lead to incorrect conclusions been drawn. The accuracy of this section of the model was therefore ensured by following best practice in the model build and adopting standard values of model parameters as detailed in the literature.

5.8 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 but is not well calibrated in other areas for the unblocked scenario. There is a strong evidence base to suggest that blockages occurred during the event and when these are considered, the model is able to reproduce the mechanisms of flooding that were observed during the event and reproduce 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.

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

6. Hydraulic Modelling Results for the existing scenario

6.1 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 6-1.

Table 6-1 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

6.2 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 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 the report.

6.3 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 5. A sensitivity on the occurrences of blockages and their impact on flood risk, will be considered at a later stage in the project.

The discussion of the existing scenario in the following sections of the report have been broken down into 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 Stream

6.3.1 Tir Cluain/Moore's Bridge (Owenacurra)

Existing Scenario

The Owenacurra river gets out of bank for the Q5 year event north of Tir Cluain estate and collects to the East of the estate but does not flow into it. For the larger Q10 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 0.05m. 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 also been removed for this event which causes water to enter 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 0.12mOD. More than sixty 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 in the paragraph above. It is also however at risk from the Owenacurra overtopping its banks upstream of the access bridge into the estate.

Moore's Bridge is overtopped for the Q100 existing scenario design event. Water gets out of bank downstream of the bridge on both sides and inundates three properties in this event. These properties are also at risk of overland flow coming from Tir Cluain for both the Q100 and Q1000 events.

The primary mechanisms of flooding on the Elfordstown stream is overtopping of the left bank of the stream immediately upstream of the stone arch bridge (Figure 6.1) 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 6.1 Bridge opening on the Elfordstown stream

6.3.2 Northern Relief Road/Railway cottages (Owenacurra)

Existing Scenario

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 set at 8.43mOD and the maximum Q100 water level upstream of the bridge is 8.57mOD, a difference of 0.14m. The bridge opening is therefore fully surcharged in the design event.

Our design runs have assumed that the brick wall and brick hut on the left bank of the Owenacurra upstream of the NRR keeps water in-bank up to the Q50 event. There is therefore no overtopping of the left bank at this location up to this event in the model.

The wall and hut however has been removed from the model for the Q100 event and water therefore overtops the left bank of the Owenacurra at this location for this event. 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 1.8m.

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

- Flood water escapes the Owenacurra River immediately downstream of the NRR and floods the site adjacent to the river for 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.
- Flood water escapes the millrace immediately upstream of the railway culvert for the Q100 event 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.

The railway crossing that intersects with the R626 road in the vicinity of the Railway cottages is elevated above existing ground levels and therefore acts as a control on flood levels. For 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 on 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.

6.3.3 Town Centre (Flood risk from Owenacurra)

Existing Scenario

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

- As discussed in the previous section, 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 New Cork Road Bridge in the Q100 event. The volume of water escaping the channel at this location however 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 Q10 event but does not inundate any of the properties. For the Q100 event however circa fifteen properties are inundated to a depth of circa 0.35m.

6.3.4 Upstream of the IDL site (Dungourney flood risk)

Existing Scenario

Water escapes the Dungourney stream at a number of locations upstream of the IDL site for the Q10 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.[9] Water does not therefore inundate the IDL site but instead collects upstream of it.

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

For the Q100 event, water levels upstream of the site are sufficient to overtop the old railway line and flow in a Westerly direction. There is a sufficient volume of water in this event to cause circa 20 residential properties in the Lauriston estate to be inundated. Three commercial properties in the adjacent industrial estate are also flooded.

The area of Lauriston Estate is also 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.

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

Existing Scenario

The main area of the town in the vicinity of The Baby's Walk and Main Street is at risk from the Dungourney River for both fluvial and tidal flooding. The threshold of flooding for fluvial flooding is the Q5 event which inundates the area of The Baby's Walk but does not flood any properties. The Q10 event leads to more significant flooding and inundates circa 25 properties at the southern section of Main Street. Extensive areas of the main town centre are flooded for the Q100 event and over 200 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 and also from overland flow coming from the Owenacurra River as detailed in 6.3.3.

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[9] 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.

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 the report.

6.3.6 Downstream of the N25 road

Flood risk downstream of the N25 is predominately tidal and is addressed later in the chapter in Section 6.4.

6.3.7 Water Rock Stream

Existing Scenario

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 inundated as a consequence. 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 fall 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 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 Cork Road.

A number of properties downstream of the Cave system are also at risk from fluvial flooding. The WWTP is also at risk. The fluvial flood extents presented on the maps downstream of the Cave system are based on model results which account for the full hydrologically estimated flow from the HEP 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.

6.3.8 Ballinacurra Stream

Existing Scenario

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 over 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 Q50 event where more than 25 properties are at risk. For the Q100 event more than 35 properties are at risk.

6.4 Discussion of the Tidal Flood Risk for the Current Scenario

Existing 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 T25 design event.

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 however greater due to higher floor levels. For the five low lying properties it is equivalent to the T25 event. The majority of properties however in this area are first inundated by the T50 event.

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. The tidal flood maps therefore do not indicate any tidal flood risk in the Ballinacurra area.

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

7. Climate Change Scenario Model Runs

7.1 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 7-1.

Table 7-1 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

The hydraulic model developed to simulate the existing scenario was modified in order to model the climate change runs. The key changes made to the model are listed as:

- The grid spacing of the 2D component of the model was increased from 2m to 4m in order to significantly reduce the computational run time of the model;
- The 1D/2D boundary of the model was modified in order to accommodate the lower resolution grid;
- The Ballinacurra model was converted from a 1D only model to a 1D/2D model;
- The Water Rock and Ballinacurra models were integrated with the main model of the Dungourney and Owenacurra such that a single composite model of the entire scheme area was developed.

The reader is referred to the Scheme Climate Change Adaptation report for further information on the set up of the climate change models.

7.2 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 the Stage 1 of the project .

7.2.1 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 30 residential properties are inundated by the 1% Fluvial AEP undefended MRFS event when compared to 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 extents. An additional 9 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 MRFS event in this area ranges from circa 0.02m - 0.07m.

7.2.2 Northern Relief Road/Railway cottages (Owenacurra)

MRFS

The MRFS mechanisms of flooding are the same as for the Current scenario only that 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 however largely the same 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 flood depths across the area.

7.2.3 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) however leads to a much greater flood extent in the tidally dominated reach.

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 introduces a new mechanism of flooding both upstream and downstream of the Lidl Bridge as water gets out of bank on both the left and right bank of the Owenacurra.

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 7 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. 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 of the maximum extent of the HEFS when compared with the MRFS. The flood depths are however circa 0.5m higher in the tidally dominated reach.

7.2.4 Upstream of the IDL site (Dungourney flood risk)

MRFS

The MRFS mechanisms of flooding are the same as for the Current scenario only 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 and the length of the tidally dominated reach is increased.

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

MRFS

Due to the higher downstream boundary associated with the MRFS, the fluvial/tidal dominant transition point is located just upstream of the Dungourney footbridge Bridge 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 of the maximum extent of the HEFS when compared with the MRFS. The flood depths are however circa 0.5m higher in the tidally dominated reach.

7.2.6 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 50 properties are inundated in the MRFS when compared with the current scenario. An extra circa 100 properties are inundated in the HEFS when also compared with the current scenario.

7.2.7 Water Rock Stream

MRFS/HEFS

The MRFS and HEFS extents are similar to the current scenario. The flood depths are however greater due to the greater volumes of water.

7.2.8 Ballinacurra Stream

MRFS/HEFS

The MRFS and HEFS mechanisms of flooding are the same as for the Current scenario only that the greater volume of water associated with the event and the longer time for 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 overtopped by the tide in the HEFS event due to the increased water level associated with this event.

8. Sensitivity Analysis – Current scenario

8.1 List of Sensitivity Runs

A number of sensitivity analysis runs were undertaken in order to assess how the 1% AEP design water levels for the existing scenario may vary under different model assumptions. 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 8-1 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

8.2 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 8.1 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.

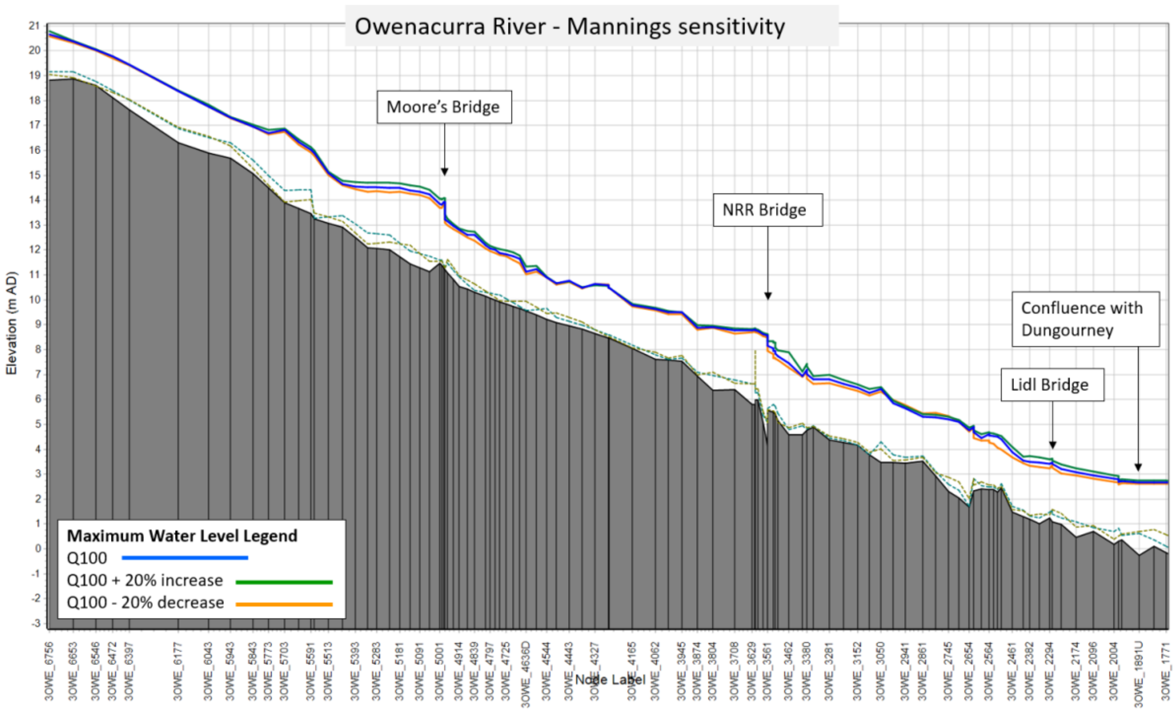

Figure 8.1 20% increase/decrease in the Manning's number –Owenacurra River

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 8.2 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. It can be seen from the plot 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 however 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.

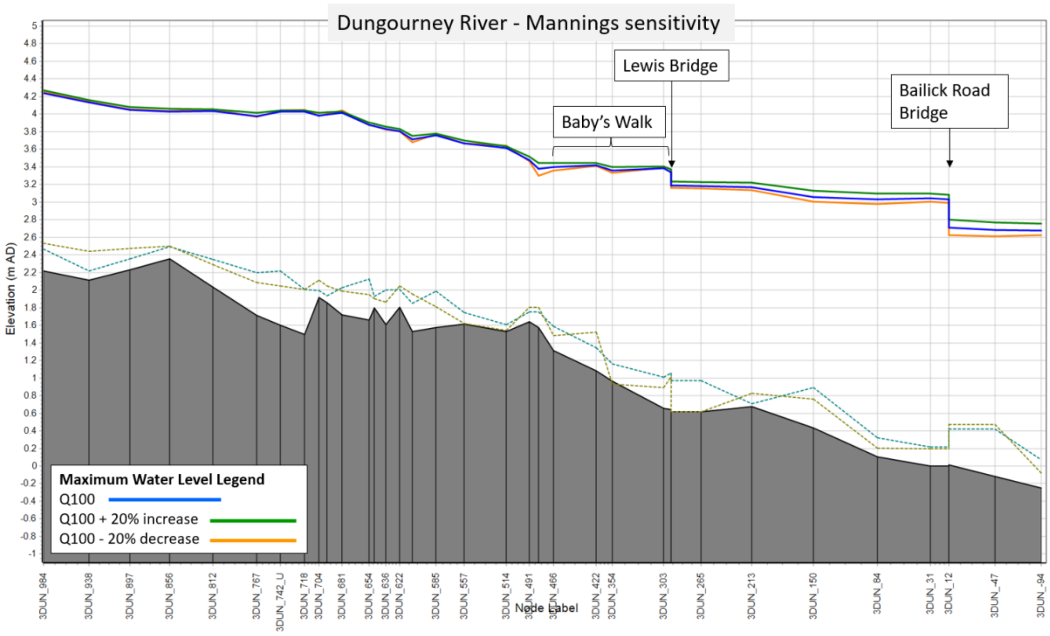

Figure 8.2 20% increase/decrease in the Manning's number – peak Q100 water levels along the Dungourney River

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.

8.3 Culvert/Bridge Head Loss Coefficients

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 which was increased to 2 for the sensitivity run.

Figure 8.3 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. 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 increase the volume of water spilling from the Owenacurra River to the floodplain.

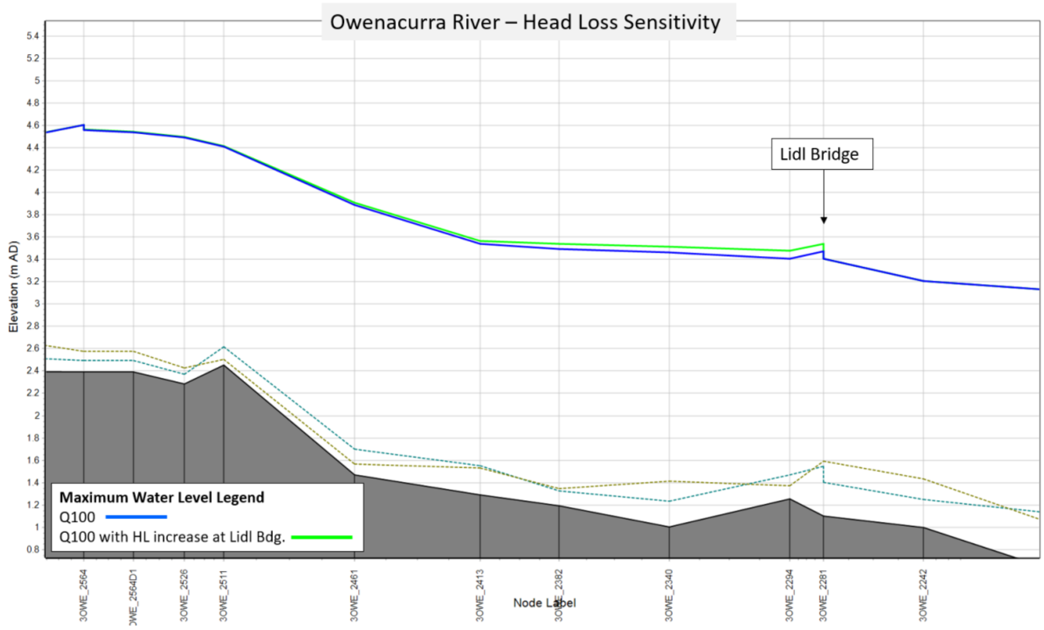

Figure 8.3 Increased head loss coefficient at the Lidl Bridge on the Owenacurra River

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 which was increased to 2 for the sensitivity run.

Figure 8.4 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. 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.

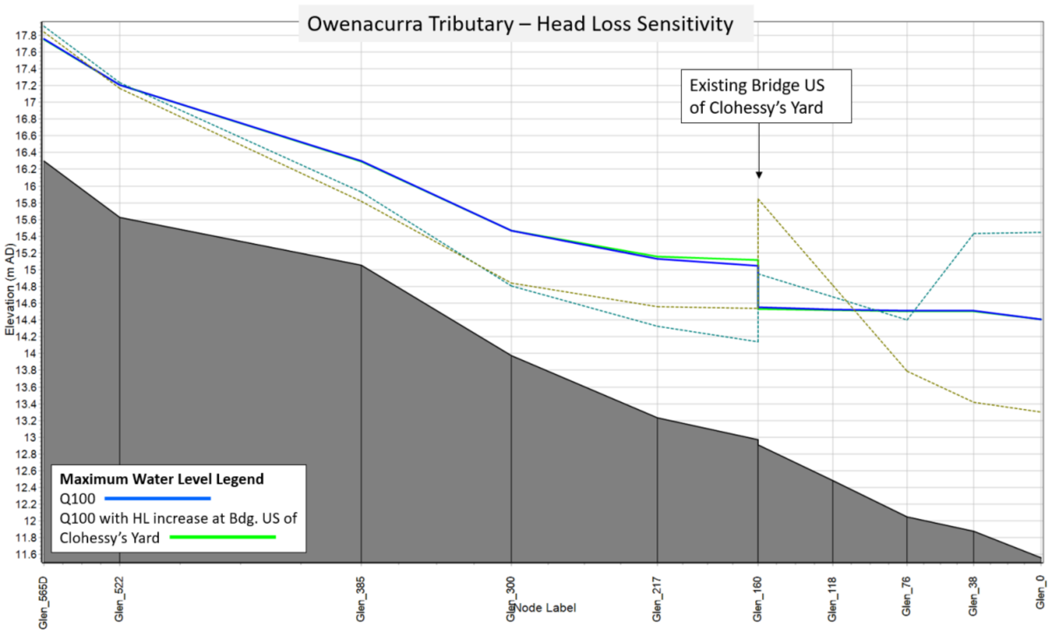

Figure 8.4 Increased head loss coefficient at the Bridge upstream of Clohessy's Yard (Carrigogna Bridge)

Lewis Bridge

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

Figure 8.5 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). 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 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.

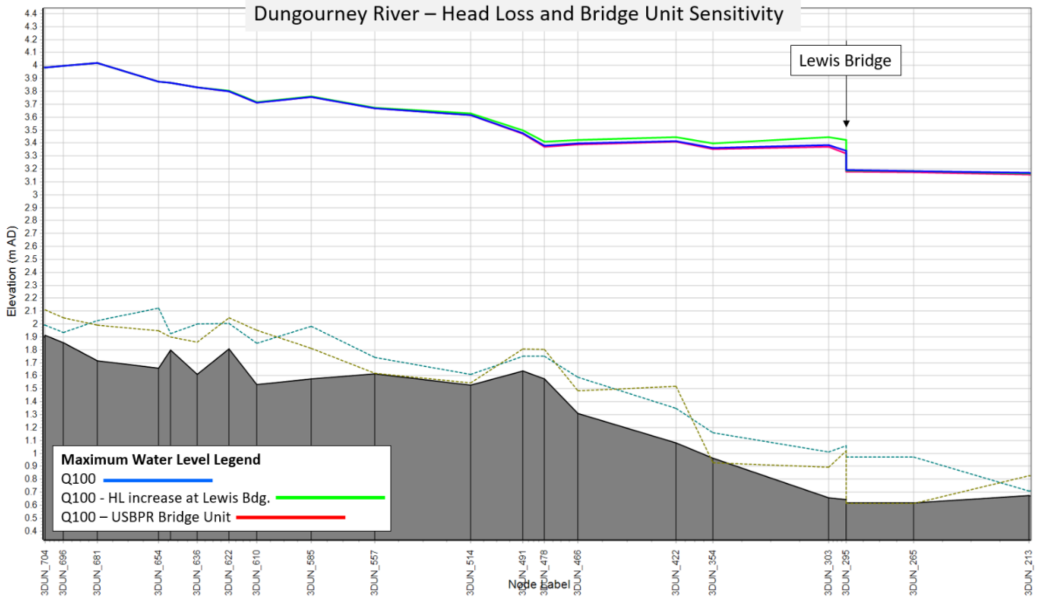

Figure 8.5 Increased head loss coefficient at Lewis Bridge / USBPR Bridge at Lewis

8.4 Bridge unit type

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 8.5 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 change in the flood risk with the alternative bridge unit.

8.5 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 8-2. 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 8-2 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. As each of these small catchments are located upstream of the Moore’s Bridge on the Owenacurra, the critical reach in this sensitivity therefore lies between the confluence of the tributaries with the Owenacurra upstream of Moore’s Bridge to the area in the vicinity of the Northern Relief Road.

Figure 8.6 presents the simulated Q100 flood extent for both the baseline and sensitivity scenario for the critical reach. Figure 8.7 presents the increase in flood depth associated with the sensitivity.

It can be seen from the figure that 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 8-2) which will increase flood risk in the catchment. This will also be assessed as part of the Optioneering.

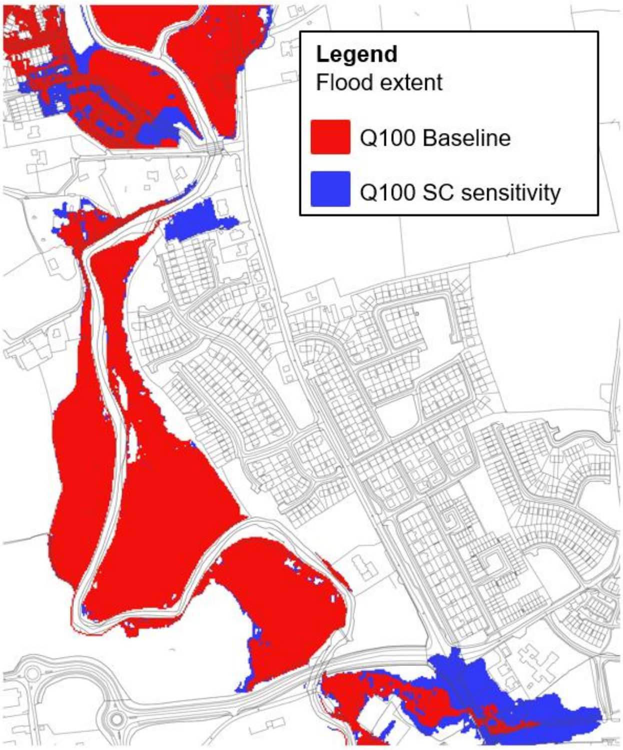

Figure 8.6 Small Catchment Hydrology Sensitivity – flood extent

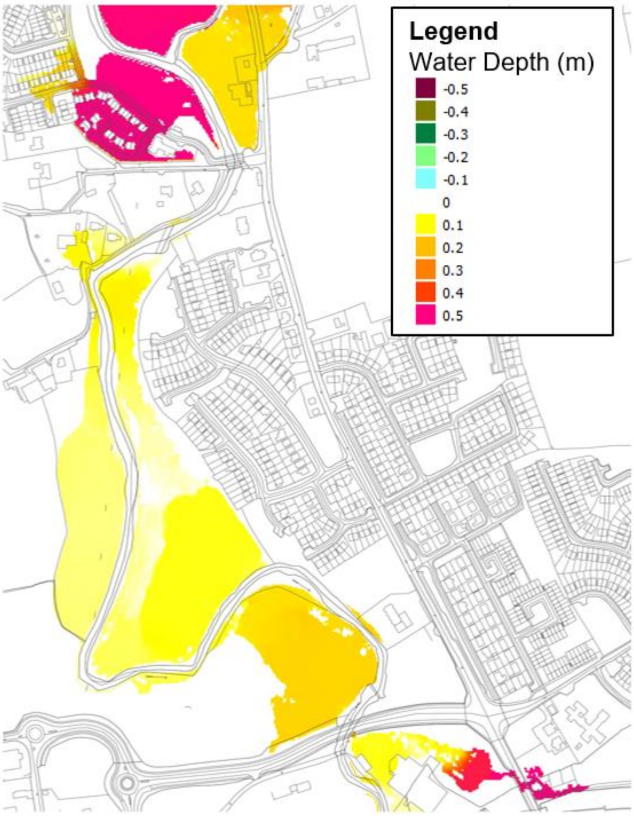

Figure 8.7 Small Catchment Hydrology Sensitivity – increase in flood depth

8.6 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 in Section 7.1.

The results from the 2m grid model were compared with the results of the updated 4m grid model for the 1% AEP event in order to assess the sensitivity of the results. The comparison in the modelled maximum flood extents is presented in Figure 8.8 while the peak water level comparison is presented in Figure 8.7. 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 8.7) 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.

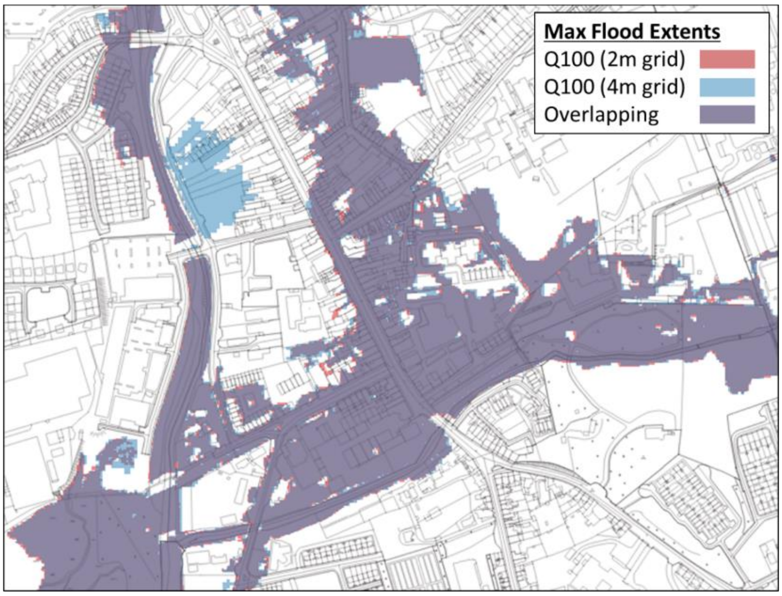

Figure 8.8 Fluvial AEP maximum flood extent – 2m vs 4m grid comparison

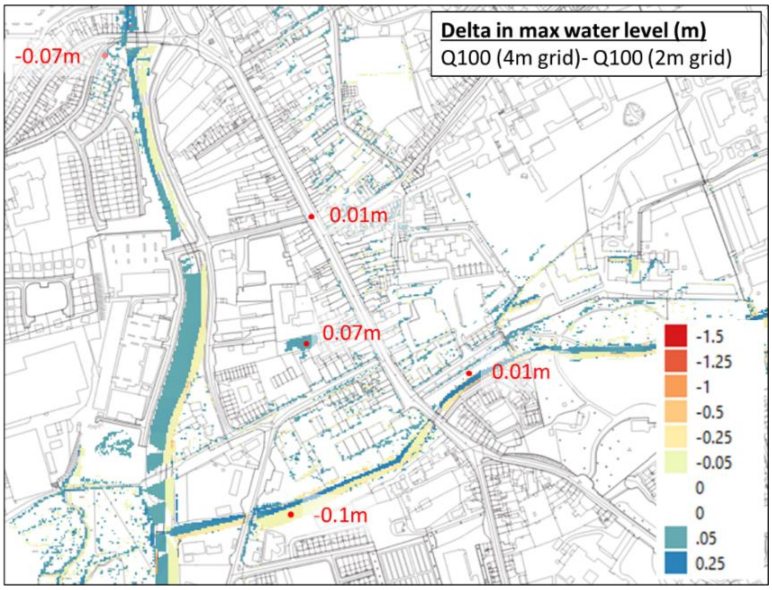

Figure 8.9 1% Fluvial AEP max water level (4m – 2m grid) comparison

9. Pluvial and Groundwater flood risk mapping

9.1 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.

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

As noted in Section 3.3.6.

9.3 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.

9.3.1 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 9.1 presents the finalised 100yr rainfall event hyetograph that was used as the boundary condition to the pluvial model.

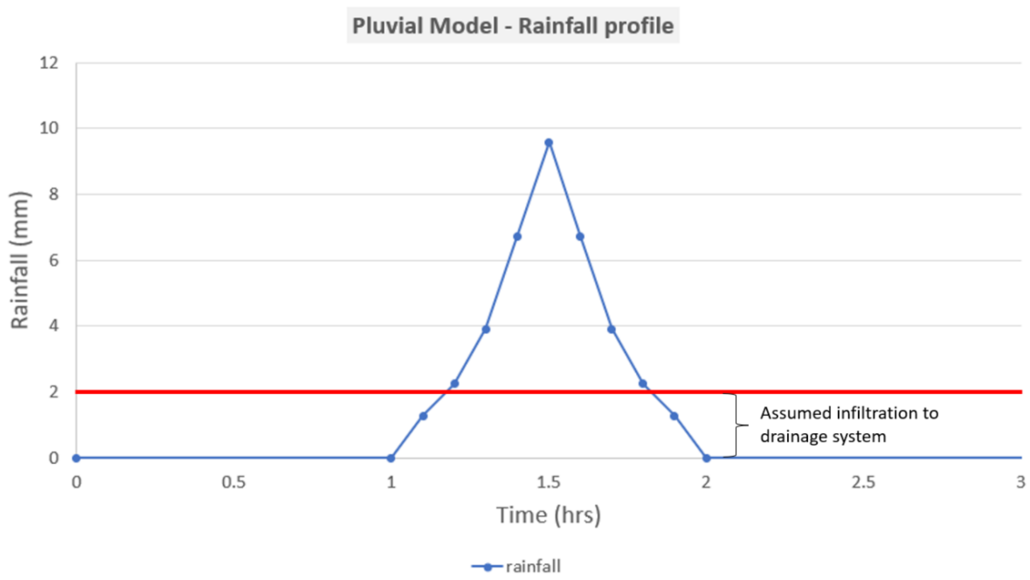

Figure 9.1 Pluvial model hyetograph

9.3.2 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.

9.3.3 Pluvial Model – Results

The pluvial flood risk map is presented in Appendix A. It can be seen from the results that there are a number of areas in Midleton at risk of pluvial flooding. These are listed as:

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

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

9.4 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 therefore:

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

When the area in the vicinity of Water Rock House is inundated during a significant groundwater event, it acts as a blockage on the Water Rock Stream from entering into the Cave System. This 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 is the Water Rock Stream it is classified as fluvial flooding and is presented on the fluvial flood maps. The mechanism which drives the flooding however is a function of a blockage of the Cave system which arises from a ground water flood event. Therefore, while the source of the overland flow route is fluvial, it is driven by the ground water flood risk in the immediate vicinity of the Cave System.

10. Conclusions

A dynamic 1D/2D hydraulic model of all the relevant watercourses in Midleton and associated floodplain areas 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 estimated by the Lee CFRAM study at Roches Point which we propagated into the Owenacurra estuary using a two-dimensional model of the Cork Harbour.

The 1D/2D hydraulic model was calibrated 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 fall 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 but which is not represented in the model. When the calibration for the three events as a whole is considered it is evident that the model is suitable for use in order to simulate design flood events across the scheme area for the study.

Fluvial and tidal flood maps were produced from the result files of the model and highlight all the flood risk areas in Midleton/ Ballinacurra and Water Rock. It was seen from the results that large areas of Midleton are at risk from both fluvial and tidal flooding. Approximately 460 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 10-1 No of properties inundated across the Epochs for the entire Scheme area

Epoch	AEP	Residential	Commercial	Total
Current	10% (F & T)	62	32	94
Current	1%F & 0.5%T	463	188	651
Current	0.1% (F & T)	819	241	1060
MRFS	10% (F & T)	160	75	235
MRFS	1%F & 0.5%T	716	265	981
MRFS	0.1% (F & T)	985	315	1300
HEFS	10% (F & T)	275	118	393
HEFS	1%F & 0.5%T	855	289	1144
HEFS	0.1% (F & T)	1074	320	1394

A number of sensitivity analysis simulations were also undertaken as part of the study. Generally it was found that the model is not sensitive. 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.

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.

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