Checking and Approval

Prepared by:		Marcelo Esper
		Loic Laureau
		Patricia Jaime *Hydraulic Modellers*
		Paul Dunne BE MSc MIEI *Engineer*
		Sergio Herbon, Civil engineer, specialising in hydraulics *Senior Hydraulic Modeller*
Checked by:		Clare Dewar BE MSc CEng MCIWEM MIEI *Principal Engineer*
		Anne Marie Conibear BE CEng MICE, FIEI *Project Manager*
Approved by:		Richard Crowder BSc(Eng), PhD MICE, CIWEM, CEng *Project Director*

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Halcrow Barry has prepared this report in accordance with the instructions of Fingal County Council, Meath County and the OPW for their sole and specific use. Any other persons who use any information contained herein do so at their own risk.

Halcrow Barry
Tramway House, 32 Dartry Road, Dublin 6, Ireland
Tel +353 1 4975716, Fax +353 1 4975716
www.halcrow.com
www.jbbarry.ie

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BRADY SHIPMAN MARTIN

MarCon Computations International Ltd

Executive Summary

Fingal County Council, Meath County Council and the Office of Public Works are undertaking a catchment-based flood risk assessment and management study of the Fingal and East Meath area called the Fingal East Meath Flood Risk Assessment and Management Study (FEM FRAMS). The main output from this study will be a suite of flood hazard and risk maps and a Catchment Flood Risk Management Plan (FRMP), which will identify a programme of prioritised studies, actions and works to manage the flood risk in the Fingal East Meath study area in the long-term. The plan will also make recommendations in relation to appropriate development planning. FEM FRAMS is one of the pilot projects for a new national approach to catchment flood risk management.

This report details the hydraulic assessment that has been undertaken for this study with the objective of determining the flood risk for 23 watercourses in the Fingal and East Meath area, the three estuaries and the Fingal and East Meath coastal area for specific design events and future scenarios. For this, the study has developed hydraulic models for all 23 watercourses and their estuaries, of which three are one dimensional (1D) models and the remaining twenty are 1D-2D linked models. In addition, a 2D coastal model and a pluvial model were developed for the coastal and pluvial analysis of the study area.

Rivers, streams and estuaries included in the FEM FRAM study

River name (abbreviation)		
Mayne River (MAY)	Baleally Stream (BAY)	Balbriggan North Stream (BNS)
Sluice River (SLU)	Bride's Stream (BRI)	Delvin River (DEL)
Gaybrook Stream (GAY)	Jone's Stream (JON)	Mosney Stream* (MOS)
Ward River (WAR)	Rush West Stream (RWS)	Nanny River (NAN)
Broadmeadow River (BRO)	Rush Town Stream (RUT)	Brookside's Stream (BSS)
Lissenhall Stream (LIS)	St Catherine's Stream (CAT)	
Turvey River (TUR)	Rush Road Stream (RUR)	Baldoyle Estuary
Ballyboghil River (BAL)	Mill Stream (MIL)	Broadmeadow Estuary
Corduff River (COR)	Bracken River (BRA)	Rogerstown Estuary

** The Mosney Stream is also known as the Bradden Stream*

In order to prepare river and estuary hydraulic models to undertake the hydraulic assessment a comprehensive data collection phase has been completed. This included the surveying of river and estuary cross-sections and structures, capturing of the ground information in a digital terrain model (DTM), surveying of defence assets and collation of anecdotal flooding information. Approximately 305km of river channel were surveyed along the 23 watercourses, of which approximately 165km consisted of high priority watercourses and 140km were medium priority watercourses. The coastal and pluvial models were developed using the DTM of the study area.

Design events have been run for eight annual exceedence probabilities (AEP) (i.e. 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1% AEP), fluvial and tidal combined, with and without defences and for the current and future flood risk scenarios. Structure blockage and defence failure scenarios have also been carried out for 10%, 1% and 0.1% AEP events.

One of the main outputs of the study is a suite of flood maps, providing a visual interpretation of the results of the hydrological and hydraulic assessments. The flood mapping formats developed map flood information in a number of formats, including flood extent, zone, depth, velocity and hazard. Uncertainty associated with the hydrological and hydraulic assessment has been estimated and the level of confidence associated with the flood outlines communicated to the user on the flood maps.

The outputs from this hydraulic assessment will inform the subsequent stages of this study, in particular the benefit cost analysis and options assessment, which will use the modelling results and flood maps to identify properties (residential and non-residential), assets (utility, transport, social, environmental and cultural) at risk of flooding, the potential economic damage of that flooding and the preferred options for managing the flood risk.

Volume 1 Hydraulics Report details the hydraulic assessment that has been undertaken for the FEM FRAM Study.

Volume 2 Flood Maps contains predictive flood maps (extent, depth, velocity and hazard) and flood zone maps for each of the modelled watercourses in the study area.

Volume 3 Digital Data contains the digital data associated with the hydraulic models and flood maps.

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1. Introduction

1.1. Background and scope of report

Fingal County Council (FCC) commissioned Halcrow Barry to undertake the Fingal-East Meath Flood Risk Assessment and Management Study (FEM FRAMS) in May 2008. The study is being carried out in conjunction with the Office of Public Works (OPW) and Meath County Council (MCC).

FCC, OPW and MCC have recognised the existing flood risk in the Fingal and East Meath area. There is also potential for significant increases in this risk due to climate change, ongoing development and other pressures that may arise in the future. FCC, OPW and MCC are therefore looking to undertake a catchment-based flood risk assessment and management study as a means of addressing this problem. This approach is also in compliance with the EU Floods Directive which requires flood maps by the end of 2013 and a Flood Risk Management Plan by the end of 2015.

The Fingal East Meath study area comprises a group of 23 rivers and streams, three estuaries and the Fingal and Meath coastline. The catchment is approximately 772km^2 in plan area and lies within the Irish Hydrometric Area 08 and some of Hydrometric Area 09 (Figure 1-1). A more detailed figure of the study area with HPW and MPW watercourses, 2D domains, defences and APSRs is included at the back of the report as Figure 1. The study area is bounded by the River Boyne & Mornington River catchment areas to the north and west, the Tolka and Santry river catchments to the south, and by the Irish Sea to the east. All watercourses in the study area flow to the Irish Sea either directly or via the three estuaries (Baldoyle, Broadmeadow and Rogerstown).

The study involves modelling 23 rivers and streams in the study area and three estuaries as detailed in (Table 1-1) below.

Table 1-1 Rivers, streams and estuaries included in the FEM FRAMS

River name (abbreviation)		
Mayne River (MAY)	Baleally Stream (BAY)	Balbriggan North Stream (BNS)
Sluice River (SLU)	Bride's Stream (BRI)	Delvin River (DEL)
Gaybrook Stream (GAY)	Jone's Stream (JON)	Mosney Stream* (MOS)
Ward River (WAR)	Rush West Stream (RWS)	River Nanny (NAN)
Broadmeadow River (BRO)	Rush Town Stream (RUT)	Brookside's Stream (BSS)
Lissenhall Stream (LIS)	St Catherine's Stream (CAT)	
Turvey River (TUR)	Rush Road Stream (RUR)	Baldoyle Estuary
Ballyboghil River (BAL)	Mill Stream (MIL)	Broadmeadow Estuary
Corduff River (COR)	Bracken River (BRA)	Rogerstown Estuary

* *The Mosney Stream is also known as the Bradden Stream*

Figure 1-1 Fingal-East Meath study area (refer to Figure 1 at the back of the report for more detail)

1.2. Objectives

The objectives of this project are to:

- Identify and map the existing and potential future flood hazard and risk areas within the study area;
- Build the strategic information base necessary for making informed decisions in relation to managing flood risk;
- Identify viable structural and non-structural measures and options for managing the flood risks for localised high-risk areas and within the catchment as a whole; and
- Prepare a Flood Risk Management Plan for the study area, and associated Strategic Environmental Assessment, that sets out the measures and policies, including guidance on appropriate future development, that should be pursued by the Local

Authorities, the OPW and other Stakeholders to achieve the most cost-effective and sustainable management of flood risk within the study area taking account of the effects of climate change and complying with the requirements of the Water Framework Directive.

The flood hazards and risks to be addressed include both those that currently exist and those that might potentially (foreseeably) arise in the future. The flood risk management measures, options and management plan should equally address both existing and potential future hazards and risks.

The Flood Risk Management Plan will include prioritised studies, actions and works (structural and non-structural), including indicative costs and benefits, to manage the flood risk in the area in the long-term, and make recommendations in relation to appropriate development planning. The Project is intended to develop a *strategic* flood risk management plan, and is *not* intended to develop *detailed* designs for individual flood risk management measures.

1.3. Approach

In order to the meet the objectives set out in Section 1.2, hydraulic modelling of the relevant watercourses and mapping of the flood hazard and potential risk zones is required. The approach adopted for the hydraulic analysis of the FEM study area incorporated:

- Collection and analysis of data relevant to flooding within the study area;
- Identification, and condition and performance assessment, of flood defence assets;
- Analysis of the hydrology of the catchments within the study area;
- Construction of 20 river models representing the 23 rivers (of which 3 are 1D models and 17 are 1D-2D linked models), one coastal model and one pluvial model;
- Running of design hydrology through the models in order to produce annual exceedence probability water levels and other results to be used in the study;
- Determination of flood mapping formats, including the electronic and hard copy data formats and uncertainty analysis; and
- Production of flood extent, depth, velocity, hazard and zone maps (electronic and hard copy) for the current situation and for potential future catchment changes.

In addition to the above and to provide further information on the potential flood hazard in the study area, the following specific high level assessments were undertaken:

- Groundwater flood hazard assessment;
- Pluvial flood hazard assessment; and
- Geomorphological assessment.

1.4. Technical approach overview

This report details the work and analysis undertaken in relation to, and findings and conclusions of, the surveys and the hydraulic analysis (including flood hazard mapping) for the FEM FRAM study. The report should be read in conjunction with Volume 2 (flood maps) to appreciate and understand the outputs from the hydraulic analysis. For technical readers of the report, Volume 3 (digital deliverables) provides the reader with additional technical data.

- **Chapter 2** details the data collection and surveys undertaken for the hydraulic analysis;
- **Chapter 3** gives an overview of the hydrological approach adopted (full details of the hydrological analysis can be found in the Final Hydrology Report (February 2010));
- **Chapter 4** contains the generic concepts and methodologies which apply to all the models;
- **Chapter 5** summarises the development and calibration/verification (where relevant) of each river model, together with the model sensitivity tests and summary of model results;
- **Chapter 6** details the coastal modelling and approach;
- **Chapter 7** details the flood hazard mapping approach;
- **Chapter 8** details the results of the defence failure scenarios on both fluvial and coastal defences;
- **Chapter 9** details the results of the risk of blockage of structures for 30% and 70% blockage;
- **Chapter 10** summarises the groundwater assessment with further detail included in the Technical Note in Appendix D;
- **Chapter 11** summarises the pluvial flood hazard with further detail included in the Technical Note in Appendix E;
- **Chapter 12** summarises the geomorphological assessment with further detail included in the Technical Note in Appendix F; and
- **Chapter 13** provides the conclusions and recommendations to this hydraulics report.

2. Data collection and surveys

2.1. Introduction

A significant amount of data was collected to provide the basis for undertaking the hydraulic modelling. The data collected included reports, photographs of major flood events, anecdotal evidence of historic flood events, mapping data and survey data. A list of data collected is contained in Appendix A1.

This section provides a summary of the data collected for the hydraulic analysis which was received in a number of different formats. The majority of the datasets were used to develop the hydraulic computer models of the rivers, estuary and coast. Geographic Information Systems (GIS) have been used for the spatial representation of a range of datasets, data storage, data analysis, data management, data calculation and graphical display.

A number of organisations and websites have been consulted to obtain the necessary data including FCC, MCC, Dublin City Council (DCC), the Environmental Protection Agency (EPA), the OPW and the Department of Agriculture, Fisheries and Food (DAFF). A summary of the data gathered for the hydraulic analysis is outlined below.

2.2. Map information

Mapping data has been used to inform the survey specification, the development of hydraulic models and the presentation of model outputs through flood mapping. The main mapping datasets used in the hydraulic analysis are presented in Table 2-1 below. Further information on the uses of the mapping datasets is described in the relevant report sections.

Table 2-1 Mapping data used for hydraulic analysis

Mapping dataset	Source	Use
1:1,000 OSi vector maps	FCC and MCC	Hydraulic model build, flood maps, generation of building polygons
1:2,500 raster maps	FCC and MCC	Hydraulic model build, flood maps
1:2,500 vector maps	FCC and MCC	Hydraulic model build, flood maps
1:5,000 vector maps	FCC and MCC	Hydraulic model build, flood maps
1:50,000 Discovery Series raster maps	FCC and MCC	Survey specification, hydraulic model build, flood maps
Aerial photography	FCC and MCC	Hydraulic model build, digitisation of building polygons.
Google map, OSi online map	Online	Hydraulic model build

2.3. Hydrometric data and historic flood data

Hydrometric data for the gauging stations in the study area and for some of the other gauging stations in the neighbouring catchments was provided by the OPW and the EPA. The information on historic floods in the study area was sourced from the OPW, FCC, MCC, DCC, and from various websites, organisations and individuals. Some information on the summer 2008 flood was also collected by the Halcrow Barry field team during the defence asset survey.

As reported in the FEM FRAMS Hydrology Report, there are nine hydrometric gauging stations on six rivers in the study area as detailed in Table 2-2 below. However, seven out of the nine hydrometric gauging stations were closed between 1995 and 2001 (only stations 08011 Nanny and 08008 Broadmeadow are currently operational).

The OPW provided a list of 141 historic flood events in the study area in MapInfo Tables. Further information related to these historic floods, in the form of reports, wrack mark surveys and photographs of flooding, was sourced from the National Flood Hazard Mapping website www.floodmaps.ie and from the local authorities. Chapter 3 of the Final Hydrology Report (Halcrow Barry, 2010) and Chapter 4 of the Preliminary Hydrology Report (Halcrow Barry, 2009) provide details on the review and analysis of historic floods in the Fingal East Meath study area. Of the information available, the more recent events and particularly the significant flood events (e.g. Hurricane Charlie 1986) provide the more extensive data.

For hydraulic modelling calibration, it is necessary to have water level data at the gauging stations and ideally other observed flood level data such as wrack marks, at other locations along the modelled reach. Given the fact that seven of the hydrometric stations in the study area were closed by 2001 and that the historic flood data is better for recent events, it was difficult to identify events that had both observed recorded hydrometric data plus other observed flood level data. Table 2-2 summarises the rivers that had suitable calibration data. Further information on the calibration methodology and use of the hydrometric data can be found in Section 3.3, Section 4.4.3 and in the relevant individual model sections in Chapter 5.

Table 2-2 Gauging Stations within the catchment

Station	River	Hydrometric Data		Suitable for calibration
		Instantaneous	AMS	
08002 Naul	Delvin	1977 – 2001		
08003 Fieldstown	Broadmeadow	1976 – 1998		
08005 Kinsaley Hall	Sluice	1977 – 2001		Yes
08007 Ashbourne	Broadmeadow	1977 – 1997		Yes
08008 Broadmeadow	Broadmeadow	2006 – 2008	1978 – 2006	Yes
08009 Balheary	Ward	1980 – 1996		
08010 Garristown	Garristown (trib. of Delvin River)	1983 – 1997		
08011 Duleek	Nanny	1979 – 2008	1979 – 2008	Yes
08012 Ballyboghil	Ballyboghil	1980 - 1999		Yes

2.4. Channel and structure survey

The channel and structure cross section survey was carried out by DigiTech 3D (D3D) Surveys. The survey commenced in January 2009 and was completed in November 2009. The survey was used to gather details of river and structure profiles, including cross-sections of the river bed, the river banks and any structures in the river channel such as bridges, culverts and weirs. A total of 3,665 channel cross sections, 360 bridge structures, 475 culverts, 117 weirs were surveyed along the 23 watercourses and their tributaries.

An additional survey was undertaken by D3D at Baldoyle and Rogerstown estuaries. Bathymetric and cross section data for the Broadmeadow Estuary was provided by FCC. The scope of survey works at the estuaries included 20 estuary cross sections, 2 bridges and approximately 5km of defence survey.

Survey works were delivered on a river by river basis to allow development of individual hydraulic models. Volume 3 of the report contains the digital survey deliverables including AutoCAD drawings, ISIS text files and photographs.

2.4.1. Coverage and classification of watercourses

The existing towns and villages in the study area which are subject to existing flooding and for which significant development is anticipated are defined as the Areas of Potential Significant Risk (APSRs). The watercourses that give rise to the existing or potential future flood risk within the APSRs are defined as the High Priority Watercourses (HPWs). In addition, the other areas where the flood risk is considered to be moderate are defined as the Areas of Potential Moderate Risk (APMRs). The watercourses that give rise to the existing or potential future flood risk within the APMRs are defined as the Medium Priority Watercourses (MPWs).

In total, approximately 305km of river channel were surveyed along the 23 watercourses and their tributaries in the study area. The scope of channel and structure survey included approximately 165km of HPWs and 140km of MPWs. Figure 1 shows the location of the HPWs and MPWs within the study area.

2.4.2. Specification

A detailed specification for the channel and structure cross section survey were provided in the FEM FRAMS Channel Structure and Geometric Defence Asset Survey Tender Documents (August 2008) prepared by Halcrow Barry. A copy of these documents is provided in Appendix A2.

The number of cross sections surveyed along a river reach depended on the classification of the watercourses (refer to Section 2.4.1). In accordance with the clients' brief, for HPWs, cross sections were generally surveyed at approximately 50 to 100m intervals and for MPWs, cross sections were generally surveyed at 750m intervals. Additional cross sections were surveyed if the channel topography changed significantly, if there was a bend in the river or if there was a significant bed level drop. In addition, all culverts (inlet and/or outlet as appropriate) and structures were to be surveyed. To improve the hydraulic representation of culverts and structures in the models, additional channel cross sections were surveyed both upstream and downstream of culverts and structures.

A detailed channel and structure cross section survey plan was prepared and included in the survey Tender Documents. This plan, based on the 1:50,000 discovery maps, provided geo-referenced point locations for all the required channel and structure cross-sections in the catchment. During the field survey, the survey team made some adjustments on the location of the channel cross sections and chainage of the rivers, so as to represent the actual field condition.

Some minor amendments and additions were made to the specification following the receipt of the first set of survey data. The culverts and structures were surveyed on one face, if the upstream and downstream faces were similar (e.g. for short culverts). Survey/inspection inside the conduit was not part of the survey scope. Therefore, the inlet and/or outlet information (material, shape, size, condition, etc.) was adopted for long culverts. If the inlet and outlet were different a linear interpolation was used in order to characterise the conduit along its length. However, the survey data was supplemented with additional information including drawings and photographs from the client for the following watercourses:

- Balbriggan North Stream;
- Gaybrook Stream;
- Rush West Stream; and
- Mayne River.

Any vertical bed level drop larger than 0.5m was considered as a weir. For symmetrical weirs, a cross section at the crest and the depth of drop was recorded. But for non-symmetrical weirs, a section at the weir crest and another at the downstream of the weir were surveyed.

Cross sections were surveyed looking downstream from left bank to right bank and generally extended for 10m into the left and right bank floodplains to allow for tie-in to the LiDAR Digital Terrain Model (DTM). However, at locations with access problems, extensions of left and/or right banks were limited to a few metres. The surveyors used a range of surveying techniques including GPS and total stations. For the deeper waters in the estuaries, data was captured using a boat and an echo sounder.

2.4.3. Deliverables

The surveyors provided the survey data for the hydraulic model build in a number of formats as detailed below:

- ISIS text file of cross sections (for importing directly to ISIS computer models);
- AutoCAD digital cross-section location key plan;
- AutoCAD digital cross-section and structure data files; and
- Digital photographs of cross-sections and structures (4 photographs per cross section looking upstream, downstream, left bank and right bank).

All of the above are included in the project's digital deliverables available in Volume 3 of the report.

2.4.4. Quality check

Regular meetings were held throughout the duration of the survey to discuss survey progress, health and safety and quality of deliverables.

An initial check and review on the quality of the interim survey data was carried out to check that:

- Specified cross sections had been surveyed;
- Cross section labels (chainage) matched the watercourse length;

- There was no crossing of sections (i.e. in the floodplain at meanders);
- Cross sections extended the specified distance into the floodplain;
- Cross sections were surveyed from left to right bank looking downstream; and
- Survey deliverables matched specification (drawings, photographs, ISIS files).

Based on this review, further instructions were given to the surveyor to improve the quality and format of the data, where this was needed.

Further detailed checks on all the survey data (e.g. ground level consistency against the LiDAR DTM) were carried out during the hydraulic model build. Further discussion on these quality assurance checks can be found in Section 4.4.10.

2.5. Defence asset survey

2.5.1. Introduction

The defence asset survey (DAS) involved gathering information on the type and condition of various defence assets within the HPW and MPW reaches and along the Meath coastline. The Dublin/Fingal coastline defence assets were surveyed as part of the Dublin Coastal Flood Protection Project (DCFPP) and provided by the OPW to Halcrow Barry.

The main defence asset survey was undertaken between the 21st of July and the 9th of September 2008. Further field survey was undertaken between 18th May 2009 and 29th May 2009.

The survey information was entered into a Flood Defence Asset Database (FDAD) on site using the OPW Toughbook survey computer notebooks. The information in the FDAD was used to inform the location of flood defences in the hydraulic computer models (Section 4.4.6) and the defence failure scenario modelling (Chapter 8).

The extent of river defence assets surveyed was approximately 46.85km (left and right river banks counted separately). The survey predominantly covered river defence assets along the watercourses in the following urban areas:

- Swords (Ward and Broadmeadow Rivers);
- Malahide (Gaybrook Stream);
- Ashbourne (Broadmeadow River);
- Ratoath (Broadmeadow River);
- Dunshaughlin (Broadmeadow River);
- Dublin Airport (Cuckoo Stream, a tributary of the Mayne River);
- Lusk (Baleally Stream);
- Rush(Rush Town and Rush West Streams);
- Skerries (Mill Stream);
- Balbriggan (The Bracken River);

- Stamullen (Delvin River); and
- Duleek (Nanny River).

In addition to the above river defence assets, approximately 10.5km of coastal defence assets along the Meath Coastline was also surveyed.

The Dublin/Fingal coastline defence asset data, provided by the OPW to Halcrow Barry, was also entered into the FDAD. Halcrow Barry identified that there were significant quantities of missing data. Additional information, received from Royal Haskoning on the Dublin Coastal data, was entered into the FDAD using OPW in-house resources in early 2010.

During the hydraulic model build, a number of defences were identified which were not surveyed as part of the DAS. Further discussion on these defences is in each of the hydraulic model sections (Chapter 5) and Chapter 6 with recommendations for additional DAS in Chapter 13.

2.5.2. Methodology

Halcrow Barry's two survey teams carried out a visual inspection to determine the condition of the defence assets, with information recorded directly in the FDAD using Toughbooks. The field-entered data was then quality checked in detail in the office. Defence assets surveyed included walls, embankments, flap valves, culverts and bridges. The draft FDAD was submitted to the OPW in December 2008 and the second draft FDAD was submitted in June 2009. The final version of the database was provided to the client in November 2010. A report on the DAD is included in Appendix A3 with further detailed information on the surveyed flood defences available in the FDAD.

2.5.3. Topographical survey of defence assets

The geometry of the defence assets was one of the components of the topographic survey contract described in Section 2.4 above. DigiTech 3D carried out the geometric survey of the defence assets in the study area watercourses and along the Meath coastline.

The topographic survey data of defence assets was reviewed and processed into a format ready for interrogation. Data from the survey was used to attribute asset, structure and/or element information within the FDAD as necessary. It was originally planned that this geometric data be transferred to the FDAD though an automated procedure. However, it was found to be impracticable to do this via automated procedures as originally planned and the work was undertaken manually.

The topographic survey data is available in Volume 3, digital data.

2.6. Floodplain survey

The DTM of the floodplain is a bare earth model of the ground which has all the buildings, structures and vegetation removed. The DTM developed for the study is based on a 2m, 5m and 10m grid cell resolution and was used in the development of the hydraulic models and generation of flood maps.

2.6.1. Sources of data

The LiDAR data of the study area was provided by the OPW in February 2009. The OPW commissioned Terra Imaging Ltd, who used fixed-wing aircraft to capture the LiDAR data. The extent of LiDAR data consists of:

- 2m, 5m and 10m DTM (digital terrain model) covering the HPWs, MPWs, APSRs and APMRs in the study area;
- 2m DEM (digital elevation model) covering the HPWs, MPWs, APSRs and APMRs in the study area; and
- 2m low tide LiDAR DTM along the coastal area and estuaries.

2.6.2. Accuracy of data

According to the OPW, the LiDAR contract specifies a required minimum accuracy of ±0.2m in horizontal and vertical direction (a RMSE of less than 0.2m, with 99% of all points falling within 2RMSE (i.e., two times root mean square error).

The OPW carried out quality checks on the LiDAR data using the active GPS network and using a Trimble R8 GNSS RTK rover unit. Quality checks were undertaken at three different locations within the project area, namely, at Duleek, Dunshaughlin and Swords/Kinsaley. The OPW found that the quality of the LiDAR data satisfied the specifications of the LiDAR contract.

Halcrow Barry carried out further checks on the accuracy of the LiDAR data as part of the hydraulic model build. Surveyed channel cross sections were extended for 10m into the left and right bank floodplains to allow for an overlap with the LiDAR DTM in the floodplain. This allowed for a comparison in levels between the surveyed cross sections and the LiDAR DTM in the floodplains.

3. Hydrological analysis

3.1. Introduction

The Preliminary Hydrology Report, published in February 2009, and the Final Hydrology Report, published in January 2010, details the hydrological assessment that has been undertaken with the objective of determining hydrological inputs for the 23 watercourses in the study area that are to be modelled, for specific design events and for future scenarios. The report also identifies the historical flood data for use in the hydraulic model calibration and validation. The hydrological analysis is based on a review and analysis of historic flood information and use of meteorological and hydrometric records. The Flood Studies Report (FSR), Flood Estimation Handbook (FEH) and the Irish Flood Studies Update (FSU) methodologies have been used to enable the determination of design hydrological inputs, which also consider potential future catchment changes likely to influence flood risk.

The analysis presented in the Final Hydrology Report is concerned with the estimation of extreme flows, which form the basis for providing inflows to the hydraulic models. To distribute the inflows along the river reach, the HPWs and MPWs were further sub-divided into a total of 270 sub-catchments. The catchment characteristics of these sub-catchments have been extracted using GIS automation tools aided by manual checking. Design inflows at these sub-catchments were calculated using the catchment characteristics, FSU-based rainfall inputs and applying the FSSR 16 and IOH Report No. 124 unit hydrograph methods.

3.2. Hydrological linkage

The FSSR 16 and IOH UH method was applied using the ISIS FSSR 16 boundary units, which derive an inflow hydrograph from a catchment or sub-catchment. The ISIS FSSR 16 boundary units were prepared for all 270 sub-catchments and for all the required annual exceedence probability (AEP) events considered. Each ISIS FSSR 16 boundary unit was applied to each sub-catchment node in the hydraulic models as follows:

- For a sub-catchment at the upstream end of the modelled reach, it was applied as a point inflow to the first node at the upstream end of the reach;
- For a sub-catchment across a river reach where there is a non-modelled tributary, it was applied as a point inflow to a single node at the confluence; and
- For a sub-catchment across a river reach where there are not any tributaries, it was applied as a lateral inflow which will act as a distributor to apportion the inflow along the river reach, as opposed to applying a point inflow to a single node.

The following hydrological calibrations were undertaken using the iterative simulations in the hydraulic models:

(i) Optimisation of the critical storm duration

For both gauged and ungauged catchments, the critical (design) storm durations in the FSSR 16 boundary units were optimised using iterative simulation in the hydraulic models (refer to Section 4.4.7). The models were run for different storm durations in the FSSR 16 inflow units for the 1% AEP fluvial event. A comparison was undertaken between the maximum water levels for each storm duration along the HPWs. Therefore, the adopted critical storm durations were the ones which produced the highest water level in the watercourses along their high priority reaches.

(ii) Reconciliation of flows at the hydrometric station

The total routed inflows from all the upstream sub-catchments at the gauging stations were reconciled with the statistical method design floods at the gauging stations for the corresponding annual exceedence probability events; using iterative simulations in the river hydraulic models, to ensure that the modelled flows at the gauges matched the flows estimated using the statistical method. The scaling factors for the reconciliation of flow at the gauged catchments were used to estimate the scaling factors for the ungauged catchments (refer to Section 4.4.7).

3.3. Calibration

Table 3-1~~Table 3-1Table 3-1~~ provides information on the availability of concurrent rainfall and hydrometric data in the study area. Due to the unavailability of suitable rainfall data, it was not possible to use rainfall derived flow hydrographs for the purpose rainfall-driven model calibration for the majority of calibration events. A consistent approach was adopted for the study area which involved using design inflow hydrographs to calibrate the models. Further details on this calibration approach are provided in Section 4.4.3.

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Table 3-1 Availability of concurrent rainfall and hydrometric data in the study area

Stn No.	Location (river and town)	Hydrometric data availability	Calibration / verification events	Rainfall data availability	Comment
08011	River Nanny at Duleek	1980-2006	06/11/2000	Daily rainfall data @ Stn 1032 - Duleek	No rainfall data
			26/08/1986	(1949 - 1991)[1]	No Rainfall data for August 1986
			12/06/1993		No rainfall data
08007	Broadmeadow River at Ashbourne	1977 - 1996	26/08/1986	Daily data @ Stn 2432 - Ratoath (1998 - 2006) [2]	No rainfall data
08008	Broadmeadow River at Broadmeadow	1978 - 2007		Daily data @ Stn 2532 -Dunshaughlin (1998 - 2006)	
08012	Ballyboghil at Ballyboghil	Hydrometric data (1981-1998)	26/08/1986	Nearest rainfall station 2232 @ Garristown (1995 - 2000). Data record too short & not used in rainfall analysis[3].	No rainfall data

Stn No.	Location (river and town)	Hydrometric data availability	Calibration / verification events	Rainfall data availability	Comment
08002	Delvin River at Naul	1977-2001	None	Gauges 2232 Garristown has rainfall data for 1995-2000. The nearest Gauge 1632 has 1975 - 1983 data and Gauge 2332 has 1997-2006 data.	
08005	Sluice River at Kinsaley Hall	1977-2000	26/08/1986	Nearest rain gauge is Dublin Airport where hourly data is available.	

[1]Although the data availability period is 1949 to 1991, the rainfall data for the August 1986 event was missing. [2]Concurrent daily rainfall data for Broadmeadow 1986 event was not available at both Dunshaughlin and Ratoath rain gauges. [3] Concurrent rainfall data for the Ballyboghil calibration event (1986) was not available at Garristown (nearest rainfall gauge) and also not available at further upstream rain gauges (Ratoath and Dunshaughlin)

The table indicates that some of the stations were installed very recently (e.g., Dunshaughlin, Ratoath and Garristown in 1998, 1998, 1995 respectively) and were not used in the rainfall analysis due to the short length of records. The remaining stations were either closed or no rainfall data was available for the selected calibration events (e.g., Duleek was closed in 1991 and it also did not record the rainfall data in August 1986). Only the Dublin Airport rainfall station has hourly rainfall data whereas all other stations have daily rainfall data.

3.4. Design events

The design annual exceedence probability flows at the eight hydrometric stations estimated using statistical method are presented in Table 5-6 of the Final Hydrology Report and are included in Appendix B. These design flood values were not directly used in the models but were distributed across each sub-catchment through ISIS FSSR 16 boundary units (refer to Section 3.2) together with the appropriate scaling factors for both gauged and ungauged catchments (refer to Section 4.4.7).

3.5. Future scenarios

The dominant factors influencing future flood risk in the Fingal and East Meath catchments include changes in climate, land use and urban growth. The effects of these three factors are described in Section 7 of the Final Hydrology Report.

As little afforestation is likely to occur in the FEM FRAM study area, the main factors for future flood risks can be considered as climate change and urbanisation. Table 3-2~~Table 3-2~~Table 3-2 (reproduced from Table 7-6 of the FEM FRAMS Final Hydrology Report) collates both these projections (climate change and urbanisation) for the two future scenarios, namely, the mid range future scenario (MRFS) and the high end future scenario (HEFS).

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Table 3-2 Relevant combinations of drivers to provide boundaries for future flood risk

Driver	Scenario	
	MRFS	HEFS
Climate change - rainfall	+ 20%	+30%
Climate change - net sea level rise	+0.35m	+1.0m
Land use change – urbanisation	100% increase in urban area	400% increase in urban area

To incorporate climate change into the hydraulic model boundary units, the % increase in rainfall was applied to the ISIS FSSR16 boundary units and the change in sea level rise was applied to the tidal boundaries. To incorporate the future changes in urbanisation into the hydraulic model boundary units the changes in the URBAN factor were applied to the relevant sub-catchments in the ISIS FSSR 16 boundary units.

'Urban Fraction' is one of the catchment characteristics of the ISIS FSSR 16 boundary unit. For the current scenario, the 'Urban Fraction' for all sub-catchments was derived directly from the 'Urban polygon' of the study area, using GIS automation. In the MRFS scenario (for which urbanisation will increase by 100%), the current scenario 'Urban Fraction' in the FSSR 16 boundary unit was doubled manually. Similarly, for the HEFS scenario (for which urbanisation will increase by 400%), the current scenario 'Urban Fraction' was increased by four times in the FSSR 16 boundary units. However, for both MRFS and HEFS scenarios, the maximum value of 'Urban Fraction' used in the FSSR 16 boundary units were 1.0, given the fact that the maximum possible extent of urbanisation is 100% of the total sub-catchment area.

3.6. Joint probability analysis

Detailed investigation of the fluvial/tidal joint probability analysis (JPA), based on the approach of the UK Defra/EA (2006) and the Lee CFRAMS, was undertaken during the hydrological analysis and reported on in Chapter 8 of the Hydrology Report. Additional research on this topic and sensitivity analyses involving further simulations of the hydraulic models, was undertaken during the hydraulic analysis. The results of this research and sensitivity analysis are presented in Section 4.4.5 of this report and in the JPA Technical Note included in Appendix C2.

4. River hydraulic modelling overview

4.1. Introduction

Dynamic river hydraulic models have been developed for HPWs and MPWs to estimate design and potential future flood levels, depths, velocities and extents, and to assist in the development and appraisal of potential flood risk management measures and potential strategies. Where possible the models have been calibrated and verified against observed flood events. The models have been run for design flood events with a range of annual exceedence probabilities (AEPs) of 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1% for existing conditions and for the MRFS, and for design flood events for the 10%, 1% and 0.1% AEP events for the HEFS.

This chapter provides an overview of the river hydraulic modelling approach adopted for the FEM FRAM Study, including the generic concepts and methodologies which apply to all the river models. Chapter 5 summarises the development and calibration/verification (where relevant) of each river model, together with the model sensitivity tests and a summary of the model results.

All the model results (calibration, design, 'without defences', MRFS and HEFS, sensitivity test, blockage, defences failure) for all the AEP fluvial and tidal events are available in digital format in Volume 3.

4.2. Software

The ISIS Software Suite was used on the FEM FRAM Study. It has a flexible and comprehensive range of tools for assisting in the design of cost-effective engineering flood defence schemes and developing catchment strategies. The 1D (ISIS Professional) and 2D (ISIS 2D) numerical solvers in the ISIS Software Suite were used in combination (linked) specifically for estimating peak water levels to support the generation of flood outlines.

4.2.1. Numerical software

The 1D models were built using ISIS Professional (version 3.3). The 2D components of the linked 1D/2D models were built and run in ISIS2D (version 1.0).

The 1D models were constructed using either ISIS Mapper and/or ISIS-GIS. ISIS Mapper is part of the ISIS Software Suite and ISIS-GIS is an ArcView add-on. Both these tools allow for pre and post processing of the ISIS models. Used in conjunction with a DTM they create new geo-referenced ISIS model data, such as reservoirs and spill lines. They can also be used to import models that already include geo-referenced data and to extend this information (cross-sections) using the DTM.

ISIS 2D uses two solution methods for calculating the required output parameters (water level etc): "ADI" (Alternating Direction Implicit) or "TVD" (Total Variation Diminishing). The ADI solver is the standard method of calculation for the majority of applications and has been adopted for all the 2D models in this study. The TVD solver allows complex hydraulics (e.g. dam breaks) to be calculated more accurately but has a significantly increased run time compared to the ADI solver. The TVD solver has not been used for any of the 2D modelling.

ISIS2D model construction was undertaken in GIS software such as ArcView and ISIS Mapper, enabling direct geo-referenced visualisation of all model elements. This facilitates the identification of model elements, visual inspection of model schematisation and rapid production of flood mapping.

4.3. Model extents

Only the watercourses defined as HPWs and MPWs were included in the hydraulic modelling. The extents of the modelled HPW and MPW watercourses are shown Figure 1.

A total of 20 river models were developed. The river reaches included within these models and their associated floodplains are summarised in Table 4-1~~Table 4-1Table 4-1~~, together with the model type.

Table 4-1 River reaches and model types

Model	Model Name	HPW Length (km)	MPW Length (km)	Model Type
1	Broadmeadow and Ward Rivers (BRO_WAR)*	57.6	35.1	1D – 2D
2	River Nanny (NAN)	12.5	35.9	1D – 2D
3	Lissenhall Stream (LIS)	4.4	-	1D
4	Turvey River (TUR)	5.4	-	1D – 2D
5	Rushroad Stream (RUR)	-	2.2	1D
6	Mosney Stream (MOS)	1.4	3.3	1D – 2D
7	Delvin River (DEL)	11.7	15.5	1D – 2D
8	Brookside Stream (BSS)	3.0	-	1D – 2D
9	Ballyboghil and Corduff Rivers (BAL_COR)*	8.8	16.3	1D – 2D
10	Balbriggan North Stream (BNS)	3.1	-	1D – 2D
11	Bracken River (BRA)	10.5	3.6	1D – 2D
12	Mill Stream (MIL)	3.2	1.0	1D – 2D
13	Gaybrook Stream (GAY)	5.7	-	1D – 2D
14	Mayne River (MAY)	11.3	11.3	1D – 2D
15	Sluice River (SLU)	16.7	5.1	1D – 2D
16	St Catherine's Stream (CAT)	1.2	1.2	1D – 2D
17	Baleally Stream (BAY)	2.0	2.8	1D – 2D
18	Bride's Stream and Jone's	1.9	6.0	1D – 2D

Model	Model Name	HPW Length (km)	MPW Length (km)	Model Type
	Stream (BRI_JON)*			
19	Rush Town Stream (RUT)	2.1	0.6	1D
20	Rush West Stream (RSW)	1.9	0.6	1D – 2D

** Where two river reaches merged before discharging to the Irish Sea, one hydraulic model was developed to represent this system.*

Details of the 1D reaches and 1D - 2D reaches for each watercourse are given in Chapter 5.

4.4. Common methodology

4.4.1. Model build

Modelling approach

The brief sets out that modelling of in-bank fluvial reaches for HPWs and in-bank and floodplain modelling for channels and floodplains of MPWs may require only 1D modelling and that out-of-bank fluvial (floodplain) modelling for all HPWs shall be undertaken using 2D modelling or other types of modelling capable of accurately simulating the 2-dimensional propagation of flow. Before determining the most appropriate approach to modelling each watercourse a thorough review was undertaken of the characteristics of each watercourse (including stream size, catchment steepness, floodplain, structures, urban areas etc).

A combination of 1D only and linked 1D-2D hydraulic models were used to model the watercourses in the study area. 1D only models were used when the flow paths could be reasonably well represented with a 1D approach under the following situations:

- The watercourses had a constrained flow path (i.e. narrow river corridor); and
- The out-of-bank flows were reasonably parallel to the river corridor (i.e. parallel contour lines).

Due to the requirement in the brief for all HPW floodplain flow to be modelled in 2D, all HPW watercourses were reviewed thoroughly. However, at some locations it was not appropriate to use a 2D modelling approach due to the following factors:

- Steep slopes, especially within the upper catchment, potentially causing instability in the 2D domain; and
- Very narrow ditch-like watercourses, especially within the upper catchment, in comparison to a reasonable computational cell size (i.e. 2- 5m) and therefore not practical to model with a 2D domain.

Based on the above factors and best practice, a 1D approach was adopted for all MPWs and some HPWs except in populated areas and/or where the flow path could not be well represented by a 1D model. Figure 1, at the back of this report, details the HPW and MPW watercourses, APSRs and 2D domains.

Digital Terrain Model

To facilitate the modelling of flood flow over the floodplains, a DTM was created for the whole project area. The data obtained and used is described in Section 2.6 of this report.

Schematisation

An appropriate model schematisation was selected for each model to represent the channel and floodplain conveyance as follows:

- For HPWs the cross section spacing is approximately 50 – 100m between sections. A 2D approach was adopted to represent the floodplain flow pattern except where it was inappropriate (refer to section on modelling approach above); and
- For MPWs the cross-section spacing is approximately 750m between sections with reduced spacing at structures. For MPWs and some HPWs a 1D approach was adopted in order to represent the floodplain flow pattern. For some MPWs a 2D modelling approach was adopted where the flow path pattern on the floodplain could not be well represented using a 1D model.

For each 2D model a cell size of 2m or 5m was used depending on the size of the model. The cell sizes of 2D domains need to be sufficiently small to reproduce the hydraulic behaviour and accurately model peak flood levels but without excessive run times. It is considered that:

- 2m is the minimum cell size because the DTM is a 2m grid cell size; and
- 5m is the maximum cell size to appropriately represent the flow over the roads (main flood path within the urban areas).

The DTM was modified in some circumstances in order to represent flow paths (e.g. gaps in embankments) and obstacles (e.g. defences) in the 2D domain(s). Using photographs and site visit notes, a number of structures were identified which required manual adjustment of the DTM. For example, in order to model flows through the railway underpass in Malahide, it was necessary to modify the DTM levels to match the access road levels east and west of the railway embankment. Further details on any modifications to the DTM are detailed in the individual model sections in Chapter 5.

Each model reporting section (Chapter 5) contains a summary of the schematisation for each model.

Channel cross-sections

Channel cross-sections were modelled using ISIS river section units and the elevation data collected from the channel survey, which is described in Section 2.4 of this report.

Structures

Topographic surveys have been undertaken on weirs, culverts and bridges with cross-sections specified at the upstream face of the structure if both faces were similar, and at upstream and downstream faces if there were significant differences between faces. The generic assumptions for modelling structures are discussed in Section 4.5.2 and when required, specific details are included in the individual model sections in Chapter 5.

Channel, floodplain and structures roughness values

The Manning's n coefficient, which is used in the ISIS model to represent the channel and floodplain roughness, was selected based on the site conditions (i.e. channel and structure photographs, aerial photographs, OSI mapping and site visit notes) and compared to published references (Chow, 1959). Table 4-2 presents a sample of the typical Mannings values used in the 1D and 2D model domains. For the 2D domains, a GIS layer was used for setting Manning's n values in order to define material zones (land-use) in the floodplain. The Manning's n values for the 1D cross sections are represented in the 1D model domain.

Table 4-2 Sample Manning's n values used in the 1D and 2D model domains

1D Manning's n values	2D Manning's n values
Main channel. Clean, straight, full stage, no rifts or deep pools, 0.030	Asphalt, 0.050
Main channel. Clean, winding, some pools and shoals, 0.040	Buildings, 1.000
Main channel. Same as above, but some weeds and stones, 0.045	Gardens, 0.500
Floodplains. Pasture, short grass, 0.030	Green spaces, 0.050
Floodplains. Scattered Brush, heavy weeds, 0.050	River, 0.030
Floodplains. Medium to dense brush in Summer 0.100	Roads, 0.050

The Colebrook White coefficient is used in the ISIS model to represent the culvert roughness. The value selected was based on the condition of the culverts and published references (HR Wallingford, Charts for the hydraulic desigh of channels and pipes, 5th edition). A sample of typical coefficients are listed below:

- Concrete (good condition) 0.002m;
- Concrete (poor condition) 0.006m; and
- Corregated metal pipe 0.03m.

Photographs of the culverts and examples of the highest/lowest open channel and floodplain section roughness are detailed in each of the individual model sections in Chapter 5.

In addition, the Manning's n coefficients were adjusted as part of the calibration process where historical and/or recorded data were available (refer to Section 4.4.3).

4.4.2. Boundary conditions

The hydraulic models have different types of boundary condition as follows:

- Hydrological inflow boundaries;
- Tidal boundaries (at the downstream end of the models); and
- Water level elevations abstracted from downstream river models.

The model inflows have been represented using ISIS FSSR16 boundary units, as described in Section 3.2 of this report. The FSSR16 Boundary Method (FSSR16BDY) derives an inflow hydrograph from a catchment or sub-catchment. The hydrograph then becomes a boundary condition equivalent to a Flow Time Boundary (QT).

Tidal boundaries were generated at 13 locations by MarCon Computation Ltd (please refer to Section 6.3.2 for further details) for the return periods for which predicted extreme tide levels were provided (i.e. for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1% AEP). These boundary conditions were applied at the downstream end(s) of the river models using the Stage Time Boundary (HT). The HT boundary allows the input of a stage hydrograph as a boundary condition by specifying (i) water levels above datum and (ii) time.

4.4.3. Hydraulic model calibration

Model calibration, where data supports this, is achieved through carrying out simulations of recorded flood events and then making adjustments to the hydraulic model parameters through the comparison of observed and modelled results. Often the variables are quite interdependent, but are also not necessarily constant between event periods, so it is preferable to use more than one event to provide a comparison and an indication of parameter variability.

It is important to recognise that modelling and model calibration it is not an exact science and that the modelling tools are designed to enable a wide range of natural systems to be represented. Consequently the process of model calibration is to achieve, where possible, a good match between the observed event and the model prediction. Calibration depends on several factors, such as:

- The amount of data available for each event;
- The reliability of the recorded data sets;
- The extent of suitable event records, and
- The underlying data used within and the schematisation of the hydraulic model.

There are nine hydrometric stations in the study area which are located on six rivers (Sluice, Broadmeadow, Ward, Ballyboghil, Delvin and Nanny). Out of the nine gauging stations, three major stations (08005-Sluice, 08009-Ward and 08008-Broadmeadow) are located close to the downstream boundary of the respective hydraulic model. Seven out of the nine hydrometric stations in the study area were closed down by the EPA between 1995 and 2001. Hydrometric stations on two other rivers, the Delvin and the Garristown, were made operational in November/December 2009. However, the very short data period was considered not useful for hydraulic model calibration. Therefore, limited gauged data was available for calibration purposes.

Reports and photographs detailing the extent of flooding in the study area are available for the recent flood events; however most of the gauging stations were non-operational during these events. Therefore it was very difficult to identify events that had both observed stage data at the gauging stations and other observed flood levels and/or photos for model calibration.

The gauges and flood event dates for which suitable calibration data was available are listed in Table 4-3~~Table 4-3Table 4-3~~.

Table 4-3 Selected calibration events for the gauged catchments

Station	River	Date of flood	Other flood data
08005	Sluice	26/06/1986	
08007 and 08008	Broadmeadow	26/08/1986	
08011	Nanny	06/11/2000	Water level upstream of the Ashbourne Road Bridge = 20.81 to 20.90 m OD; upstream of the Drogheda Road Bridge = 20.04 to 20.16m OD; at Beaumont Bridge = 14.60m OD (Malin Head).
		26/08/1986	
		12/06/1993	
08012	Ballyboghil	26/08/1986 1972	

Flood hydrographs at the gauging stations listed for the above events were derived from recorded water levels using the revised rating curves (refer to FEM FRAMS Hydrology Report, 2010, for further information).

The hydraulic models were calibrated to the peak flows and levels at the gauging stations (GSs) as well as to recorded flood marks at other locations in the river listed in Table 4-3~~Table 4-3Table 4-3~~. As discussed in Section 3.3 (refer to Table 3-1), concurrent rainfall data and flow data was not available for the selected major events for calibrating the model. For example, in the case of the Nanny River, although daily rainfall data was available at Duleek station for the period of 1949 – 1991, no rainfall record was available for the month of August 1986, and hence the Hurricane Charlie event could not be modelled. Thus, it was not possible to use rainfall derived flow hydrographs for model calibration and design flood hydrographs were used instead. This approach has been successfully used on previous projects where the availability of concurrent rainfall and hydrometric data is poor. The model calibration was undertaken in the following two steps:

Step 1 – flow calibration

The ISIS FSSR 16 inflow units of the closest AEP event to that of the selected historic flood at the GS (for which a full hydrograph is available) were used. The purpose of using the ISIS FSSR 16 units was to distribute the single observed flow hydrograph available at the GS into several inflows for the upstream sub-catchments such that the resulting model peak flow at the GS would match the corresponding observed peak flow at the GS.

The model and observed flow at the GS was compared. If the model flow did not match the observed flow, then the full hydrograph scaling factor in the ISIS FSSR 16 units were adjusted globally until the model flow at the GS matched the observed flow.

The focus for the flow calibration was on matching the peak flows at the gauging stations by adjusting the full hydrograph scaling factor in the ISIS FSSR 16 units globally. The matching of the flood volumes was not undertaken as there is no data available to justify adjusting the parameters of the ISIS FSSR 16 unit for one upstream sub-catchment differently to the ISIS FSSR 16 unit for another sub sub-catchment in order to match the volumes at the gauge.

Step 2 – level calibration

Once a reasonable match between the modelled and observed flows at the GSs was achieved (Step 1), the hydraulic model was calibrated to observed levels. This was undertaken by matching the model water levels with the corresponding observed water levels at the GS and other locations by changing the Manning's n value, on both 1D and 2D domains, and uniformly for the entire catchment.

The global scaling of the full hydrograph (step 1) resulted in bigger changes in flows at the gauging stations than adjustments in the Manning's n values. As the Manning's n values in the channel and floodplain are based on the site conditions compared to published references (refer to Section 4.4.1), limited adjustments of these roughness values were required to match the levels at the gauging stations (step 2). The limited adjustments to the roughness values resulted in negligible changes in flows at the gauging stations and no other adjustments to the scaling of flows following step 2 was required. Further information on the model calibration for the individual river models is reported in Chapter 5.

4.4.4. Hydraulic model verification

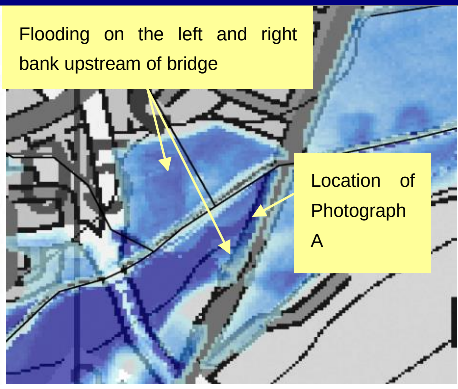
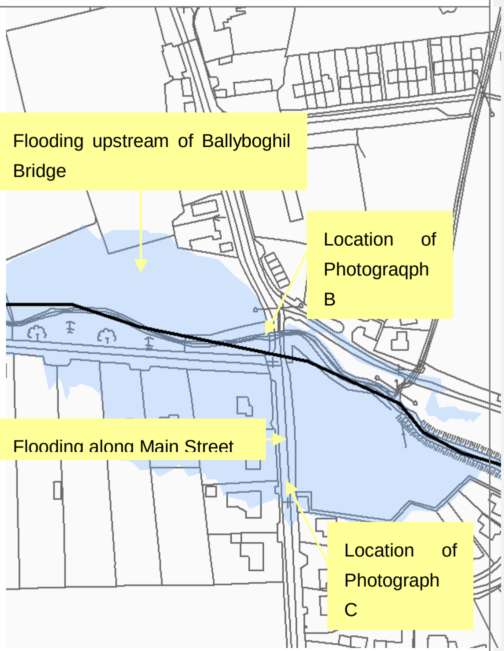
Given the limited data available for model calibration, model verification was carried out using historical flood information and by holding flood mapping workshops.

Historical flood information checks

Historical flood information was used to check that the models were predicting flooding in locations with a history of flooding. A full review was undertaken of available flood reports, photos and other information on the OPW flood mapping website and reports made available from other sources. In the majority of cases, there was limited detailed information available in terms of water levels, flood extents and detailed flood mechanisms. However, the information did provide details of locations with historical flood risk. Using GIS layers of historic floods, the hydraulic modellers carried out a check on areas where spilling/surcharging should be expected in the hydraulic models. Appendix C1 contains a map showing the location of historical flood events against modelled flood outlines.

Where more detailed information was available, additional checks were carried out to compare the model results to the available data. Table 4-4~~Table 4-4Table 4-4~~ shows a sample of the data used for verifying the models. The majority of this verification data can be found on the OPW flood mapping website (www.floodmaps.ie).

Table 4-4 Sample of data used for model verification

Historical data	Model data
	
Photograph A. Flooding upstream of N2 road bridge on left and right banks of Nanny River during November 2000 floods	*Modelled flood depth map for November 2000 flood showing flooding in the areas identified in the photograph*
 *Photograph B Flooding upstream of Ballyboghil Bridge for November 2002 floods* 	
Photograph C Flooding along Main Street, Ballyboghil, for November 2002 floods	*Map of the 1% AEP current scenario flood extent in Ballyboghil*

Flood map workshops

Draft flood maps were prepared and reviewed at two workshops (14 December 2009 and 9 March 2010) by the water and transport area engineers from FCC and MCC and the area engineers from the OPW. The draft flood maps showed the historic flood locations and the flood extent for the 10% AEP and 1% AEP (fluvial) and 0.5% AEP (tidal) events. The maps were generally considered to be representative of the flooding experienced in the study area. In some locations some further investigation or review of the survey data was undertaken and minor changes were made to update some of the hydraulic models. The workshops also identified that additional rivers/tributaries should be included in the study as there was a history of flooding. As a result the tributary to the north of Ashbourne was included in the study. These workshops and the review procedure also helped to verify the hydraulic models.

4.4.5. Joint probability analysis

A fluvial/tidal joint probability analysis was carried out as part of this study, as described in Chapter 8 of the Final Hydrology Report (February 2010). Additional research on this topic including the review of JPA recommendations from other studies, dependency assessment and sensitivity analyses involving further simulations of the hydraulic models was undertaken during the hydraulic analysis. The results of this research and sensitivity analysis are in the JPA Technical Note included in Appendix C2 and summarised below.

- Detailed dependence mapping of variables is not available for Ireland. In addition, the quality and length of flow data and tidal records is not sufficient to enable a meaningful or robust correlation between them to be assessed;
- The assessment made for the joint probability design scenarios was based on the best available information on dependence from the Defra/EA study which estimates dependence between river flow and surge;
- Reference was also made to the Greater Dublin Strategic Drainage Study, discussions with HR Wallingford and Dr Michael Bruen (UCD);
- The JPA TN considered geographical influences on dependency and climate change on dependency and concluded that a conservative estimate of dependence of χ =0.2 is considered appropriate for the Fingal East Meath catchments;
- A sensitivity analysis was undertaken for three different dependence values, namely χ =0.2, χ =0.1 and χ = 0.05. Joint exceedence curves for various annual exceedence probability events and a design event combination table were produced for each dependence value;
- Table 8.4 in the JPA TN summarised the 1% AEP fluvial combinations for different dependency values. It was noted that the 1% AEP fluvial combination for χ =0.1 of 2% AEP fluvial/50% AEP tidal is the same combination as the 2% AEP fluvial combination for χ =0.2 for which we already have results. In addition, the combination for χ =0.05 of 10% fluvial/50% tidal was considered to be too low (particularly when considering recommendations from other studies in this catchment);
- Additional modelling of the Lissenhall, Ballyboghil, Broadmeadow, Corduff, Nanny and Ward models with long sections showing the maximum stage results for the

fluvial Q(1%) / T(20%), Q(1%) / T(MHWS) model runs; the tidal Q(20%) / T(1%) water level profile was also shown so that the fluvial/tidal transition point could be assessed for the two fluvial conditions;

- The JP combination in the Ward model does not affect the flood maps; for the Corduff, Ballyboghil and Nanny models the JP combination may impact the flood maps in the tidally dominated zone; for the Lissenhall and Broadmeadow models the JP combinations has a greater impact on the fluvial flood maps in the tidally dominated zone. The TN concluded that a cautionary approach is considered more appropriate where there is the potential for risk to life or where there is a lack of supporting data; and
- Table 4-5~~Table 4-5Table 4-5~~ shows the sixteen scenarios for a dependence of χ =0.2, combining fluvial and tidal events, which were simulated. In the odd numbered scenarios the fluvial component is dominant; in the even numbered scenarios the dominant component is tidal.

Table 4-5 Joint probability scenarios (χ =0.2)

Scenario N°*	Design Event (AEP)	Boundary AEP	
		Fluvial Boundary	Tidal Boundary
1& 2	50% (2 year)	50%	50%
3	20% (5 year)	20%	50%
4	20% (5 year)	50%	20%
5	10% (10 year)	10%	50%
6	10% (10 year)	50%	10%
7	4% (25 year)	4%	50%
8	4% (25 year)	50%	4%
9	2% (50 year)	2%	50%
10	2% (50 year)	50%	2%
11	1% (100 year)	1%	20%
12	1% (100 year)	20%	1%
13	0.5% (200 year)	0.5%	10%
14	0.5% (200 year)	10%	0.5%
15	0.1% (1000 year)	1%	2%
16	0.1% (1000 year)	2%	0.1%

**The joint probability scenarios referenced above are also referenced on the long section profiles in the digital deliverable (Volume 3) for each of the rivers i.e. S01 is the first scenario above.*

4.4.6. 'With' and 'without' defences

As part of the study, a defence asset schedule and database, the FDAD, has been created from the DAS (refer to Section 2.5). The data contained in this FDAD has informed where defences are included in the hydraulic models. In addition to this FDAD data, the following information was also available to inform where defences should be included in the models:

- Channel and structure cross sections;
- Left bank, right bank, upstream and downstream photographs at each cross section;
- Aerial photography;
- LiDAR DTM;
- Information from walk over surveys; and
- Information made available from the client.

The defences included in the models are classified under two different types of defences:

1. Formal defences (e.g. flood defence embankments and walls in Duleek which were constructed as part of a flood alleviation scheme for Duleek) and;
2. Informal effective defences (e.g. embankments at the Somerville housing development in Ratoath).

All of the formal and informal effective defences along the watercourses and the Meath and Fingal coastlines have been modelled with a 'with defences' and a 'without defences' scenario. For the 'with defences' scenario, the majority of the defences have been added to the model with either the linked ISIS 1D and 2D HX lines or a 2D domain Z lines feature; the model schematisation and/or model performance determining the approached being adopted. The exception to this was the flapped tidal outfalls, which were modelled with a suitable ISIS model unit (please refer to Chapter 5 for further details).

The elevation and dimensions of these defences were obtained from the following sources:

- Topographical survey of defences in the FDAD;
- Information on coastal defences in Fingal provided by the OPW;
- LiDAR data; and
- Channel and structure cross section data.

The 'without defences' scenario was used to identify the 'areas benefiting from defences' (ABDs) which are shown on the flood hazard maps (refer to Chapter 7). For the 'without defences' models, all of the defences in the hydraulic models are removed allowing flood water to spread, unhindered, into areas protected by these defences. This allows areas and properties which are currently protected by the defences to be clearly identified.

Table 4-6~~Table 4-6Table 4-6~~ lists the defence locations within the study area which have been removed in order to develop the 'without defences' models. Using all of the information available to the modellers, a comment on whether these defences are 'formal' or 'informal effective' flood defences is also included in the table.

Table 4-6 Defence locations

N°	Waterbody	Defences	Defence classification
1	Broadmeadow	Raised defence between 4Ba21573 and confluence with 4Bay142	Informal effective
2	Broadmeadow tributary at Ashbourne	No raised formal defences along this reach but garden/property walls provide some protection	Informal effective
3	Turvey	Flapped outfall tidal defence.	Formal
4	Nanny and Paramadden at Duleek	Earth embankment and concrete walls at Duleek along the left bank of the Nanny River and both banks of its tributary, the Paramadden	Formal
5	Bracken	Some protection provided by garden/property walls along the downstream reach (approx. 300m) (i) RB u/s R132 bridge (ii) & (iii) LB & RB d/s R132 bridge	Informal effective
6	Skerries	LB & RB walls u/s of Holmpatrick Road along Millers Lane	Informal effective
7	Mayne	Flapped outfall tidal defence	Formal
8	Sluice	Flapped outfall tidal defence	Formal
9	Coastal	Combination of defences along the coast including natural sand dunes, quay walls and walls	Formal and informal effective

Further discussion on the defences included in the river models and the coastal model are detailed in the individual model report sections (Chapter 5 and Chapter 6). Appendix C3 contains additional information and maps showing the locations of the defences.

Not all of the defences included in the hydraulic model have been surveyed as part of the DAS, for example, the formal flood defences in Duleek. Recommendations for additional assets to be included in the FDAD are included in Section 13.3.1.

In addition to these formal and informal effective defences, structures located in the river floodplains which alter the direction of flow and hence the extents of flooding have also been included in the model. These structures are not considered as defences and have not been included in the 'without defences' scenario. However they do influence the direction of flow in the floodplain without necessarily resulting in a build up of flood water behind them or providing any protection to properties. For example, directly downstream of the railway underpass in Skerries, walls alongside the R127 at the junction of Dublin Road and Miller's Lane prevent flood water from entering the park and divert the flood water along Miller's Lane. Further details on the location of these structures are discussed in the river model report chapter (Chapter 5) and Appendix C3.

Defence failure modelling has also been undertaken as part of the study, details of which are contained in Chapter 7.

4.4.7. Model simulations

The selection of the model run parameters is important for the success of the hydraulic model. All model parameters have been left within their standard range, unless stated in the modelling section for the relevant river model.

Linked 1D-2D models can be prone to instability when the distance between river sections is too large (greater than 100m) or too small (generally less than 5m). In models where there are long distances between surveyed sections, interpolated sections have been used to improve the model stability.

Unsteady flow simulations were carried out as the majority of the models contain a tidal boundary and the floodplain flow and volume needed to be evaluated.

The fluvial and tidal events modelled include 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1% AEP events. These events were modelled for the 'with' and 'without' defences scenarios and for the current and mid-range future scenarios (MRFS). The high-end future scenarios (HEFS) were modelled for the 10%, 1% and 0.1% AEP events.

Critical storm duration

For both gauged and ungauged catchments, the critical (design) storm durations (i.e. the storm duration resulting in the maximum water level) in the FSSR 16 boundary units were optimised using iterative simulations in the hydraulic models based on the 1% AEP current scenario fluvial event. The CSD for the 1% AEP event will not necessarily be the CSD for other AEP events. However, it is the standard approach for catchment based studies to adopt the CSD for the 1% AEP event for the remaining AEP events. The results of the 1% AEP event are used, for example, in preparing Zone A of the flood zone maps (refer to Section 7.3.2) and the 1% AEP event is the typical design standard adopted for fluvial flood defences. The following steps provide a summary of the method used for determing the CSD:

1. 1D runs were carried out for a range storm durations for the 1% AEP event, with an initial estimate of the critical storm duration based on the outputs of the hydrological assessment which used the FSSR16 methodology;

2. The critical storm duration resulting in the maximum water level was identified for each model node. The most frequently occurring critical storm duration for the HPWs in the model was adopted for potential use as the storm duration for the entire model; and

3. To assess the impact of adopting a single storm duration for the full model reach, the difference between the maximum level at each node (i.e. resulting from the critical storm duration to that node) and the level of the storm duration adopted at step 2 was calculated. If the difference between the water levels was less than 0.2m then the adopted storm duration was used in the model. If the water levels in any particular HPW reach were greater than 0.2m then a different storm duration was used for that catchment.

Table 4-7 shows the frequency analysis of water level differences Δ (maximum water level – adopted duration water level) for the Broadmeadow River. The results show that for 99% of the model nodes (cross sections) the difference is less than 0.05m, when all the sections of the model are considered. The analysis for cross-sections located in high priority watercourses also shows that in 99% of cases the difference is less than 0.05m.

Table 4-7 Frequency analysis of water level differences (1D model runs) along the Broadmeadow River

Δ (m)	**HPWs & MPWs**		**HPWs only**	
	No. of nodes	Frequency	No. of nodes	Frequency
0.02	6182	93.61%	1772	93.41%
0.05	369	5.59%	106	5.59%
0.1	28	0.42%	17	0.90%
0.15	18	0.27%	2	0.11%
0.2	7	0.11%	0	0.00%
0.5	0	0.00%	0	0.00%
1.0	0	0.00%	0	0.00%
Total	6604	100%	1897	100%

Table 4-8 shows the average and maximum difference between the maximum water level at each node (i.e. resulting from the critical storm duration to that node) and the water level at each node for the adopted storm duration. The results are presented for all of the remaining river models.

Table 4-8 Water level differences between critical storm duration for each node and adopted storm duration

Model	**Storm duration (hours)**	**Average water level difference (m)**	**Maximum water level difference (m)**
BRA	21	0.00	0.06
BSS	9	0.00	0.02
GAY	4	0.00	0.08
LIS	11	0.02	0.18
MOS	17	0.00	0.01
NAN	15	0.02	0.11
RUR	18	0.00	0.00
RUT	17	0.00	0.02
RWS	6.5	0.03	0.13
CAT	21	0.00	0.04
BAL-COR	15	0.00	0.04
BAY	6.5	0.00	0.03
BNS	2.5	0.00	0.03
BRI-JON	15	0.00	0.00
DEL	23	0.02	0.13
MAY	5.5	0.00	0.05
MIL	18	0.01	0.07
SLU	12	0.01	0.13
TUR	9	0.05	0.20

A review of the storm durations shows a large variation in the storm duration for the various models. The modelled storm duration is dependent on a number of factors such as the design inflow hydrograph, channel slope, length of channel, presence of structures and floodplain storage. These factors will vary from model to model and explains the differences between various model storm durations.

A review of the water level differences in the river indicates that models are generally not sensitive to changes in storm duration. The critical storm durations obtained from the hydraulic models for each of the rivers are provided in each river model reporting section (Chapter 5). Note that some critical storm durations adopted for the models are significantly longer than that identified in the FSSR boundary. These increases in storm duration can be explained by hydrodynamic modelling factors such as floodplain storage and channel conveyance and the methodology used to identify the critical storm duration at each node (automatically identifying the maximum water level for each duration modelled).

The results of the CSD show that the models are not sensitive to storm duration (different storm durations result in only small differences in levels).The full results of this assessment are in Volume 3 (digital deliverables).

Model reconciliation

The design flows at the eight gauging stations estimated using the statistical method are presented in Table 5-6 of the Final Hydrology Report and reproduced in Appendix B. However, these design flow values were not used directly in the hydraulic models. For the gauged catchments, the modelled (routed) flows at the gauging stations were reconciled with the design event flows estimated from the statistical method, through iterative simulations of the hydraulic models, to ensure that the modelled flows at the gauges matched the flows estimated using the statistical method. The methodology involved in the reconciliation is summarised in Chapter 6 of the Final Hydrology Report.

The scaling factors obtained from the reconciliation of each of the gauged catchments are presented in Table 4-9~~Table 4-9Table 4-9~~. The scaling factors are based on the scaling of the full flow hydrograph.

Table 4-9 Scaling factors from model reconciliation

Rivers and gauging stations	Annual Exceedence Probability							
	50%	20%	10%	4%	2%	1%	0.5%	0.1%
08011 Duleek on the Nanny	1.57	1.62	1.65	1.70	1.73	1.76	1.79	1.87
08010 Garristown on Garristown Stream	0.72	0.86	0.94	1.02	1.06	1.10	1.13	1.13
08007 Ashbourne on the Broadmeadow	1.00	1.09	1.16	1.26	1.33	1.40	1.47	1.64
08008 Broadmeadow on the Broadmeadow	0.74	0.82	0.89	0.97	1.04	1.10	1.16	1.31
08009 Balheary on the Ward	1.45	1.53	1.59	1.68	1.74	1.80	1.86	2.01
08012 Ballyboghil on the Ballyboghil	0.96	1.08	1.12	1.16	1.17	1.19	1.19	1.13

Rivers and gauging stations	**Annual Exceedence Probability**							
	50%	20%	10%	4%	2%	1%	0.5%	0.1%
Average Scaling Factor (excluding Delvin and Sluice)	**1.07**	**1.17**	**1.23**	**1.30**	**1.35**	**1.39**	**1.43**	**1.51**
08002 Naul on the Delvin	0.49	0.58	0.62	0.66	0.68	0.71	0.73	0.73
08005 Kinsaley Hall on the Sluice	0.81	1.04	1.13	1.19	1.20	1.22	1.19	1.13

The study area average scaling factor (refer to third last row in Table 4-9~~Table 4-9Table 4-9~~ above) for each AEP event was calculated from the average of the scaling factors of six stations (excluding the Delvin and Sluice gauges). For the ungauged catchments, the study area average scaling factors were used.

The scaling factors for the River Delvin were excluded because the values were much lower than those of the rest of the catchments for all AEPs, and considered non-representative. Similarly, the scaling factors for the Sluice River were also excluded for calculating study area average scaling factor, because the design flows for this river were calculated using a site specific growth curve instead of the regional growth curve. This is because the Sluice catchment area consists of more significant urbanisation than most of the other catchments in the study area (refer to Appendix C2.2 of Hydrology Report, Halcrow Barry, 2010). Thus the exclusion of the Sluice River scaling factors is considered to produce a more representative study area average scaling factor to be used for the ungauged catchments.

4.4.8. Model sensitivity tests

Sensitivity runs were carried out in order to predict and assess the impact on flood levels of a ±20% change to:

- Roughness (Manning's n) for the 1D and 2D domains; and
- Design flows.

In each case the sensitivity run was carried out for the 1% AEP fluvial event.

In addition, sensitivity runs were carried out in order to predict and assess the impact on flood levels of a ±0.25m change to the downstream boundary condition (tidal). In this case the sensitivity run was carried out for the 0.5% AEP tidal event.

Since the 1D and 2D models are run together the sensitivity test for design flows and the downstream boundaries affects both domains (1D and 2D). The results of the sensitivity tests are presented in each of the individual model sections in Chapter 5.

Overall, the results of the sensitivity analyses are generally within acceptable limits. As expected the models were generally more sensitive to variations in inflows and downstream boundaries than to changes in roughness values. The biggest impacts occurred upstream of structures due to constrictions in flow areas/culvert capacities. Further details on the model sensitivity results are reported in each of the river model sections in Chapter 5.

4.4.9. Model stability

In order to achieve model stability, a number of methods have been used by the hydraulic modellers and include:

- Reducing model run time steps (however this significantly increases the model run times);
- Addition of ISIS interpolated sections to improve the model stability when the distance between river sections is large (i.e. greater than 100m);
- Addition of ISIS in-channel "Spill" units to improve model stability along steep channel reaches or where there was a significant change in bed levels.
- For low flow situations, the base flow is increased when the channel runs dry; and
- When culverts become surcharged, a narrow slot is added to the invert of the culvert to improve model stability when ISIS changes the unit mode from drowned weir flow through the culvert to orifice flow.

While the majority of instabilities were eliminated it was not possible to fully eliminate all of the model instabilities from all of the models. In order to evaluate the stability conditions of the models, a non-convergence analysis was performed using the ISIS Inquisitor software. The diagnostics analysis allowed non-convergence errors to be ranked in order of the number of times they occur. Analysis of the results (included in Appendix C4) indicates that the most common causes of instability resulted from:

- Changes to the ISIS unit mode calculation (e.g. an orifice unit would cause instabilities when switching from drowned weir flow calculations through a culvert to orifice flow calculations); and
- Instabilities in lateral spill units that occur when the flow spills out of the river channel and back into the river channel due to similar water levels either side of the spill unit (i.e. in the river channel and floodplain).

The analysis shows that 14 of the models had non convergence errors. Typically these non convergence errors represent less than 1% of the model run times and occur outside of the peak of the flood event. Therefore, the instabilities encountered are within normal bounds for hydraulic models of this level of complexity and do not affect the accuracy of the model results.

4.4.10. Quality assurance

Halcrow Barry integrates its requirements for quality, health & safety and environmental management, and other aspects of the business, in a single system based on the Halcrow integrated management system (HIMS). HIMS establishes and maintains an economic and effective framework to provide assurance that the services provided by Halcrow Barry will meet the requirements of clients, international standards, legislation and Halcrow Barry management.

Senior engineers were involved in and supervised the hydraulic model build. Guidance documents have been used by the modellers to ensure a consistent and auditable approach to project work and to ensure use of our collective hydraulic modelling knowledge. Throughout the hydraulic model build, quality checks have been undertaken to ensure that the models accurately represent the river and coastal systems. A mass balance check has been undertaken on the models to ensure that the volume of water exiting the system is comparable to the volume of water entering the system in the 1D model domain.

The Froude number gives an indication of whether the flow is subcritical (Fr < 1) or supercritical (Fr > 1). This shows whether energy is transferred upstream and downstream (subcritical) or just downstream (supercritical). The maximum Froude Numbers (Fr) have been checked and any unexpected behaviour has been investigated and resolved.

The flood map outputs have been checked with the model outputs. A comparison of the top wetted widths (maximum water level width) was carried out between the model output and with the flood width of the map throughout the length of the river. A review of the flood maps has also been carried out in conjunction with the model to ensure that the flood flow routes are accurate.

The quality checks also include spreadsheets and the outputs from tools developed by the ISIS development team. These checks are reported on in a series of spreadsheets available in Volume 3 of the report and include:

- Survey check spreadsheets;
- ISIS report files; and
- ISIS run log files.

Before the start of the model build, a comparison was made between the survey data and the DTM (used in model construction and flood spreading) and during the model build. In addition, a number of other checks were undertaken as detailed below. These checks are reported on in the survey check spreadsheets available in Volume 3 of the report.

- Spacing between cross sections (50m -100m for HPW, 700m-1000m for MPW);
- Structures information, u/s and d/s data, structure length, etc;
- Cross section locations;
- Cross section photographs;
- Cross sections drawings; and
- Cross sections bank and extremity levels checked with DTM.

ISIS report files have been prepared for the full range of modelled scenarios (i.e. current, MRFS, HEFS, blockages, etc). These files contain a full quality assurance and audit trail of the model run including the model name, storm duration, run time and date.

ISIS run log files have been prepared for each model and provide details of the various stages of the model build including changes to the model .dat files and model run details. The log also includes links to the ISIS .dat files and ISIS .ied files.

4.5. Assumptions

This section contains the generic assumptions used in the model development process. Modelling assumptions specific to each model can be found in the relevant report section (Section 5.2 to 5.21).

4.5.1. River cross-sections

Generic assumptions relating to river cross-sections are as follows:

- Values for distance step (dx) were calculated using the following rules:
    - Cross sections were generally not more than 20B apart, where B is the top width of the channel; and
    - Sections were generally not more than 1/(2s) apart, where s is the mean slope of the river.
- Where the topographic survey sections showed greater distances than these rules, interpolated sections were added to the model to improve model stability;
- In general, the distance between cross-sections was not less than 10m, for model stability reasons. River sections with a surveyed chainage of less than 5m were not included in the model unless there was a change in river profile or they were needed to represent a backwater profile accurately;
- Appropriate panel markers were set at changes in the bank slopes, typically at the top of bank but other points within the floodplains were considered. Panel markers are not primarily a means to define changes in roughness values, although significant changes in roughness values can only be set at the locations of the panel markers; and
- The label names for sections in the model come from the labels defined as part of the topographic survey.

4.5.2. Structures

Generic assumptions relating to structures are as follows:

- For bridges, surveyed river sections were available at the upstream face if the inlet and outlet were similar. The chainage between inlet and outlet sections was set to the width of the bridge if the distance was more than 10m;
- Discharge and velocity coefficients in bridge units were only changed when supported by observed water level data. When no observed data were available the default values were set to 1.0;
- Each bridge was associated with a spill unit to allow for spilling over the bridge under extreme conditions. These spill units normally extend over a longer distance than the bridge itself to represent flow paths adjacent to the bridge (if expected) or up to the 2D domain (if 1D-2D approach adopted). The spill coefficient used varies between 0.7 and 1.3 depending on the type of structure or ground conditions being represented (e.g. vegetated embankment or concrete crest). If a spill unit is being used for modelling flows over natural ground, which is less efficient than a flood bank, then weir coefficient values lower than 1.0 are used;
- The USBPR and arch Bridge units were used. Both units have been extensively tested and they represent the bridge afflux reasonably well when not surcharged;
- The orifice unit has been adopted to represent bridges where the surcharged flow should be calculated in a more representative manner;

- The sluice unit has been adopted for representing bridges where there is appreciable scour of the bed in the constriction. This unit allows the head losses produced by the difference in bed levels between the upstream face of the bridge and bridge opening to be represented;
- Weirs were generally represented by 'Round nosed broad crested weir' or 'Crump weir' units. Spill units were used for irregular shaped weirs and waterfalls. 'General purpose weir' units were not used, for model stability reasons;
- Long culverts (length > 20 conduit diameter/width) were modelled as conduit units and the roughness coefficient was selected based on the material of construction from the survey photographs, plus reference to publications (e.g. HR Wallingford, Charts for the hydraulic desigh of channels and pipes, 5th edition). The conduit equations are appropriate if the length of the conduit is longer than approximately 20 times the diameter;
- Short culverts (length < 20 times conduit diameter/width) were modelled as orifice or sluice units as appropriate based on the differences between the culvert invert level and the channel bed levels;
- The head losses at each structure were taken into account at the upstream face (inlet or orifice unit) and/or the downstream face (outlet unit) using the most appropriate ISIS unit depending on the structure, bed conditions, water levels and flow paths;
- Trash screens are generally not included in the models as the flow capacity through the trash screen is usually greater than the controlling flow through the culvert and therefore the trash screen will have a negligible impact on water levels. In addition, a number of trash screens only partly cover the culvert inlets and the non standard nature of some of the screens (i.e. mesh reinforcement) means it was not possible to accurately model these screens;
- The inlet and/or outlet information (material, shape, size, condition, etc.) was adopted for long culvert barrels. If the inlet and outlet were different a linear interpolation was used in order to characterise the conduit along its length. Additional information was received from the client for some structures which informed the modelling of those structures (refer to Section 2.4.2); and
- Flap valves were generally represented by orifice units with no reverse flow.

Full details of all the structures in the models are available in the digital deliverables in Volume 3.

4.5.3. Floodplains

Generic assumptions relating to floodplains are as follows:

- In 1D model areas floodplains were represented by cross-sections extending sufficiently far to contain the 0.1% AEP flow, unless river banks were more than 1m higher than the ground level behind the banks in which case flood storage areas ('Reservoir' units) or secondary channels ('River' units) were used in combination with 'Spill' units. Flood storage areas were interlinked using 'Floodplain section' or 'Spill' units with appropriate section data and distance values; and

- In combined 1D-2D hydrodynamic model areas the 2D domain was used to simulate the routing of fluvial flows over the floodplains.

Further details on the floodplain model units used in the river models are detailed in Chapter 5. Figure 1 provides a graphical presentation of the ISIS reservoir units and locations of the 2D model domains.

4.5.4. Confluences and bifurcations

Generic assumptions relating to confluences and bifurcations (junctions of rivers) are as follows:

- ISIS 'junction' units were used;
- If no survey section data was available at the junctions, river sections were determined from the nearest upstream and downstream surveyed sections. Changes in vertical elevation when copying these sections were allowed for such that the minimum bed level at all junction sections were identical, i.e. no sudden drop in bed level; and
- Energy lines are not equated at junction nodes, i.e. water levels were considered to be the same at each river section located at the junction.

4.5.5. Boundaries

Generic assumptions relating to boundary conditions are as follows:

- Upstream boundaries representing inflow hydrographs were represented using ISIS 'FSSR 16 boundary' units;
- Downstream boundaries were represented with 'Head-time boundary' units and 'Normal-head boundary' units;
- All time dependent boundaries were specified in external ISIS event data files (*.ied). Only base flows as 'Abstraction' units were specified within the ISIS model data file; and
- Initial conditions used were approximately 10-20% of the 50% AEP peak flow event.

5. River hydraulic model details

5.1. Introduction

The following sections report on the 20 individual river models in the FEM FRAM study area. Table 4-3~~Table 4-3Table 4-3~~ in Section 4.3 provides details of each of the individual river models. Figure 1 shows the extent of each model with additional information on the location of HPW and MPW watercourses, 2D model domains and ISIS reservoir units. A separate report section has been prepared for each of the river models in the study area. Where the river model incorporates two river systems, each of the river systems are described separately. This occurs for the following watercourses:

- Broadmeadow and Ward Rivers (Section 5.2) ;
- Ballyboghil and Corduff Rivers (Section 5.10); and
- Bride's Stream and Jone's Stream (Section 5.19).

The purpose of this chapter of the report is to provide details on the following elements of the hydraulic model build and model runs:

- Location of the watercourse and extent of model catchment. Each section has an overview map showing the extent of the watercourse (blue line) and the extent of the model reach (HPW – red line) and MPW (green line). The maps also reference the number of tributaries (marked T) and the upstream extent of the watercourse (marked with a bracketed number);
- Modelled structures (bridges, weirs, fluvial flood defences, etc);
- Roughness values for channel, floodplains and structures;
- Boundary conditions (inflows, downstream boundaries and interaction of flows between models);
- Model calibration (where calibration information is available);
- Critical storm duration;
- Sensitivity analysis (Manning's n, inflows and downstream boundaries);
- Defences (without defences scenarios); and
- Summary of results (current, MRFS and HEFS).

Additional details are also provided where the model development differs from the common methodology detailed in Section 4.4.

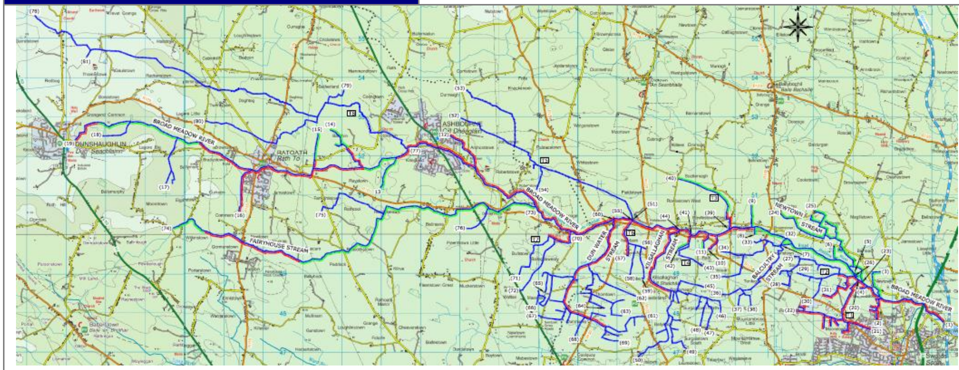
This chapter of the report needs to be read in conjunction with the flood maps for the relevant river models contained in Volume 2. These flood maps contain information on channel cross section labels and locations, water levels, flood outlines, confidence in levels and outlines, flows, depths, velocities and hazards for a number of AEP events (please refer to Chapter 7 for a further details on the flood maps). Reference to specific flood maps is also included where appropriate in this chapter.

For technical readers of the report, this chapter should also be read in conjunction with the digital deliverables contained in Volume 3 of the report. This volume contains additional information including the hydraulic model files, detailed model schematics and full breakdown of water levels and flows at every cross section for all AEP events and scenarios (current, MRFS, blockage scenarios, etc.).

5.2. Broadmeadow and Ward Rivers

5.2.1. Broadmeadow River

Introduction	
Water bodies	Broadmeadow River, Broadmeadow estuary
APSRs	Dunshaughlin area, Ratoath area, Ashbourne area and Swords area.



The Broadmeadow River has its source at Dunshaughlin. It flows in an easterly direction and discharges to the Broadmeadow Estuary. It is joined by the Ward River in Swords before discharging to the estuary. The map above provides an overview of the extent of the Broadmeadow River and its tributaries. Please refer to Figure 1 for more details on the extent of the Broadmeadow River hydraulic model and elements of the hydraulic model build (i.e. 2D model domains and Broadmeadow estuary). The catchment drains an area of 114.4km² and is broken down into 58 sub-catchments (refer to FEM FRAMS Hydrology Report, 2010). The main channel length is 26km and the 18 tributaries (excluding the Ward River) combined contribute an additional length of 37km. There are two gauging stations with complete records on the Broadmeadow River and these were used in the hydrological analysis: Stn 08007 at Ashbourne and Stn 08008 at Broadmeadow (Northern part of Swords). The tidal/fluvial dominance transition point is at Balheary Bridge (between the R132 and the M1) based on the 1% AEP fluvial and tidal event.

Model Build

The Broadmeadow River forms part of the larger Broadmeadow and Ward River model. The two rivers were modelled as one river model to ensure that any interaction in flood flows between the Ward and Broadmeadow rivers is accurately captured. The full model includes the Ward River, Broadmeadow River and the Broadmeadow estuary. This section provides details of the Broadmeadow River element of the model which extends from Dunshaughlin to Swords and includes the Broadmeadow estuary. Further information on the Ward River element of the model is detailed in Section 5.2.2. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the highly urbanised catchment of Broadmeadow River.

The Broadmeadow and Ward river model has been modelled together with the Broadmeadow estuary model, as the tidal boundary conditions were calculated at an offshore location near the mouth of the estuary. The western side of the Broadmeadow estuary is partly controlled by the viaduct constriction (Dublin to Belfast railway line). Therefore, a 50% AEP fluvial base flow from the other rivers discharging into the western side of the Broadmeadow estuary, i.e. the Lissenhall and Gaybrook Streams, was considered. The Lissenhall and Gaybrook Streams' 50% AEP fluvial baseflows were considered negligible compared to the

Broadmeadow and Ward River flows and were therefore not included in the model.

The reach 4Bau in Ashbourne was modelled as a separate model as this was added to the original scope of work following comments at the workshops to review the flood extent maps. In addition, it was necessary to adopt a finer 2D computational domain cell size to accurately model flooding along this tributary. Limitations with the software in modelling different domain cell sizes meant it was not possible to include the 4Bau reach as part of the Broadmeadow and Ward river model. This does not affect the accuracy of the model results.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	143	47 culverts/bridges on the main channel and 96 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR) and ORIFICE units.
Weir	13	11 weirs on the main channel and 2 on the tributaries. Each structure was represented by a SPILL unit due to its irregular shape.
Gauging Stations	2	Stn 08007 at Ashbourne and Stn 08008 at Broadmeadow (northern part of Swords)
Flood defences	2*	**Refers to location.* Informal effective defences included at two locations: tributary 4Ba between section 4Ba21573 and confluence with 4Bay142 and on tributary 4Bau.
Other	1	1 river expansion represented by BERNOULLI LOSS unit on the main channel.

At section Baqa791, a culvert has not been included in the model due to instability problems that could not be resolved. The impact of omitting this structure on the model results is negligible because of the low flows (even for high return periods), the significant storage and cross-section conveyance capacity of the channel and the rural location. The omission of this culvert will have no impact on the flood extents and hence flood risk management options at this location. However, should any planning application be considered at this location then this should be reviewed.

An assessment of culvert blockages was undertaken at 5 locations as follows: a stone bridge at cross section 4Ba19220 (Moulden Bridge in Ratoath); a stone bridge at Bridge Street in Ashbourne (cross section 4Ba15420); Robertstown Bridge at cross section 4Ba12867; Warblestown Bridge at cross section 4Ba5770; and a 65m culvert at cross section (4Bau2326) in Ashbourne. Further details on the culvert blockage assessment for the Broadmeadow River are in Section 9.2~~9.29.1~~.

The defences in Ratoath consist of a raised flood embankment on both the left and right banks of the river channel between cross sections 4Ba21573 and the confluence with 4Bay142 on the Broadmeadow River (the image opposite shows a sample of these defences). These defences were surveyed as part of the DAS and were likely to have been constructed as part of the Somerville housing development. No information was available as to whether these are formal flood defences and these are therefore considered as informal effective defences. The defences have been represented in the ID hydraulic model with the elevation of defences obtained from the channel and structure cross sectional survey data. A map showing the location of the defences in the model is available in Appendix C3.

In Ashbourne, informal effective defences have been added along the right and left bank of the 4Bau tributary. The defences consist of masonry walls which form part of houses or garden/park walls (refer to figure opposite). The walls are set back slightly from the concrete lined channel. Defences have been added along the right and left bank between cross sections 4Bau1635 and 4Bau1525. Further downstream defences have been added on the right bank only between 4Bau1525 and 4Bau1401. These defences are considered effective defences because they form part of a large structure (i.e. house) or are supported on the landward side by the build up of an embankment (e.g. at the park). The defences have been represented in the hydraulic model as an ISIS HX line with the elevation of defences obtained from the channel and structure cross sectional survey data. A map showing the location of the defences in the model is available in Appendix C3.

Further information on the impact of these defences on flood extents is reported on later in this section of the report in the 'without defences' scenario section. An analysis of flood risk and flood hazard due to sudden failure of these defences is reported on in Chapter 8.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
380	0	1 (on a tributary)	7 (6 with grid size of 5m and 1 with grid size of 2m). Refer to the following text for description of location of 2D domains.

Two-dimensional domains were established on the main river channel at the following locations: from cross section 4Ba21927 (Brownstown) to cross section 4Ba14197 (Archerstown); from cross section 4Ba13332 to chainage 12976 at Donaghmore Lodge; between cross sections 4Ba11068 and 4Ba10688 including the confluence of the 4Bas tributary in Greenogue; upstream of Rowlestown Bridge (in the vicinity of the confluence with 4Bap branch) and along 6km from Lispopple Bridge up to its confluence with the Broadmeadow estuary. 2D modelling was also undertaken for the Bat tributary between cross section 4Bat8219 and 4Bat7722 in Legagunnia and between cross sections 4Bat3031 and 4Bat2303 in the urban area of Baltrasna. Please refer to Figure 1 for further details on the location of these 2D model domains. Along the tributary reach 4Bau it was necessary to adopt a finer 2D computational domain cell size of 2m to accurately model flooding.

Representative Manning's *n* values	
River channel. Manning's n varies between 0.025 and 0.04	
0.025: At the estuary entrance: open channel, clean, straight, full stage, no rifts or deep pools (node 4Ba786)	0.04: Minor stream, clean, winding with some pools and sandbanks (node 4Ba2627)
	
Floodplain. Manning's n varies between 0.05 and 0.15	*Culverts. Colebrook-White friction 0.002m*
0.06: Floodplain, dispersed bushes, weeds and few trees (node 4Ba26148)	0.002: Concrete culvert, monolithic construction against rough forms (node 4Bad1313)
	

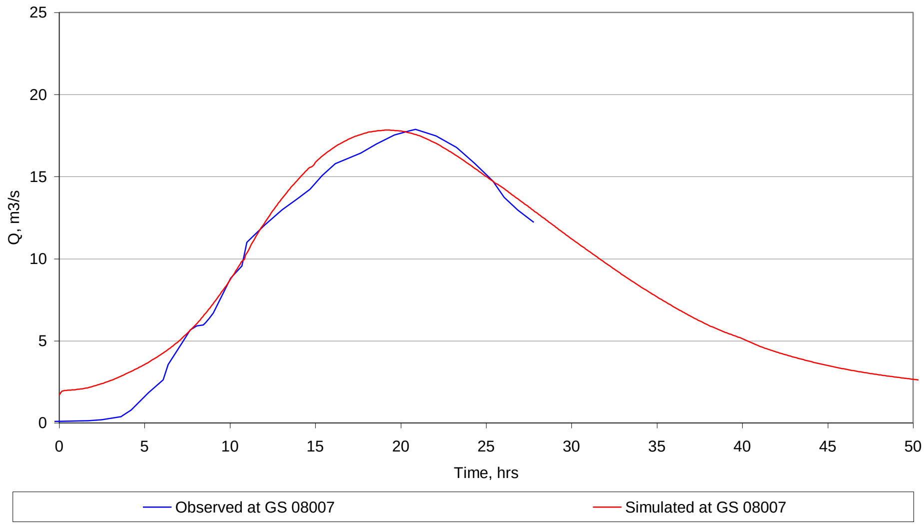
Boundary conditions

A tidal boundary at the mouth of the Broadmeadow estuary was used as the downstream boundary unit for the Broadmeadow and Ward river model. For tributary 4Bau, the downstream boundary conditions were extracted from the Broadmeadow River model. Further information on the model boundaries is available in the FEM FRAMS Hydrology Report, 2010.

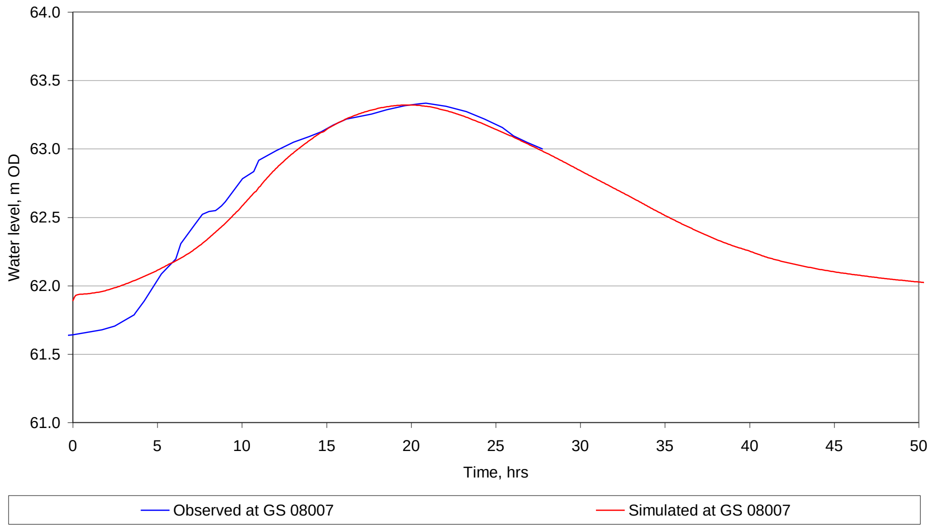
Model calibration

The August 1986 event was used to calibrate the model. As far as we are aware, there has been no flood defence/construction works carried out along the river since these dates. Therefore the design model was used to calibrate the event.

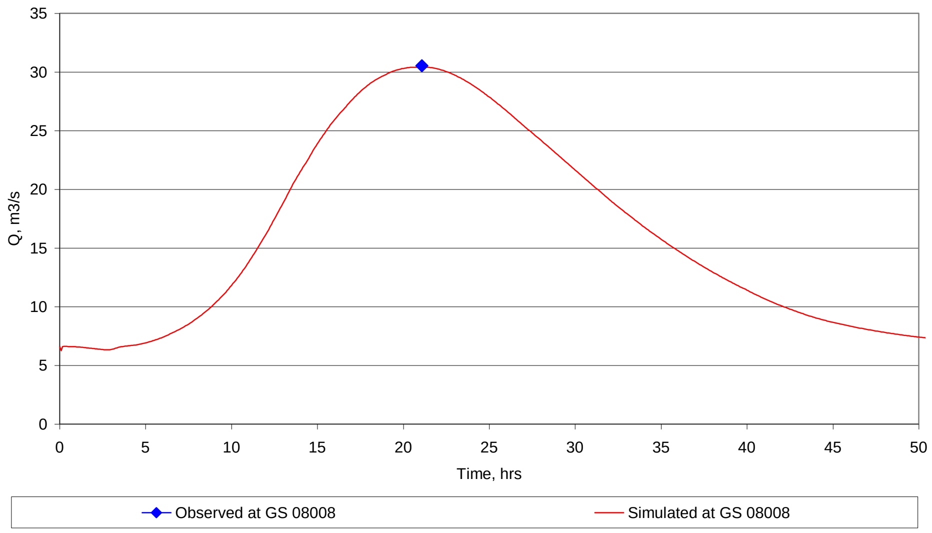
August 1986



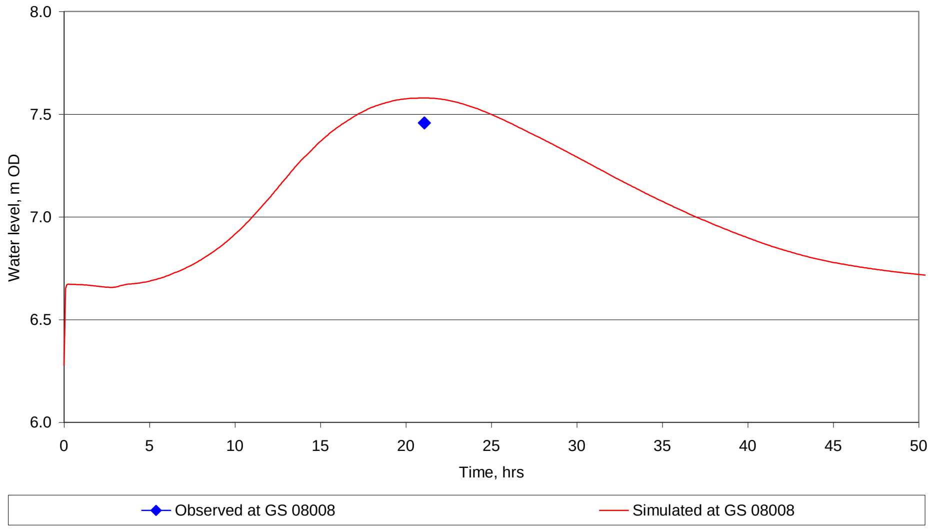
Comparison between observed flow hydrograph of the August 1986 (blue) and calibrated model flow hydrograph (red) at Ashbourne gauging station.



Comparison between the observed stage of August 1986 (blue) and calibrated model stage (red) at Ashbourne gauging station.



Comparison between the observed flow peak of August 1986 (blue) and calibrated model flow hydrograph (red) at Broadmeadow gauging station.



Comparison between the observed stage peak of August 1986 (blue) and calibrated model stage (red) at Broadmeadow gauging station.

The calibration results show that a good match was obtained between the observed flow and level data at the two gauging stations and the modelled flows and water levels. At gauging station G08007, the modelled peak flow is 0.04m^3 greater than the observed flow and the modelled water level is 0.01m greater than the observed flood level.

At gauging station G08008, the modelled peak flow is 0.11 m^3 greater than the observed flow and the modelled water level is within 0.12m lower than the observed flood level.

The results of the calibration demonstrate that the models match the observed peak flows and levels at the gauging station within acceptable limits (i.e. modelled levels within 0.2m of observed levels).

Critical Storm Duration
The hydraulic model was used to find the optimum/critical storm duration for the 1% AEP event. The critical storm duration calculated on the Broadmeadow river is 15 hours. The same critical storm duration was used for the other AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.10	-0.12	0.36	-0.38
Tributary Bac	0.03	-0.03	0.15	-0.19
Tributary Bad	0.03	-0.03	0.20	-0.32
Tributary Bae	0.02	-0.01	0.15	-0.19
Tributary Baf	0.08	-0.10	0.13	-0.17
Tributary Bah	0.16	0.03	0.24	-0.28
Tributary Baj	0.03	-0.03	0.16	-0.19
Tributary Bal	0.04	-0.04	0.08	-0.10
Tributary Bam	0.01	-0.01	0.15	-0.15
Tributary Ban	0.01	0.01	0.09	-0.10
Tributary Bap	0.04	-0.05	0.17	-0.38
Tributary Baq	0.02	-0.02	0.13	-0.13
Tributary Bar	0.05	-0.07	0.17	-0.20
Tributary Bas	0.05	-0.05	0.08	-0.10
Tributary Bat	0.09	-0.10	0.13	-0.20
Tributary Bau	0.04	-0.05	0.14	-0.19
Tributary Baw	0.07	-0.08	0.14	-0.14
Tributary Bax	0.05	-0.07	0.09	-0.12
Tributary Bay	0.04	-0.04	0.11	-0.14

Model Inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.14	-0.17	0.39	-0.49
Tributary Bac	0.03	-0.03	0.23	-0.22
Tributary Bad	0.03	-0.03	0.20	-0.29
Tributary Bae	0.01	-0.01	0.29	-0.27
Tributary Baf	0.15	-0.28	0.24	-0.35
Tributary Bah	0.13	-0.17	0.23	-0.27
Tributary Baj	0.04	-0.04	0.19	-0.22
Tributary Bal	0.06	-0.08	0.16	-0.35
Tributary Bam	0.02	-0.02	0.19	-0.21
Tributary Ban	0.04	0.00	0.15	-0.16
Tributary Bap	0.07	-0.08	0.27	-0.36
Tributary Baq	0.03	-0.03	0.10	-0.15
Tributary Bar	0.07	-0.10	0.17	-0.27
Tributary Bas	0.06	-0.07	0.15	-0.22
Tributary Bat	0.10	-0.11	0.24	-0.26
Tributary Bat	0.11	-0.15	0.52	-0.73
Tributary Baw	0.09	-0.07	0.16	-0.16
Tributary Bax	0.05	-0.08	0.09	-0.19
Tributary Bay	0.09	-0.09	0.17	-0.14

A reduction in hydraulic roughness of 20% results in an average reduction in peak water levels of 0.04m along all watercourses with a maximum reduction of approximately 0.38m. An increase in roughness results in localised increases in water levels, with an average increase along all watercourse of 0.05m and a maximum increase of approximately 0.36m. The largest increase and decrease in water levels both occur on the main Broadmeadow channel. The tributaries most affected by the roughness changes are 4Baq and 4Bar with water levels increased by a maximum of 0.17m for increases in Manning's roughness. Water levels along tributary 4Bap are most affected by a reduction in Manning's n with a maximum reduction of 0.38m for the lower roughness scenario.

Sensitivity analysis was also undertaken to determine the sensitivity of the model output to changes of the model input (i.e. inflows and downstream boundary). The results of the analysis suggest that a 20% increase in model inflows results in an average increase in levels of 0.07m and a maximum increase in levels of approximately 0.39m on the Broadmeadow

River and of 0.52m on the 4Bat tributary. With a 20% reduction in flows, the average decrease in levels is 0.08m with a maximum decrease of 0.49m and 0.73m levels observed on the main Broadmeadow channel and along tributary 4Bat branch. A tidal sensitivity (+/- 0.25m) was carried out for the 0.5% AEP tidal event. The tidal/fluvial dominance transition point is at model cross section 4Ba287. As the model doesn't have a tidal defence, the differences in the water levels are approximately +/-0.25m in the tidal reaches of the Broadmeadow River.

The results indicate that the model is sensitive to changes in both Manning's n and model inflows with notable increases and decreases in water levels at a number of cross sections along the river channels. The largest changes in water levels occur at structures with low conveyance capacity where increases and decreases in both conveyance and flow results in large increases and decreases in water levels. As the model doesn't have a tidal defence, sensitivity to tidal level along the tidal reaches of the Broadmeadow River is as expected (i.e. +/- 0.25m).

'Without defences' scenario

The defences detailed earlier in this section of the report were removed in the 'without defences' model. A review of the flood extent maps for Ratoath (BRO/HPW/EXT/CURS/002) in Volume 2 of the report) shows that the defences reduce the extent of flooding to an area of land to the east of the housing estate for the 1% AEP event. A review of the flood extent maps for Ashbourne (BRO/HPW/EXT/CURS/005) shows that there are no areas benefiting from the defences. These defences in Ashbourne were not surveyed as part of the DAS and as the maps indicate that they serve no flood defence function it is not recommended to included these as part of any future DAS.

Model Results Summary

The following section provides a brief overview of the flood hazard along the Broadmeadow River including the Broadmeadow estuary. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

Based on the results from the Broadmeadow river model, flooding begins for a 50% AEP fluvial event. Out of bank flooding occurs 570m upstream of the hydrometric gauging station 08008 and at the confluence of main Broadmeadow River channel and tributary Bax (refer to maps BRO/HPW/EXT/CURS/002 and BRO/HPW/EXT/CURS/009). The 20% AEP fluvial event results in inundation of the floodplain at Killegland (west of Ashbourne near GS 08007). For the 10% and 4% AEP fluvial events, flooding starts at Balheary Demesne (cross section 4Ba940) and at Newtown Bridge (cross section 4Ba1276). Please refer to maps BRO/HPW/EXT/CURS/004, BRO/HPW/EXT/CURS/005 and BRO/MPW/EXT/CURS/003.

For lower frequency events, there are overflows upstream of Rowlestown Bridge (4Ba8020 section), the bridge at Jamestown park (4Ba19886 section), Moulden Bridge (4Ba19221), the hydrometric station 08007 (4Ba15996) and Milltown bridge at 4Ba14272. Refer to maps BRO/HPW/EXT/CURS/002, BRO/HPW/EXT/CURS/004, BRO/HPW/EXT/CURS/005 and BRO/HPW/EXT/CURS/008.

The results from the Broadmeadow and Ward River model have been used to map the flood hazard around the estuary. Based on these flood maps, the most significant flood risk is in Malahide area APSR where roads and properties are at risk of flooding. Elsewhere around the estuary, the flooding mainly affects agricultural land.

For the MRFS and HEFS the increase in flows does not result in a significant increase in flood extents and flood risk. The average water level increase between the current scenario and the MRFS for the 1% AEP fluvial event is 0.10m and the average water level increase between the current scenario and the HEFS for 1% AEP fluvial event is 0.16m. The maximum difference is 0.78m and 1.20 m at 4Bay540 section.

Watercourse	*Average Water Level Difference(m)*		*Maximum Water Level Difference(m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.16	0.35	0.47	1.14
Tributary Bac	0.04	0.09	0.24	0.30
Tributary Bad	0.03	0.07	0.23	0.38
Tributary Bae	0.02	0.03	0.38	0.63
Tributary Baf	0.18	0.32	0.29	0.47
Tributary Bah	0.28	0.37	0.42	0.48
Tributary Baj	0.05	0.08	0.22	0.38
Tributary Bal	0.06	0.09	0.18	0.31
Tributary Bam	0.03	-0.01	0.22	0.00
Tributary Ban	0.06	0.09	0.21	0.31
Tributary Bap	0.09	0.15	0.31	0.56
Tributary Baq	0.03	0.04	0.11	0.21
Tributary Bar	0.07	0.10	0.19	0.31
Tributary Bas	0.07	0.12	0.17	0.34
Tributary Bat	0.14	0.19	0.34	0.51
Tributary Bat	0.12	0.17	0.60	0.74
Tributary Baw	0.09	0.13	0.17	0.31
Tributary Bax	0.07	0.11	0.11	0.18
Tributary Bay	0.30	0.52	0.78	1.20

5.2.2. Ward River

Introduction

Water bodies	Ward River, St. Margaret's Stream, Shallon Stream
APSRs	Swords area

The Ward River has its source near Mabestown; it flows in an easterly direction and joins the Broadmeadow River in Swords area APSR to the west of the Broadmeadow Estuary. The map above provides an overview of the extent of the Ward River and its tributaries. Please refer to Figure 1 for more details on the extent of the Ward River model and elements of the hydraulic model build (e.g. 2D model domains). The catchment drains an area of 58.13km² and is broken down into 21 sub-catchments (refer to FEM FRAMS Hydrology Report, 2010). The main channel length is 12.1km and there are six tributaries which have a combined additional length of 9km. The Balheary Gauging Station is located at the weir at cross section 4Wa324. The river upstream of its confluence with the Broadmeadow River is entirely fluvially dominated based on the 1% AEP fluvial and tidal event.

Model Build

The Ward River forms part of the larger Broadmeadow and Ward river model. The two rivers were modelled as one river model to ensure that any interaction in flood flows between the Ward and Broadmeadow rivers is accurately captured. The full model includes the Ward River, Broadmeadow River and the Broadmeadow estuary. This section provides details of the Ward River element of the model (to its confluence with the Broadmeadow River). Further information on the Broadmeadow River element of the model (including the Broadmeadow estuary) is detailed in Section 5.2.1. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised catchment of the Ward River.

Summary of structures in the model

Type	*Number*	*Summary*
Culverts/Bridges	62	24 culverts/bridges on the main Ward River channel and 38 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR) and ORIFICE units.
Weirs	16	7 weirs on the main channel and 9 on the tributaries. Each structure was represented by a SPILL unit due to its irregular shape.
Gauging stations	1	Gauge 08009 at cross section 4Wa324 (modelled as one of the 16 weir units listed above) at Balheary.

Type	*Number*	*Summary*
Flood defences	0	No flood defences have been modelled along the Ward River.
Other	1	Pond outlet represented by a SPILL unit.

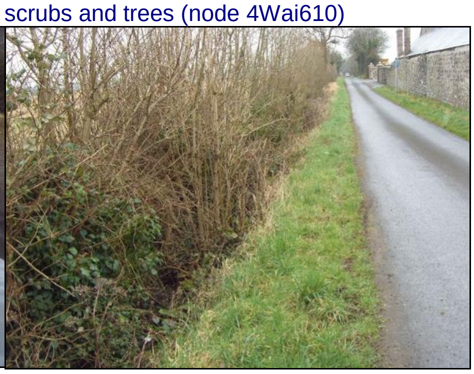
An assessment of culvert blockages was undertaken for the following culverts; a 45m concrete culvert on the Ward River at cross section 4Wa102 and a bridge at cross section 4Wa953. Further details on the culvert blockage assessment for the Ward River are in Section 9.2.4~~9.2.4~~~~9.1.4~~.

Gauge 08009 (Balheary) was reviewed, however the Q_{med} estimated from the revised rating was not consistent (grossly underestimated) with other Q_{med} values in the study area. Further details are available in Appendix C of the Final Hydrology Report (Halcrow Barry, 2010).

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
410	7 There are 2 more “on line” reservoirs.	0	3 (1 at confluence of the Ward and Broadmeadow Rivers, 1 at confluence of Ward River and Wab tributary and 1 at Coolatrath West). Grid size 5m.

A 2D model was developed on the right bank of the Wab tributary downstream from cross section 4Wab561 to the junction with the Ward River. On the Ward River, a 2D model domain was added from cross section 4Wa1829 to the confluence with the Broadmeadow River. On the Wah tributary, a 2D domain was added between the cross section 4Wah3224 in Coolatrath West up to the chainage 4Wah1783. Please refer to Figure 1 for further details on the location of these 2D model domains.

Representative Manning’s *n* values	
Channel (Manning’s n values) between 0.025 and 0.060).	
0.025: Excavated, dragged or man-made earth channel, winding and shallow, without vegetation (node 4Wah847)	0.06: Natural channel, clay, lateral slopes and bed with irregularities, almost all section with scrubs and trees (node 4Wai610)



Floodplain (varies between 0.030 and 0.085)	*Culverts (0.002m Colebrook-White friction)*
0.06: Floodplain, dispersed bushes, weeds and few trees (node 4Wa6295)	Concrete culvert, monolithic construction against rough forms (node 4Wab292)
	

Boundary conditions

The Ward and Broadmeadow Rivers were modelled as one river model to ensure that any interaction in flood flows between the Ward and Broadmeadow Rivers is accurately captured. A tidal boundary at the mouth of the Broadmeadow estuary was used as the downstream boundary unit for the Broadmeadow and Ward River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate the Ward River.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP event on the Ward River is 17 hours. This critical storm duration applies to the whole of the river model (i.e. Ward and Broadmeadow). The same critical storm duration was used for the other AEP events and scenarios.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n and model inflows.

Manning's *n*

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.08	-0.11	0.38	-0.43
Tributary Wab	0.03	-0.03	0.08	-0.18
Tributary Wad	0.02	-0.02	0.15	-0.21
Tributary Wag	0.03	-0.04	0.10	-0.14
Tributary Wah	0.09	-0.12	0.20	-0.33
Tributary Wai	0.01	0.00	0.06	-0.04
Tributary Waj	0.03	-0.04	0.11	-0.09

Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.13	-0.18	0.34	-0.48
Tributary Wab	0.07	-0.09	0.34	-0.52
Tributary Wad	0.03	-0.14	0.05	-0.04
Tributary Wag	0.09	-0.11	0.33	-0.27
Tributary Wah	0.16	-0.18	0.26	-0.35
Tributary Wai	0.01	-0.01	0.12	-0.22
Tributary Waj	0.04	-0.05	0.12	-0.18

The effect of increasing the roughness of the floodplain and channel, as expected, results in an increase in the water levels. The main river channel and Wah tributary show the largest increase in water levels of 0.38m and 0.20m respectively. The average increase in water levels along all of the watercourses is 0.04m. A reduction in the hydraulic roughness of 20% results in an average reduction in peak water levels of 0.05m along all of the watercourses, with a maximum reduction of 0.43m on the Ward River and 0.33m on the Wah tributary.

An increase of 20% in inflows results in an average increase in the peak water levels of 0.08m along all watercourses, up to a maximum of approximately 0.34m on the Ward River and Wab tributary. The reduction in inflows also results in an average decrease of 0.11m in water levels along all watercourses with localised decreases of up to 0.48m in the main channel and 0.52m in Wab branch.

The results indicate that the model is sensitive to changes in both Manning's n and model inflows with notable increases and decreases in water levels at a number of cross sections along the river channels. The largest change in water levels occurs at structures with low conveyance capacity where increases and decreases in both conveyance and flow results in large increases and decreases in water levels.

The discussion on sensitivity to changes in the downstream boundary is in Section 5.2.1 (Broadmeadow River).

'Without defences' scenario
No defences are present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Ward River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

For the current scenario, flooding along the Ward River model starts at the 20% AEP fluvial event near the confluence of the Broadmeadow and Ward Rivers in the Swords area APSR. This flooding is as a result of the low capacity of the inlet of the culvert at cross section 4Wa102. For the 4% AEP fluvial event, Bridge St. road is partially overtopped and some properties along Main Street are affected on the right bank floodplain upstream from the bridge (shopping centre and car park). Refer to map WAR/HPW/EXT/CURS/003 in Volume 2. To the north of Swords town centre, the bridge in Balheary Road is overtopped for a 2% AEP fluvial event (refer to map WAR/HPW/EXT/CURS/003). The flood maps indicate that the most significant flooding in Swords is in the area of Balheary Road where flood flows from both the Ward River and Broadmeadow River interact in the vicinity of the confluence between the two rivers.

Outside of Swords area APSR, the most extensive flooding occurs along the 4Wah tributary with flooding starting at the 2% AEP fluvial event at the following locations: Coolquoy Common, the R130 road bridge and at R135 culvert on the left bank (refer to map WAR/HPW/EXT/CURS/001).

For the MRFS and HEFS the most significant increase in flooding is in Swords town centre where increases in flows and in particular, increases in tide levels, increases the flood risk. The average increase in water levels between the current scenario and the MRFS for 1% AEP fluvial event is 0.08m and the average water level increase between the current scenario and the HEFS for 1% AEP fluvial event is 0.12m. The maximum difference is 0.62m and 0.73m, respectively for the MRFS and HEFS, with the largest increases occurring on the Ward River near the confluence with the Broadmeadow River (refer to map WAR/HPW/EXT/MRFS/003).

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.15	0.24	0.62	0.73
Tributary Wab	0.08	0.12	0.37	0.64
Tributary Wad	0.03	0.05	0.20	0.33
Tributary Wag	0.10	0.16	0.34	0.51
Tributary Wah	0.14	0.19	0.26	0.36
Tributary Wai	0.01	0.01	0.12	0.15
Tributary Waj	0.04	0.05	0.12	0.15

5.3. River Nanny

Introduction	
Water bodies	River Nanny and River Hurley
APSRs	Kentstown area, Duleek area, Julianstown area and Laytown, Bettystown and Coastal area.

The River Nanny has its source at Kentstown area APSR; it flows in an easterly direction until it meets the Irish Sea. The map above provides an overview of the extent of the Nanny River and its tributaries. Please refer to Figure 1 for more details on the extent of the Nanny River hydraulic model and elements of the hydraulic model build (e.g. 2D model domains). The catchment drains an area of 235km² and is broken down into 36 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). Its main tributary, the Hurley River, has a corresponding catchment area of 93km^2 but was not modelled. The main channel length is 27km and the 8 tributaries combined contribute an additional length of 21km. There is one gauging station on this river: Stn 08011 at Duleek. The tidal/fluvial dominance transition point is downstream of Julianstown based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the catchment of the Nanny River.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	92	28 culverts/bridges on the main channel and 64 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR) and ORIFICE units.
Weir	10	3 weirs on the main channel and 7 on the tributaries. Each structure was represented by a SPILL unit due to its irregular shape.

Type	*Number*	*Summary*
Gauging station	1	Stn 08011 at Duleek.
Flood defences	1*	*Refers to location. Embankments, walls and a pumping station along the Paramadden River and the River Nanny at Duleek area APSR.
Other	5	1 river constriction (old bridge abutment) represented by weir unit on the main channel. 2 side weirs + 2 control sluices from bypass/parallel channel entrance represented by weir and sluice units on the main channel.

An assessment of culvert blockages was undertaken at one location: a 6.5m long stone bridge with three arches where the Paramadden tributary (Nag channel) flows under Bridge Street, in Duleek. Further details on the culvert blockage assessment for the Nanny are in Section 9.2.18~~9.2.18~~~~9.1.18~~.

Along the Paramadden River, defences are located along both the right and left banks and consist of earth embankments and concrete walls. These defences run from the confluence of the Paramadden River with the Nanny River to upstream of the Main Street Bridge (refer to sample image opposite). Along the Nanny River earth embankments and walls have been constructed in the left bank floodplain alongside Abbeylands and Mill Race housing developments. The pumping station is a surface water pumping station and is not represented in the model.

The defences are considered as formal flood defences and have been represented in the model as ISIS Z lines and ISIS HX lines with elevations obtained from both the LiDAR data and channel and structure cross sectional survey data. In addition, the LiDAR survey captured the flood defence embankments around the Abbeylands and Mill Race housing developments and these were represented in the 2D model domain using the LiDAR data without the need for ISIS Z lines. A map showing the location of the defences in the model is available in Appendix C3.

Further information on the impact of these defences on flood extents is reported on later in this section of the report in the 'without defences' scenario section. An analysis of flood risk and flood hazard due to sudden failure of these defences is reported on in Chapter 8.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
375	0	2 (on a main channel)	In Balrath and in Duleek (grid size 5m).

Flood defence embankments in the floodplain of the Nanny River at Duleek were captured by the LiDAR data. Therefore no adjustments were required to the floodplain model domain to represent these embankments. Further information on these defences is available in the previous section.

Representative Manning's *n* values	
Channel. Manning's n varies between 0.024 and 0.05 (after calibration)	
0.024: Excavated, dragged or man-made channel, shallow and without vegetation (node 20Nag340)	0.05: Minor stream, clean, winding, some pools and shoals with some weeds and stones (node 20Naa397)
	
Floodplain. Manning's n varies between 0.03 and 0.099 (after calibration)	*Culverts. Colebrook-White friction varies between 0.0002m and 0.002m*
0.066: Floodplain, dispersed bushes, weeds and few trees (node 20Na23740)	0.0002 m: PVC culvert (node Nah7704)
	

Boundary conditions

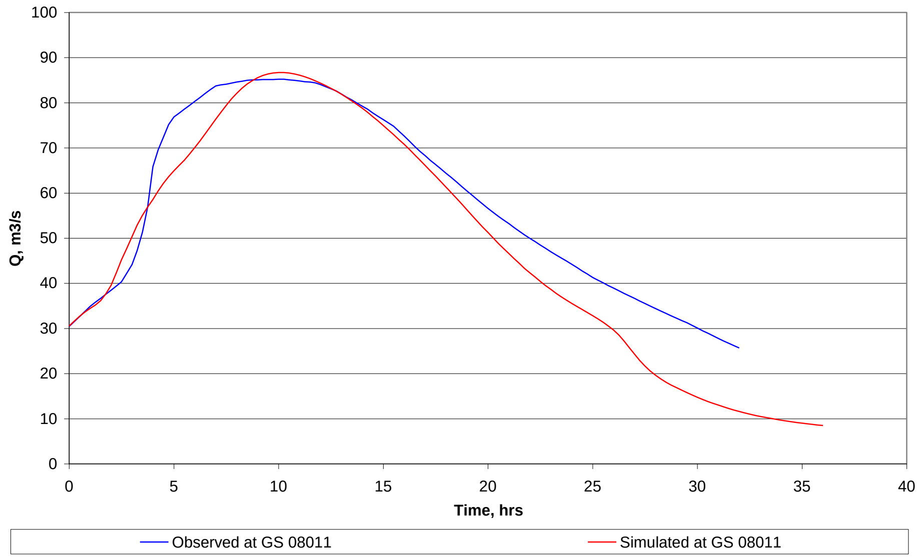
The River Hurley (Nanny's main tributary) was not surveyed but had to be considered in the model as a hydrological input (its catchment represents 40% of the whole Nanny River catchment). It was therefore assumed that the most practical way of representing its inflow for the design events would be to directly route it through the 2D domain. This would avoid instabilities when linking this inflow directly in the 1D model as a lateral inflow. As the River Hurley catchment represents 40% of the whole Nanny River catchment it is recommended that some of this river is surveyed and added to the Nanny River model to improve the schematic of this model.

A tidal boundary at the mouth of the Nanny River was used as the downstream boundary unit for the river model. Further information on the remaining model boundaries is available in the FEM FRAMS Hydrology Report, 2010.

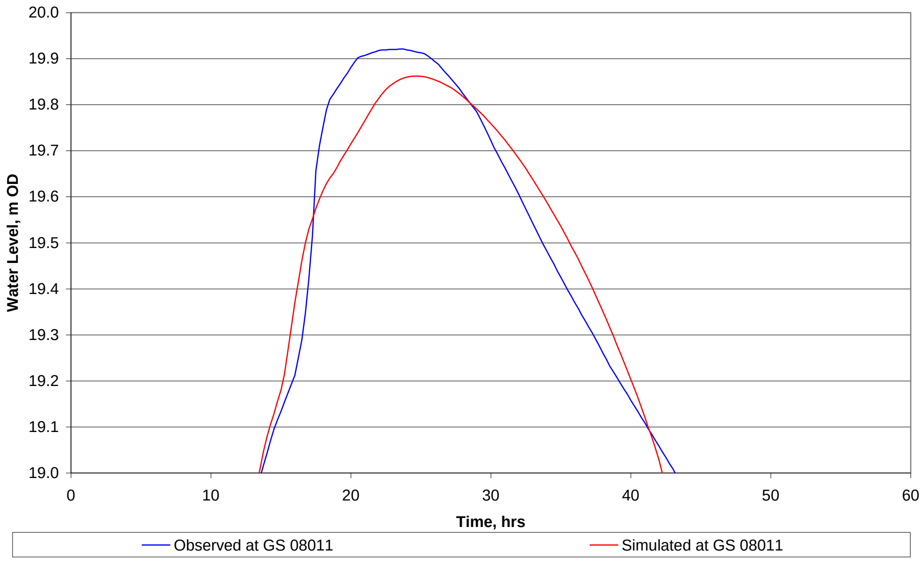
Model calibration

For calibration, three historic flood events were available; August 1986, June 1993 and November 2000. Earth embankments at Duleek's Millrace Estate, as well as the R152 bypass structure were built between the 1993 and 2000 flood events. Therefore, the 'without defences' model was used to calibrate the 1986 and 1993 events whilst the 2000 event was calibrated using the design model.

August 1986

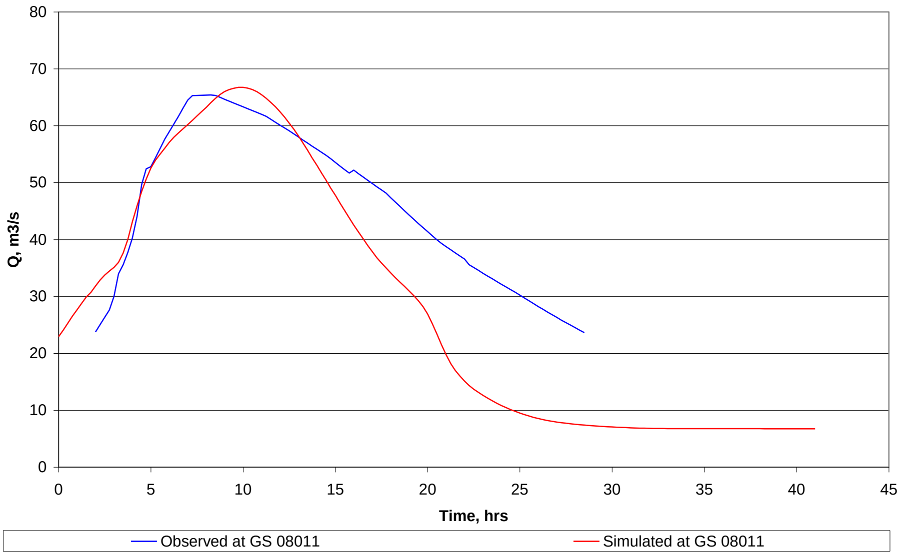


Comparison between the August 1986 flow hydrograph (blue) and calibrated model flow hydrograph (red) at Duleek gauging station.

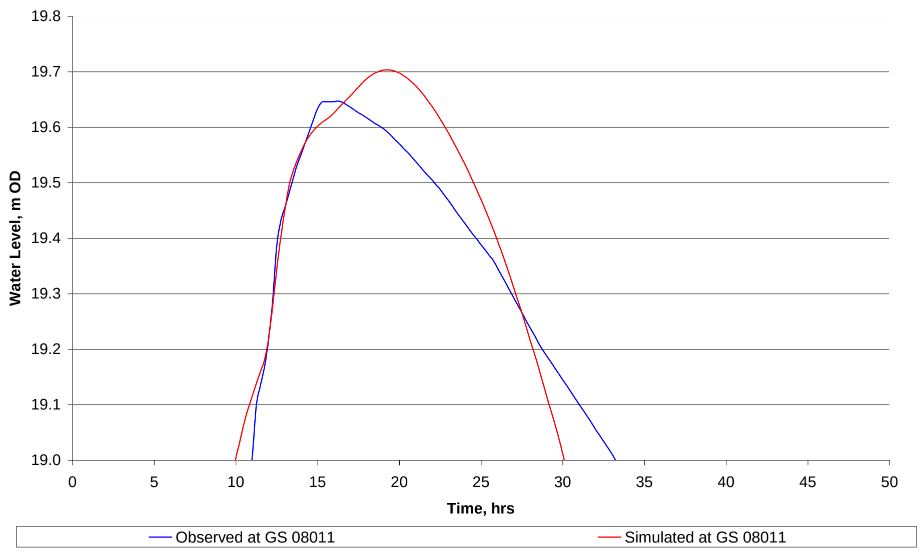


Comparison between the August 1986 stage (blue) and calibrated model stage (red) at Duleek gauging station.

June 1993

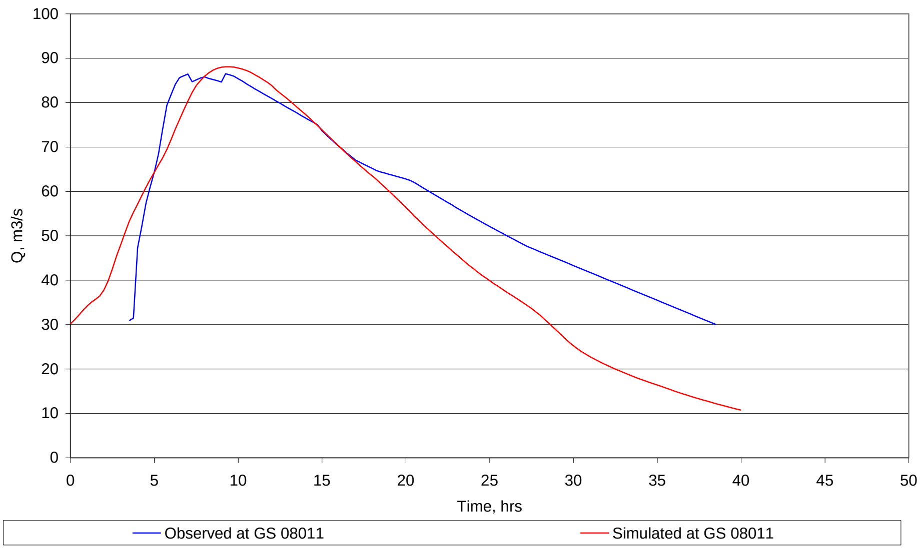


Comparison between the June 1993 flow hydrograph (blue) and calibrated model flow hydrograph (red) at Duleek gauging station.

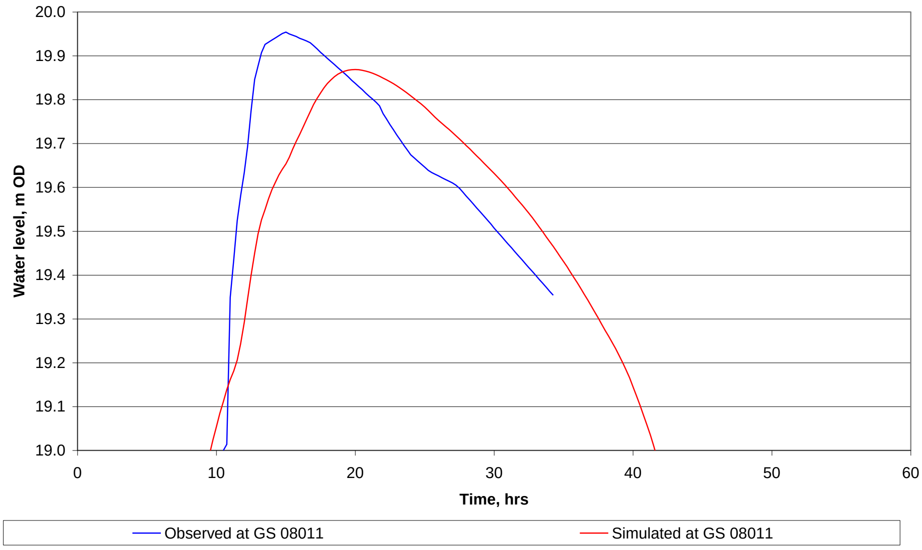


Comparison between the June 1993 stage (blue) and calibrated model stage (red) at Duleek gauging station.

November 2000



Comparison between the November 2000 flow hydrograph (blue) and calibrated model flow hydrograph (red) at Duleek gauging station.



Comparison between the November 2000 stage (blue) and calibrated model stage (red) at Duleek gauging station.

The calibration results show that a good match was obtained between the observed flow and level data at the gauges and the modelled flows and water levels. For the 1986 flood event, the modelled peak flow is 1.5 m^3 greater than the observed flow with the modelled water level 0.06m less than the observed flood level.

For the 1993 flood event, the modelled peak flow is 1.4m^3 greater than the observed flow and the modelled water level is 0.06m greater than the observed flood level.

For the 2000 flood event, the modelled peak flow is within 1.7m^3 greater than the observed flow and the modelled water level is 0.09m less than the observed flood level.

The results of the calibration demonstrate that the models match the observed peak flows and levels at the gauging station within acceptable limits (i.e. modelled levels are within 0.2m of observed levels).

Critical Storm Duration
The critical storm duration calculated for the 1% AEP for the River Nanny is 15 hours. The same critical storm duration was used for all AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.11	-0.13	0.34	-0.46
Tributary Naa	0.04	-0.04	0.06	-0.07
Tributary Nac	0.16	-0.20	0.16	-0.20
Tributary Nad	0.05	-0.06	0.18	-0.22
Tributary Nae	0.05	-0.05	0.09	-0.09
Tributary Naf	0.02	-0.01	0.04	-0.04
Tributary Nag	0.05	-0.06	0.17	-0.21
Tributary Naga	0.07	-0.06	0.47	-0.14
Tributary Nah	0.03	-0.08	0.32	-1.00
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.15	-0.18	0.46	-0.56
Tributary Naa	0.06	-0.06	0.09	-0.09
Tributary Nac	0.18	-0.21	0.18	-0.21
Tributary Nad	0.10	-0.17	0.42	-0.50
Tributary Nae	0.07	-0.07	0.13	-0.12
Tributary Naf	0.03	-0.03	0.04	-0.05
Tributary Nag	0.12	-0.24	0.30	-0.63
Tributary Naga	0.10	-0.09	0.74	-0.19
Tributary Nah	0.06	-0.13	0.35	-1.21

In terms of sensitivity to roughness, the average increase in water levels (for Manning's n+20%) along all of the watercourses is 0.06m with the highest difference along the Naga tributary. For Manning's n-20%, the average decrease in water levels along all of the watercourses is 0.07m with the largest decreases along the Nah tributary. For the increased

Manning's n values, the highest difference is located upstream of the structures located at sections 20Na7997In and 20Na8074In. These cross sections are located just downstream from Beaumont's channel loop and upstream of the M1 motorway. Along the tributaries, the highest differences are located upstream of the 20Naga4238 structure (south of Garballagh) and of the 20NahCU4327 long culvert (south of Rathdrinagh).

In terms of sensitivity to changes in flow, the average increase in water levels along all watercourses (inflows +20%) is 0.1m with the highest differences located on the Nag, Naga and Nah Tributaries. On the main river, the highest differences are located upstream of the R108 Bridge (20Na6373) downstream of the M1. With the inflows decreased, the average difference in water levels along all watercourses is 0.13m with the largest decrease along Nah tributary.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition starts to be significant from section 20Na3761 (just downstream of the R132 Bridge at Julianstown) on the main channel. As the model doesn't have tidal defences the differences in the water levels are approximately +/-0.25m in most of the affected river reach.

The results indicate that the model is sensitive to changes in both Manning's n and model inflows with notable increases and decreases in water levels at a number of cross sections along the river channels. The most significant changes in water levels occur at structures with low conveyance capacity where increases and decreases in both conveyance and flow results in large increases and decreases in water levels. As the model doesn't have a tidal defence, sensitivity to tidal level is along the tidal reaches of the Nanny River as expected.

'Without defences' scenario

A review of the flood extent maps for Ratoath (NAN/HPW/EXT/CURS/003 in Volume 2 of report) shows that there are significant areas benefiting from the defences. The defences provide protection to the majority of properties up to the 1% AEP event with a small area at risk for the 2% AEP event. A limited extent of the defences alongside the Nanny River was surveyed as part of the DAS and it is recommended that the remainder of these defences are surveyed at a future stage.

Model Results Summary

The following section provides a brief overview of the flood hazard along the Nanny River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

At Kentstown area APSR, the R153 road bridge is overtopped for a 2% AEP fluvial design event or greater. The N2 road, which crosses the floodplain of the Nanny River, floods during the 0.1% AEP fluvial design event. Refer to map NAN/HPW/EXT/CURS/001.

In Duleek area APSR, the existing defence embankments and walls offer protection to the majority of properties up to 1% AEP event (NAN/HPW/EXT/CURS/001). Flooding occurs in the western part of the Millrace Estate for the 2% AEP event and at localised areas along the Paramadden tributary as a result of flood waters overtopping the bank upstream of the defences near Main Street. However, there is significant flooding for the 0.5% AEP event or greater, principally at the Millrace Estate, Colgan Street and Abbeylands, due to overtopping of the flood defences. The R152 road between Duleek and Drogheda overtops for a 0.1% AEP fluvial design event on the left bank and for a 4% AEP fluvial design event or greater on the right bank.

At Beaumont, upstream of Beaumont Bridge, a number of properties are inundated when flood waters cross the R150 road on the left bank for a 1% AEP fluvial design event or greater. Refer to NAN/MPW/EXT/CURS/002.

At Laytown area APSR, inundation of land results from combined fluvial and tidal flooding. The flooding is mainly confined to a small area of agricultural land with a small number of properties at risk at the mouth of the Nanny River (NAN/HPW/EXT/CURS/004).

For the MRFS, the average water level increase between the current scenario and MRFS for 1% AEP fluvial event is 0.14m and for the HEFS it is 0.24m. The maximum difference for the MRFS is 0.51m occurring upstream of the Silicy Road culvert on the Naga Tributary (20Naga5282). On the main River Nanny, the maximum difference is 0.48m occurring upstream from the R108 road bridge. This large difference is mainly due to the constriction at the bridge creating a large head loss and backwater effect up to 400m upstream. The bed slope is also very flat locally. These increases in the MRFS water levels result in a marginal increase in flood extents along the course of the Nanny River with larger increases in extents at localised areas along the river reach. At the downstream extent of the model, the increase in mean sea levels combined with the increase in river flows contributes to an increase in flood extents at Laytown area APSR.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.17	0.30	0.48	0.97
Tributary Naa	0.17	0.46	0.34	0.93
Tributary Nac	0.28	0.68	0.29	0.68
Tributary Nad	0.11	0.17	0.44	0.70
Tributary Nae	0.06	0.09	0.12	0.19
Tributary Naf	0.04	0.07	0.06	0.10
Tributary Nag	0.11	0.16	0.27	0.45
Tributary Naga	0.09	0.14	0.51	0.68
Tributary Nah	0.08	0.12	0.21	0.29

5.4. Lissenhall Stream

Introduction	
Water bodies	Lissenhall Stream
APSRs	None

The Lissenhall Stream has its source south of Belinstown near Lissenhall Little. It flows in a south-easterly direction until it meets the Broadmeadow Estuary. The map above provides an overview of the extent of the Lissenhall Stream. Please refer to Figure 1 for more details on the extent of the Lissenhall Stream hydraulic model and elements of the hydraulic model build. The catchment drains an area of 3.6km² and is broken down into seven sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 3.4km and the two tributaries combined contribute an additional length of 1.1km. There are no gauging stations on this river. Although the Lissenhall Stream is flapped at the outfall to the Broadmeadow Estuary, the tidal/fluvial dominance transition point is located just upstream of the M1 motorway culvert based on the 1% AEP fluvial and tidal event. This is as a result of high tides bypassing the flapped outfall on the left bank floodplain. Please refer to model results summary for further details.

Model Build

A 1D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the catchment of the Lissenhall Stream. No 2D modelling was undertaken as the river passes through rural areas and its hydraulic behaviour can be accurately modelled using 1D modelling techniques.

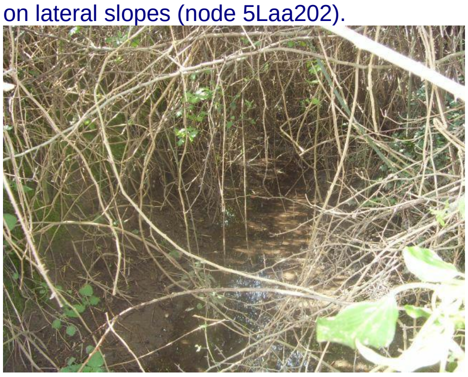
The Lissenhall Stream has been modelled together with the Broadmeadow Estuary model, as the tidal boundary conditions were calculated at an offshore location near the mouth of the estuary. The western side of the Broadmeadow estuary is partly controlled by the viaduct constriction (Dublin to Belfast railway line). Therefore, a 50% AEP fluvial base flow from the other rivers discharging into the western side of the Broadmeadow estuary, i.e. the Broadmeadow River and Gaybrook Streams, were considered. However, the Gaybrook Stream's 50% AEP fluvial baseflow was considered as negligible compared to the Broadmeadow and Ward River flows and was therefore not included in the model.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	14	9 culverts/bridges on the main channel and 5 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR), CULVERT, ORIFICE and VERTICAL SLUICE units.
Weir	0	
Gauging station	0	
Flood defences	0	
Other	1	Flapped outfall modelled as an ORIFICE UNIT

The flapped outfall located at the downstream extent of the Lissenhall Stream provides limited benefits in preventing the propagation of high tides and storm surges west of the outfall and is not considered a flood defence. This is because high tides and storm surges can bypass this structure downstream of the outfall along the Broadmeadow estuary coastline (for the 4% AEP tidal event). These tidal flows impact on the water levels in the Lissenhall Stream upstream of the flapped outfall.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
89	11	0	0

At the downstream end of the model, a stone bridge significantly constricts the discharge of the Lissenhall Stream into the Broadmeadow estuary. Therefore, for high AEP fluvial events, the main channel is bypassed on the left bank floodplain, between Seapoint and Ballymadrough and across the Seapoint coast road on the right floodplain. Floodplain reservoirs and spillways were added to represent this overtopping flow route with elevations obtained from the LiDAR DTM data.

Representative Manning's *n* values	
Channel. Manning's n varies between 0.03 and 0.05	
0.03: Open channel, clean, straight, full stage, no rifts or deep pools (node 5La221).	0.05: Natural channel, clay, lateral slopes and bed with irregularities, scrub and bushes on lateral slopes (node 5Laa202).
	

Floodplain. Manning's n of 0.06	*Culverts. Colebrook-White friction of 0.006m*
0.06: dispersed bushes, weeds and few trees (node 5La448). 	0.006: Concrete culvert, monolithic construction against rough forms with cracks (node 5La1506). 

Boundary conditions

A tidal boundary at the mouth of the Broadmeadow estuary was used as the downstream boundary unit for the Lissenhall River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP event for the Lissenhall Stream is 11 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.01	-0.02	0.06	-0.10
Tributary Laa	0.01	-0.01	0.04	-0.05
Tributary Lab	0.02	-0.03	0.05	-0.05

Model inflows

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow -20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.02	-0.03	0.08	-0.12
Tributary Laa	0.02	-0.02	0.03	-0.06
Tributary Lab	0.06	-0.05	0.41	-0.10

In terms of roughness sensitivity, there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is 0.1m and 0.2m respectively. In terms of flow sensitivity, the differences in water levels are more significant than the roughness sensitivity, with an average increase of 0.03m (flow +20%) and decrease of 0.03m (flow -20%). The largest difference is located on the Lab tributary (+20%) just upstream from the M1 long culvert (5Lab193C).

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition starts to be significant from section 5La1727, just upstream of the R132 Bridge on the main channel. The differences on tidal sensitivity in the water levels are approximately +/-0.30m in most of the tidal reaches of the watercourse.

The results indicate that the model is relatively insensitive to changes in both Manning's n and model inflows. Notable changes in water levels as a result of an increase and decrease in flows are restricted to one cross section, 5Lab193C, where the conveyance capacity of the culvert results in large increases and decreases in water levels. As tides can bypass the downstream tidal flap valve, the river channel is sensitive to changes in downstream tide levels by approximately +/- 0.25m.

'Without defences' scenario
A 'without defences' model was not run for the Lissenhall Stream as the flapped tidal outfall does not actually operate as a defence due to bypassing of the structure by tidal flows as explained earlier in this section of the report.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Lissenhall Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

There is no flood risk to properties along the Lissenhall stream. At the downstream reaches of the river, along the left bank floodplain, between Seapoint and Ballymadrough, the road is flooded for the 0.1% AEP fluvial event and for tidal events of 4% AEP or greater. Refer to map LIS/HPW/EXT/CURS/001.

Please refer to Section 5.2.1 for discussion on flooding around the Broadmeadow estuary.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.15m and for the HEFS is 0.22m. For the MRFS, the maximum difference is 0.46m. The most significant increase in flooding resulting from these increases in water levels is at the downstream extent of the river model where the river bed slope is flat and on the Lab tributary, just upstream from the M1 culvert, where the constriction of the structure creates a significant head loss and backwater effect.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.20	0.54	0.42	1.18
Tributary Laa	0.02	0.02	0.03	0.05
Tributary Lab	0.07	0.11	0.46	0.81

5.5. Turvey River

Introduction	
Water bodies	Turvey River
APSR	Donabate area

The Turvey River has its source near Baldurgan and Cookstown; it flows in a south-easterly direction until it meets the Broadmeadow Estuary. The map above provides an overview of the extent of the Turvey River. Please refer to Figure 1 for more details on the extent of the Turvey River hydraulic model and elements of the hydraulic model build. The catchment drains an area of 13.04km² and is broken down into five sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 5.52km. There are no gauging stations on this river. The Turvey River has been modelled together with Broadmeadow Estuary. The tidal/fluvial dominance transition point in the Turvey River is almost indistinguishable (based on the 1% AEP fluvial and tidal event) due to the presence of a flapped outfall at the estuary. The water level is controlled by this flapped outfall.

Model Build

The Turvey River has been modelled together with the Broadmeadow estuary model, as the tidal boundary conditions were calculated at an offshore location at the mouth of the Broadmeadow estuary. The 50% AEP fluvial baseflow of the other rivers flowing into the estuary were not included as the Turvey flows into the open part of the estuary, downstream from the railway viaduct's constriction, unlike the other rivers (Broadmeadow-Ward, Lissenhall and Gaybrook).

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	12	The bridges are modelled using BRIDGE (ARCH and USBPR1978) and ORIFICE units.
Weir	1	The structure was represented by a general weir unit.
Gauging station	0	
Flood defences	1	Flapped outfall
Other	0	

The flapped outfall at the downstream end of the model is considered a formal flood defence. The flapped outfalls prevent the high tides from propagating upstream at any modelled AEP event. Operating rules within the ISIS model unit set the gate to open or close based on the upstream and downstream water level (e.g. the gate is set to close when the downstream tidal water level is higher than the upstream fluvial water level). Additional modelling assessing the impact of the failure of this defence has been undertaken with a discussion of results later in this section of the report.	

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
62	1	0	Main channel (grid size 5m) and the estuary (grid size 10m).
The coastal road near Donabate crosses under the railway line through a tunnel. This tunnel is an important inland flow path at high tides between lands east and west of the railway viaduct. As this feature is not represented in the LiDAR data, a 2D domain Z line was used to represent this structure by lowering the LiDAR locally.			

Representative Manning's *n* values	
Channel. Manning's n varies between 0.03 and 0.06	
0.03: Minor stream, clean, straight, full stage, no rifts or deep pools (node 6Ta504) 	0.06: Natural channel, clay, lateral slopes and bed with irregularities, almost all section with scrubs and trees (node 6Ta2283) 

Floodplain. Manning's n of 0.06	*Culverts. Colebrook-White friction of between 0.030m and 0.006m*
0.06: dispersed bushes, weeds and few trees (6Ta560) 	0.03: Corrugated metal culvert (node 6Ta4822) 

Boundary conditions

Part of the Ballyboghil River overspills into the Turvey catchment. For all design events, this additional inflow was estimated using flow data from the 2D model domain of the Ballyboghil and Corduff model and accordingly distributed along the Turvey River main channel upstream of the M1 motorway. For high fluvial events, this additional flow can peak at twice the flow in the Turvey River upper catchment A tidal boundary at the mouth of the Broadmeadow estuary was used as the downstream boundary unit for the Turvey River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the Turvey River is 9 hours. The same critical storm duration was used for the other AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.05	-0.02	0.20	-0.27
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.07	-0.12	0.23	-0.40

The tables above indicate that in terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. However, larger differences occur locally, with the largest difference located upstream of the M1 long culvert (6Ta4822_C11). Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows with the largest differences occurring locally, just upstream of the M1 long culvert.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition starts to be significant from section 6Ta1229_US, just upstream of the R126 Bridge on the main channel. The model has a flapped orifice on its downstream end that acts as a tidal defence. The differences on tidal sensitivity in the water levels are +/-0.15m approximately due to the restriction on river discharges to the sea while the flap is closed.

The results indicate that the model is relatively insensitive to changes in both Manning's n and model inflows. Notable changes in water levels as a result of an increase and decrease in Manning's n and flows are restricted to one location, where the conveyance capacity of the culvert results in large increases and decreases in water levels. As the water levels in the river are controlled by a flapped outfall, the river channel is not sensitive to changes in downstream tide levels.

'Without defences' scenario

The Turvey river has a flapped outfall that acts as defence against tidal events. The flap valve was removed for the 'without defences' scenarios to determine the areas benefiting from this defence. Flood map TUR/HPW/EXT/CURS/T/002 indicates that the flood extents, for the 10%, 0.5% and 0.1% AEP events, increase significantly upstream of the railway and within Newbridge Demesne when the tidal flap valve is removed. The fluvial flood extent map, TUR/HPW/EXT/CURS/002, also indicates that for the 10%, 1% and 0.1% AEP event flooding increases without the flapped outfall in place. The increased flood risk mainly affects agricultural land, with the Staffordstown industrial estate further upstream not affected by the removal of this flapped outfall.

The difference between the defended areas on the tidal and fluvial maps is as a result of the JPA tide and river flow combinations (refer to Section 4.4.4 for further details on the JPA combinations). For the fluvial scenarios, the fluvial component is more dominant than the tidal component. In this scenario, the high tides prevent the discharge of the river flows to the Broadmeadow Estuary resulting in flooding upstream of the flapped outfall. For the tidal events, the tidal component is more dominant. The fluvial flows for the tidally dominant scenario, although lower than the fluvially dominant scenario, are large and result in flooding upstream of the outfall where high tides prevent flows discharging to the estuary.

Model Results Summary

The following section provides a brief overview of the flood hazard along the Turvey River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The flood maps and hydraulic model show that there is significant flooding just upstream of the M1 motorway. This is primarily caused by flood flows from the Ballyboghil River spilling into the Turvey River upstream of the M1. For extreme fluvial events, this additional flow can peak at twice the flow in the Turvey River upper catchment. The land affected is generally undeveloped agricultural land. However for large flood events, flooding of the northbound lane of the M1 is possible. Just downstream of the M1, the Staffordstown Industrial Estate along with the N1/R132 floods for fluvial events of 4% AEP or greater. Refer to map TUR/HPW/EXT/CURS/001. Downstream of Staffordstown Industrial Estate, within

Newbridge Demesne, there is a sizeable area of natural floodplain which floods for the 20% AEP fluvial event or greater and for the 2% AEP tidal event or greater. Refer to map TUR/HPW/EXT/CURS/002. For discussion on flooding around the Broadmeadow estuary, please refer to Section 5.2.1.

The average water level increase between the current scenario and the MRFS for the 1% AEP fluvial event is 0.12m and for the HEFS is 0.28m. High differences occur principally at the downstream end of the model where the water level is controlled by the flapped outfall. However, the largest increase in water level occurs just upstream of the M1 culvert, where the constriction of the culvert creates a significant head loss and a backwater effect. The increase in water levels results in a marginal increase in extents, with the exception of the downstream end of the model where the increase in extents is more defined.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.12	0.28	0.26	0.80

5.6. Rush Road Stream

Introduction	
Water bodies	Rush Road Stream
APSR	Rush area

The Rush Road Stream has its source near Dellabrown and Ballykea; it flows in a north-easterly direction until it meets the Irish Sea. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the Rush Road Stream model. The catchment drains a small area of 2.05km² and is broken down into two sub-catchments (refer to FEM FRAMS Hydrology Report, 2010). The main channel length is 2.19km and has no tributaries. There are no gauging stations on this river. The model is entirely fluvially dominated as the last cross section of the model has an invert level higher than the maximum tidal events water level (based on the 0.1% AEP current scenario tidal levels). The mouth of the river into the Irish Sea was not modelled due to its steep slope.

Model Build

A 1D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the catchment of the Rush Road Stream. No 2D modelling was undertaken as the river passes through rural areas and its hydraulic behaviour can be accurately modelled using 1D modelling techniques.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	10	Structures modelled using BRIDGE (ARCH and USBPR), ORIFICE and VERTICAL SLUICE units.
Weir	0	
Gauging station	0	
Defences	0	
Other	0	

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
30	0	0	0

Representative Manning's *n* values

Channel. Manning's n varies between 0.035 and 0.045

0.035: Minor stream, clean, straight, full stage, no rifts or deep pools with more stones and weeds (node 14Pa1888)	0.045: Minor stream, clean, winding, some pools and shoals with some weeds and stones (node 14Pa1079)

Floodplain. Manning's n of 0.06	*Culverts. Colebrook-White friction of 0.002m*
0.06: dispersed bushes, weeds and few trees (node 14Pa2330)	0.002: Concrete culvert, monolithic construction against rough forms (node 14Pa884)

Boundary conditions

A normal head boundary is used as the downstream boundary unit for this model. This is because the last cross section in the model has an invert level higher than the maximum tidal event water level. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP on Rush Road Stream is 18 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n and model inflows.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.02	-0.02	0.06	-0.07
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.04	-0.04	0.20	-0.17

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows. However, more significant differences occur locally, with the largest difference located upstream of 14Pa2102 structure due to its small opening (approximately 0.20m^2) and high spill level (i.e. the level at which flood waters will start to spill over the structure).

The results of the sensitivity analyses indicate that the model is on average relatively insensitive to changes in flows or Manning's n. However, there is one location where increases in flows result in a large increase in water levels. As the model is entirely fluvially dominated, no tidal sensitivity has been carried out for this model.

'Without defences' scenario

No defences present in the model.

Model Results Summary

The following section provides a brief overview of the flood hazard along the Rush Road Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

Rush Road stream is located entirely in a rural area. Hydraulic modelling and flood extent maps indicate that there is limited flooding along this watercourse. Some out of bank flooding occurs at two culverts: the first culvert is 437m in length and starts to flood for a 4% AEP fluvial event and the second one is 180m in length and flooding starts for a 2% AEP fluvial event. Refer to map RUR/MPW/EXT/CURS/001.

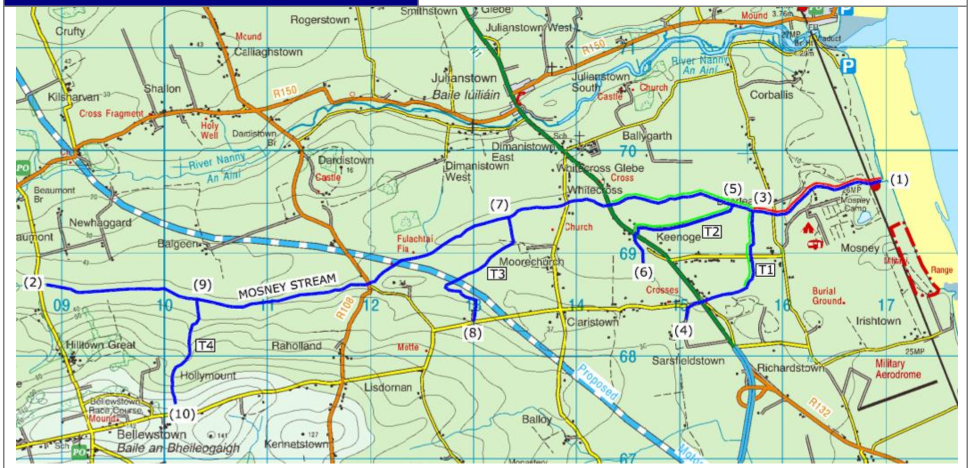
The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.08m and for the HEFS it is 0.17m. The maximum increase in water levels occurs upstream of the bridge at section 14Pa_2102. The increase in water levels results in a limited increase in flood extents.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.08	0.17	0.40	0.98

5.7. Mosney Stream

Introduction

Water bodies	Mosney Stream
APSR	None



The Mosney Stream (also known as the Bradden Stream) has its source at Bellewstown; it flows in an easterly direction and discharges into the Irish Sea at Mosney. The map above provides an overview of the extent of the Mosney Stream and its tributaries. Please refer to Figure 1 for more details on the extent of the Mosney Stream model and elements of the hydraulic model build. The catchment drains an area of 14.6km² and is broken down into six sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 2.8km and has two tributaries which have a combined length of 1.7km. There are no gauging stations on this river. The tidal/fluvial dominance transition point is downstream of the railway crossing based on the 1% AEP fluvial and tidal event.

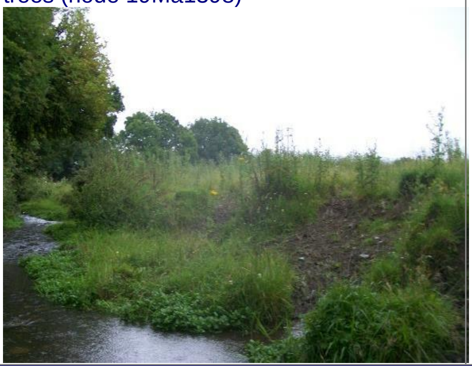
Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through this catchment.

Summary of structures in the model

Type	*Number*	*Summary*
Culvert/Bridge	10	6 culverts/bridges on the main channel and 4 on the tributaries. Bridges modelled using the ARCH and ORIFICE units.
Weir	3	Each structure was represented by a 'Spill' unit due to its irregular shape
Gauging station	0	
Defences	0	
Other	0	

An assessment of culvert blockages was undertaken for the following culvert; a 79m long culvert at Mosney Road (cross section 19Maa548). Further details are contained in Section 9.2.17~~9.2.17~~~~9.1.17~~.

<table>
<tr><td colspan="4">Floodplain model build</td></tr>
<tr><td>Extended cross sections</td><td>Reservoir units</td><td>Parallel river sections</td><td>2D domain</td></tr>
<tr><td>68</td><td>0</td><td>0</td><td>Main channel right bank and tributaries (grid size 5m).</td></tr>
<tr><td colspan="4">Representative Manning's n values</td></tr>
<tr><td colspan="4">Channel. Manning's n varies between 0.03 and 0.06</td></tr>
<tr><td colspan="2">Channel 0.03. Minor stream, clean, straight, full stage, no rifts or deep pools (19Ma2695)
</td><td colspan="2">0.06: Natural channel, clay, lateral slopes and bed with irregularities, almost all section with scrubs and trees (node 19Maa561)
</td></tr>
<tr><td colspan="2">Floodplain. Manning's n of 0.06</td><td colspan="2">Culverts. Colebrook-White friction varies between 0.0002m and 0.0015m</td></tr>
<tr><td colspan="2">0.06: Dispersed bushes, weeds and few trees (node 19Ma1398)
</td><td colspan="2">0.0002m: Concrete culvert (node 19Maa548)
</td></tr>
<tr><td colspan="4">Boundary conditions</td></tr>
<tr><td colspan="4">A tidal boundary at the mouth of the Mosney Stream was used as the downstream boundary unit for this model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.</td></tr>
<tr><td colspan="4">Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the Mosney Stream is 17 hours. The same critical storm duration was used for the other AEP events.</td></tr>
</table>

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.06	-0.07	0.23	-0.31
Tributary Maa	0.04	-0.05	0.09	-0.10
Tributary Mab	0.05	-0.06	0.10	-0.09
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow -20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.09	-0.11	0.27	-0.23
Tributary Maa	0.07	-0.11	0.11	-0.17
Tributary Mab	0.14	-0.09	0.22	-0.28

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels are 0.05m and 0.06m respectively. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows with an average increase and decrease of 0.1m. However, changes in flow and Manning's n result in more significant local differences, with the largest differences occurring upstream of structures.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition starts to be significant just downstream of the Laytown Road long culvert on the main channel. As the model doesn't have tidal defences the differences in the water levels are +/-0.25m approximately in most of the affected area.

Generally, the model is more sensitive to changes in flow than to changes in roughness values. The results for both sets of sensitivity test demonstrate that the largest impacts occur upstream of the structure at 19Ma2723 on the main Mosney Stream channel. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the Mosney Stream as expected.

'Without defences' scenario

No defences present in the model.

Model Results Summary

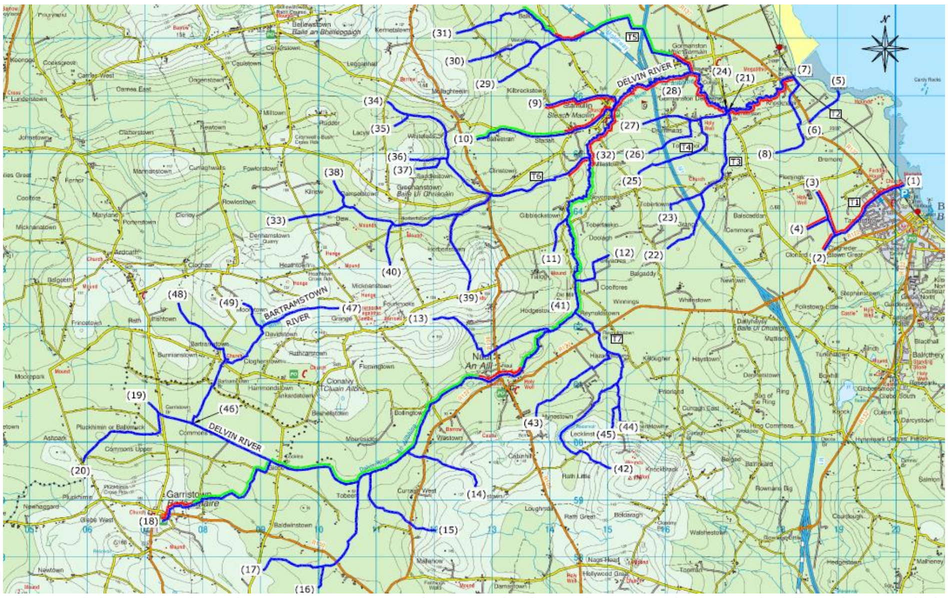
The following section provides a brief overview of the flood hazard along the Mosney Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

Flooding from the Mosney Stream primarily affects agricultural land. The culvert on the Maa tributary (19Maa548) causes overland flooding on the right bank floodplain of this tributary. This overland flooding flows across Mosney Road and Briarleas Road before returning to the main watercourse further downstream. This overland flooding occurs for the 2% AEP fluvial design event or greater and for a 0.1% AEP tidal event. Refer to map MOS/HPW/EXT/CURS/001.

<table>
<tr><td colspan="5">The average water level increase between the current scenario and the MRFS for the 1% AEP fluvial event is 0.11m and for the HEFS is 0.17m. Large differences occur principally at the downstream end of the model, after the railway embankment where the water level is directly controlled by the tide. The increases in water levels result in a marginal increase in extents along the modelled watercourses.</td></tr>
<tr><th rowspan="2">Watercourse</th><th colspan="2">Average Water Level Difference (m)</th><th colspan="2">Maximum Water Level Difference (m)</th></tr>
<tr><th>MRFS</th><th>HEFS</th><th>MRFS</th><th>HEFS</th></tr>
<tr><td>Main channel</td><td>0.11</td><td>0.21</td><td>0.37</td><td>1.02</td></tr>
<tr><td>Tributary Maa</td><td>0.07</td><td>0.10</td><td>0.11</td><td>0.16</td></tr>
<tr><td>Tributary Mab</td><td>0.14</td><td>0.19</td><td>0.23</td><td>0.27</td></tr>
</table>

5.8. Delvin River

Introduction	
Water bodies	Delvin River
APSRs	Garristown area, Naul area, Stamullin area and Gormanston area



The Delvin River has its source at Garristown; it flows in a north-easterly direction until it discharges to the Irish Sea at Gormanstown. The map above provides an overview of the extent of the Delvin River and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model and elements of the hydraulic model build (e.g. 2D model domains). The catchment drains an area of 79.37km² and is broken down into 21 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 19.5km; there are four tributaries which have a combined length of 7.6km and a loop with a length of 0.96km. The Naul gauging station is located at section 18Da11980 and the Garristown gauging station is located at section 18Da18962U. The tidal/fluvial dominance transition point is at model cross section 18Da361 based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised areas of the catchment.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	60	27 culverts/bridges on the main channel and 33 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR), ORIFICE and VERTICAL SLUICE units.
Weir	8	7 weirs on the main channel and 1 on the tributaries. Each structure was represented by a SPILL unit due to its irregular shape.

Type	*Number*	*Summary*
Gauging station	2	The Naul gauging station and Garristown gauging station
Flood defences	0	
Other	5	Weir and gate represented by a SPILL and a SLUICE unit, respectively. 2 sudden drops in bed level caused by stones represented by SPILL units. River expansion represented by BERNOULLI LOSS unit. River narrowing represented by SPILL unit.

Floodplain model build

Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
399	3	0	Stamullin urban area and Commons Lower reservoir area (grid size 5m).

Representative Manning's *n* values

Channel. Manning's n varies between 0.03 and 0.04

0.03: Minor stream, clean, straight, full stage, no rifts or deep pools (node 18Da3539)	0.04: Minor stream, clean, winding with some pools and sandbanks (node 18Da19223)
	
Floodplain. Manning's n varies between 0.03 and 1.0	*Culverts. Colebrook-White friction varies between 0.002m and 0.02m*
0.06: Dispersed bushes, weeds and few trees (node 18Da1546)	0.02m: Corrugated metal culvert (node 18Daa1280)
	

Boundary conditions

A tidal boundary at the mouth of the Delvin River was used as the downstream boundary unit for this model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP on Delvin River is 23 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.06	-0.07	0.16	-0.23
Tributary Daa	0.05	-0.06	0.10	-0.12
Tributary Dab	0.01	-0.01	0.06	-0.05
Tributary Dac	0.03	-0.03	0.07	-0.06
Tributary Dad	0.07	-0.08	0.13	-0.15
Tributary Daq	0.03	-0.05	0.09	-0.12
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.08	-0.10	0.34	-0.28
Tributary Daa	0.06	-0.07	0.13	-0.15
Tributary Dab	0.02	-0.02	0.06	-0.06
Tributary Dac	0.04	-0.04	0.16	-0.16
Tributary Dad	0.08	-0.10	0.18	-0.19
Tributary Daq	0.03	-0.06	0.09	-0.16

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient with an average increase and decrease of 0.04m and 0.05m respectively. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows with an average increase and decrease of 0.05m and 0.07m respectively. However, changes in flow and Manning's n result in larger local differences, with the largest differences occurring upstream of structures. The maximum difference is located upstream of the culvert at the M1 (for Q+20%) and upstream of the bridge at section 18Da5557 (for Q-20%), both located in Stamullin.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition point is at model cross section 18Da499. As the model doesn't have tidal defences the differences in the water levels are +/-0.25m approximately in most of the affected area.

The results indicate that the model is more sensitive to changes in flow than changes in Manning's n, however, on average there is a minimal impact on levels when either model parameter is changed. The more significant impacts occur at localised areas on the main river channel. Changes in flow result in average differences in the order of +/-0.09m. As the model doesn't have a tidal defence, sensitivity to tidal level is along the tidal reaches of the Delvin River as expected.

'Without defences' scenario
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Delvin River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The majority of flooding along the Delvin River is confined to agricultural land. In Stamullin area APSR, a long culvert (270m length) causes out of bank flooding for the 50% AEP fluvial event due to the low flow capacity at the inlet. Refer to map DEL/HPW/EXT/CURS/003.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.07m and for the HEFS is 0.08m. The largest increases in water level occur near the mouth of the river. The increases in water levels result in a marginal increase in extents along the modelled watercourses.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.08	0.13	0.35	0.97
Tributary Daa	0.06	0.09	0.13	0.20
Tributary Dab	0.02	0.04	0.08	0.13
Tributary Dac	0.04	0.05	0.17	0.25
Tributary Dad	0.09	0.13	0.19	0.27
Tributary Daq	0.03	0.04	0.08	0.12

5.9. Brookside Stream

Introduction	
Water bodies	Brookside Stream
APSRs	Laytown, Bettystown and Coastal area

The Brookside Stream has its source near Ministown; it flows in an easterly direction until it discharges to the Irish Sea south of Bettystown. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the hydraulic model. The catchment drains a small area of 1.68km² and is broken down into three sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 2.20km with a 700m loop. There are no gauging stations on this river. The tidal/fluvial dominance transition point is immediately downstream of the Coast Road based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanized areas of the catchment.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	14	Structures modelled using BRIDGE (ARCH and USBPR), ORIFICE and VERTICAL SLUICE units.
Weir	0	
Gauging station	0	
Flood defences	0	
Other	0	

An assessment of culvert blockages was undertaken for the following culvert: a 13m long culvert at Laytown Road. Further details on the culvert blockage assessment are in Section 9.2.19~~9.2.19~~~~9.1.19~~.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
30	0	1	Main channel and loop after railway crossing (grid size 5m).

Representative Manning's *n* values	
Channel. Manning's n of 0.03	*Floodplain. Manning's n varies between 0.03 and 1.0*
Minor stream, clean, straight, full stage, no rifts or deep pools (node 21Ma1305) 	0.06: dispersed bushes, weeds and few trees (node 21Ma2194) 
Culverts. Colebrook-White friction varies between 0.0015 and 0.007m	
0.0015m: Concrete culvert, monolithic construction against rough forms (node 21Ma63) 	0.007m: stone culvert (node 21Ma1262) 
Boundary conditions	
A tidal boundary at the mouth of the Brookside Stream was used as the downstream boundary unit for this model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.	

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the Brookside Stream is 9 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.03	-0.09	0.07	-0.24
Loop Mab	0.04	-0.08	0.08	-0.13
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.05	-0.07	0.10	-0.22
Loop Mab	0.05	-0.04	0.08	-0.07

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient with an average increase and decrease along all of the watercourses of 0.03 and 0.08m respectively. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows (+0.05m for increase in flows and -0.06 for decrease in flows). However, changes in flow and Manning's n result in more significant local differences, with the largest differences occurring upstream of structures. The results indicate that the Ministown Road culvert (around 21Ma1959) is most sensitive to changes in both inflows and roughness values.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition is almost undefined as it occurs at the very end of the main channel.

The sensitivity results demonstrate that the upper part of the main channel just upstream of the Ministown Road culvert (around 21Ma1959) is the most sensitive to changes in model inflows and roughness values as a result of surcharging of the river culvert and backing up in the river channel. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the Brookside Stream as expected.

'Without defences' scenario
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Brookside Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The Brookside Stream is generally rural and the majority of the flooding is confined to agricultural land along the watercourse. At the downstream end of the modelled reach, the R150 coast road bridge causes a constriction forcing the water to back up in the channel and flood surrounding land. Refer to map BSS/HPW/EXT/CURS/001. A primary school has recently been built in this area and there is a planning application for a secondary school in this area also. It is understood that Meath County Council has applied to the OPW for funding for a flood defence scheme for this local area.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.06m and the maximum difference is 0.12m. The largest differences occur at the downstream end of the model, on the sea shore where the water level is directly controlled by the tide.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.05	0.09	0.11	0.70
Loop Mab	0.07	0.11	0.12	0.16

5.10. Ballyboghil and Corduff Rivers

5.10.1. Ballyboghil River

Introduction	
Water bodies	Ballyboghil River
APSRs	Ballyboghil area and Oldtown area

The Ballyboghil River has its source at Oldtown; it flows in a south-easterly direction until it reaches the Rogerstown Estuary. Before reaching the estuary (the east of the M1 and N1), the river is joined by a large tributary, the Corduff River. The estuary itself flows in an easterly direction until it reaches the Irish sea. The map above provides an overview of the extent of the Ballyboghil River and its tributaries. Please refer to Figure 1 for more details on the extent of the Ballyboghil and Corduff River model and elements of the hydraulic model build (e.g. 2D model domains). The catchment of the Ballyboghil River drains an area of 45.4km² and is broken down into 10 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 11.2km, its tributary has a length of 2.2km (excluding the Corduff River, which is reported on in Section 5.10.2) and the Rogerstown Estuary adds another 2.2km. There is one gauging station on this river: Stn 08012 at Ballyboghil. The tidal/fluvial dominance transition point is at the R132 bridge based on the 1% AEP fluvial and tidal event.

Model Build

The Ballyboghil River forms part of the larger Ballyboghil and Corduff River model. The two rivers were modelled as one river model to ensure that any interaction in flood flows between the rivers is accurately captured. The full hydraulic model includes the Ballyboghil River, the Corduff River and the Rogerstown estuary. This section provides details of the Ballyboghil River element of the model which includes the Rogerstown estuary. Information on the Corduff River is detailed in Section 5.10.2.

The Ballyboghil and Corduff River model has been modelled together with the Rogerstown estuary. The estuary extends eastwards from the N1 roadway to the coastal town of Rush. A number of other watercourses discharge to the estuary including the Baleally Stream, Bride´s Stream and the Rush West Stream. Therefore, a 50% AEP fluvial base flow discharging to the Rogerstown Estuary from these watercourses was considered. However, the baseflows from these watercourses were considered negligible compared to volume propagating through the estuary from the tidal cycles and have not been included in the model.

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised areas of the catchment.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	29	18 culverts/bridges on the main branch of the Ballyboghil and 11 on the tributary. Structures modelled using BRIDGE (ARCH and USBPR), CULVERT and ORIFICE units.
Weir	2	2 weirs on the main channel. Each structure was represented by a SPILL unit due to its irregular shape.
Gauging station	1	Ballyboghil Gauging Station 08012
Flood defences	0	
Other	2	1 bed constriction due to piers of the railway bridge on the Rogerstown Estuary represented by SPILL units. 1 river expansion represented by BERNOULLI LOSS unit on the Rogerstown Estuary.

An assessment of culvert blockages was undertaken on the Ballyboghil River at Ballyboghil Bridge, a 6.7m long bridge with 2 openings separated by a vertical pier. Further details on the culvert blockage assessment are in Section 9.2.10~~9.2.109.1.10~~.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
168 (Ballyboghil) + 19 (Rogerstown Estuary)	0	2	One in the Ballyboghil village and one at the confluence with the Corduff (grid size 5m).

Representative Manning's *n* values	
Channel. Manning's n of between 0.030 and 0.035	
0.035. Natural channel, clay, some irregularities on lateral slopes, regular and clean bed (node 7Ba3582)	0.03. Open channel, clean, straight, full stage, no rifts or deep pools (node ROG38)
	

Floodplain. Manning's n varies between 0.03 and 1.0	*Culverts. Colebrook-White friction varies between 0.002m and 0.025m*
Floodplain Manning's n of 0.06; dispersed bushes, weeds and few trees (node 7Ba7248) 	0.025m: Corrugated metal culvert (node 7Bab803) 

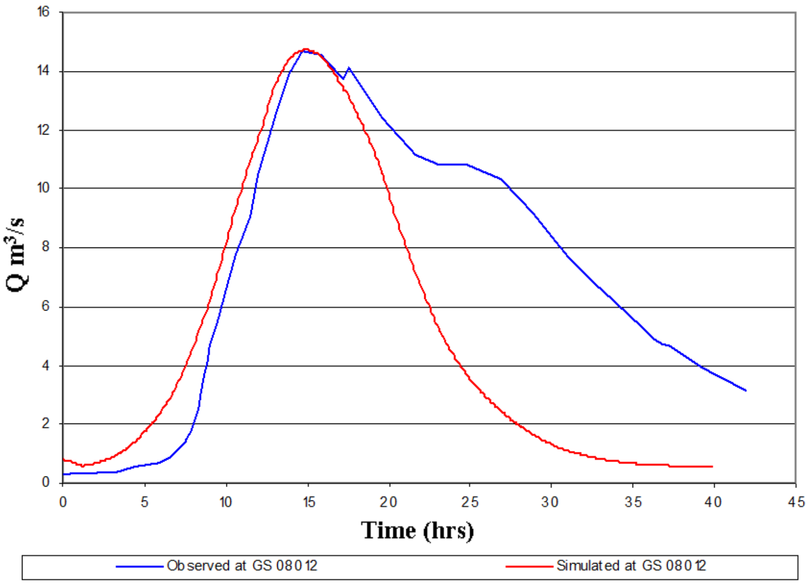
Boundary conditions

A tidal boundary at the mouth of Rogerstown estuary was used as the downstream boundary unit for the Ballyboghil and Corduff River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

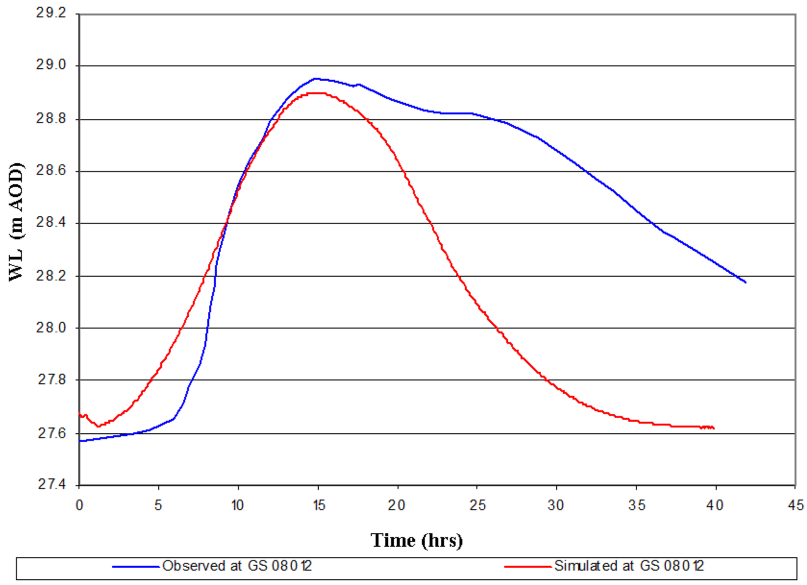
Model calibration

For calibration, two historic flood events were available; November 1982 and August 1986. As far as we know, there has been no flood defence/construction works carried out along the river since these dates. Therefore the design model was used to calibrate both events.

November 1982

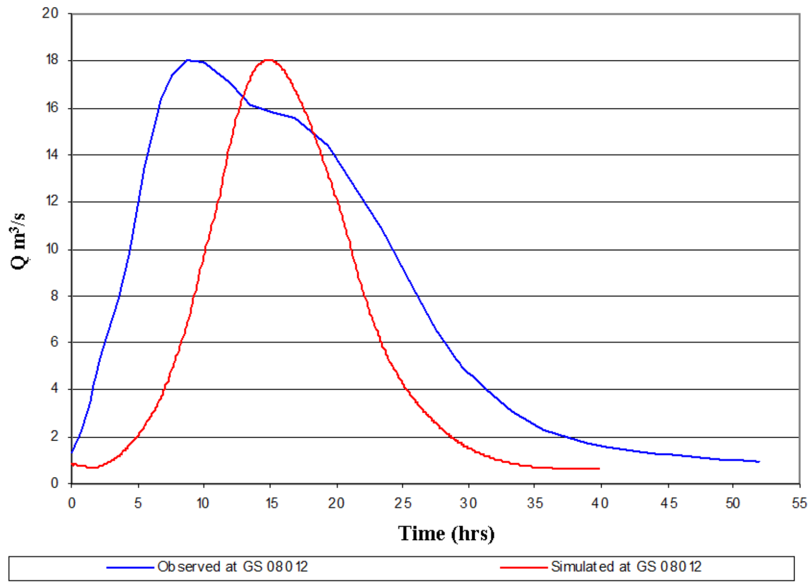


Comparison between the observed November 1982 flow hydrograph (blue) and the calibrated model flow hydrograph (red) at Ballyboghil gauging station.

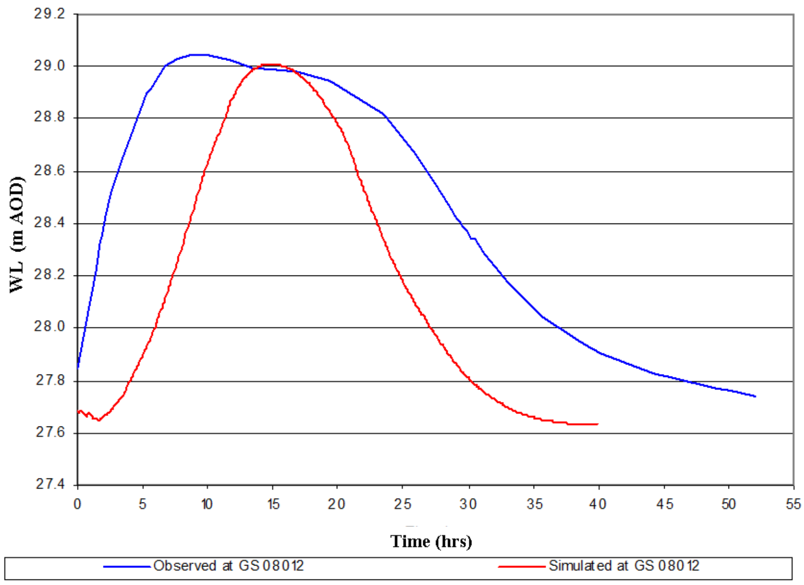


Comparison between the observed November 1982 stage (blue) and calibrated model stage (red) at Ballyboghil gauging station.

August 1986



Comparison between the observed August 1986 flow hydrograph (blue) and the calibrated model flow hydrograph (red) at Ballyboghil gauging station.



Comparison between the observed August 1986 stage (blue) and calibrated model stage (red) at Ballyboghil gauging station.

For the 1982 flood event, the modelled peak flow is within 0.1m^3 of the observed flow. The modelled water level is 0.06m less than the observed flood level.

For the 1986 flood event, the modelled peak flow is within 0.1m^3 of the observed flow. The modelled water level is 0.04m less than the observed flood level.

The results of the calibration demonstrate that the models match the observed peak flows and levels at the gauging station within acceptable limits (i.e. <0.2m). The peak of the modelled hydrograph and rising limb shape match with the observed hydrograph although the shapes of the falling limb of the hydrograph do not fully match with the observed hydrographs. The flatter falling limb of the observed hydrographs might be as a result of possible blockages in the vicinity of the GS, which may have been removed since the 1986 flood. However, as there is an excellent calibration to both peak flow and level and as the flood extent maps are consistent with historic records, the model is considered to be representative of observed conditions.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the Ballyboghil River is 15 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.04	-0.06	0.24	-0.23
Tributary Baa	0.06	-0.08	0.19	-0.30
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.06	-0.09	0.17	-0.30
Tributary Baa	0.09	-0.11	0.31	-0.28

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient with an average increase and decrease along all of the watercourses of 0.05m and 0.07m with increases and decreases in Manning's n. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows. However, changes in flow and Manning's n result in more significant local differences, with the largest differences occurring upstream of structures. The most significant local impacts resulting from changes to roughness values occur in the main river channel for n+20% and in the tributary for n-20%. With increased n values, the largest change in water levels occurs upstream of the structures at sections 7Ba9077 and 7Ba9020. In the tributary, the largest change in water levels occurs at section 7Baa912. For increases in flow, the largest impact occurs upstream of the Ballyboghil Bridge on the main river channel. In the tributary, the biggest impact occurs upstream of the 7Baa478 structure.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition point is at model cross section 7Ba1262 (just downstream the M1 motorway) in the main river channel and at section 7Bab354 on the tributary. As the model doesn't have tidal defences the differences in the water levels are +/-0.25m approximately in most of the affected area.

The results indicate that the model is, on average, relatively insensitive to changes in both Manning's n and model inflows with notable increases and decreases in water levels occurring upstream of structures with low conveyance capacity. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the Ballyboghil River as expected.

'Without defences' scenario
No defences present in the model

Model Results Summary
The following section provides a brief overview of the flood hazard along the Ballyboghil River and around Rogerstown estuary. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The results from the Ballyboghil model have been used to prepare the flood hazard maps for the Rogerstown estuary. Flooding around the Rogerstown estuary is primarily confined to agricultural land. To the east of the estuary properties are at risk of flooding both at Rush and The Burrows.

To the west of the Rogerstown estuary, immediately upstream of the M1 motorway, flooding from the Ballyboghil River interacts with flood water from the Turvey River to the south. This interaction in flood flows increases the flows in the Turvey River resulting in surcharging of the Turvey River culvert under the M1 motorway (refer to Section 5.5 and map BAL/HPW/EXT/CURS/003 for further details).

Further upstream, flooding occurs both upstream and downstream of Ballyboghil at Ballyboghil Bridge/R108 and the R129 which runs parallel to the river, Flooding at Ballyboghil starts for the 4% AEP fluvial event. Some properties on right bank upstream of Ballyboghil Bridge and on Riverside Street are at risk as a result of this flooding. Refer to map BAL/HPW/EXT/CURS/002.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.09m and for the HEFS is 0.17m. The maximum difference is 0.34m and is located at the confluence with the Corduff and the Rogerstown estuary. The largest increase in flood risk is around the Rogerstown estuary which is primarily as a result of the increase in sea levels. There is a limited increase in flood risk along the fluvial reaches of the watercourse.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.09	0.21	0.34	0.98
Tributary Baa	0.09	0.14	0.28	0.41

5.10.2. Corduff River

Introduction	
Water bodies	Corduff River
APSRs	None

The Corduff River has its source near Brownstown and Gerrardstown. It flows in a south-easterly direction until it reaches the Ballyboghil River east of the M1 and N1. The map above provides an overview of the extent of the Corduff River and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model and elements of the hydraulic model build. The catchment drains an area of 32.76km² and is broken down into 10 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main modelled channel length is 10.9km and the single modelled tributary adds another 0.7km. There are no gauging stations on this river. The tidal/fluvial dominance transition point is at the R132 bridge based on the 1% AEP fluvial and tidal event.

Model Build

The Corduff River forms part of the larger Ballyboghil and Corduff River model. The two rivers were modelled as one river model to ensure that any interaction in flood flows between the rivers is accurately captured. The full hydraulic model includes the Ballyboghil River, the Corduff River and the Rogerstown estuary. This section provides details of the Corduff River element of the model. Information on the Ballyboghil River and Rogerstown estuary is detailed in Section 5.10.1. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the this catchment.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	19	18 culverts/bridges on the main channel and 1 on the tributary. The bridges were modelled using BRIDGE (ARCH), the culverts have been modelled using ORIFICE and VERTICAL SLUICE units.

Type	*Number*	*Summary*
Weir	1	1 weir on the main channel that was represented by a SPILL unit due to its irregular shape.
Gauging station	0	
Flood defences	0	
Other	1	1 river expansion represented by BERNOULLI LOSS unit on the tributary.

An assessment of culvert blockages was undertaken on the Corduff River at the R132. The main channel of the Corduff River passes through a 19.4m culvert where it crosses the R132 (Old N1) (cross section 8Ca1129). Further details on the culvert blockage assessment are in Section 9.2.11~~9.2.11~~~~9.1.11~~.

Floodplain model build

Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
283	0	2	One in the Ballyboghil village and one at the confluence with the Corduff (grid size 5m).

Representative Manning's *n* values

Channel. Manning's n of 0.03	*Floodplain. Manning's n of 0.06*
0.03: Minor stream, clean, straight, full stage, no rifts or deep pools with stones and weeds (node 8Ca3183)	0.06: dispersed bushes, weeds and few trees (node 8Ca301)
	

Culverts

No Culvert Units have been used in the Corduff River model

Boundary Conditions

A tidal boundary at the mouth of Rogerstown estuary was used as the downstream boundary unit for the Ballyboghil and Corduff River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP for the Corduff River is 15 hours (same as Ballyboghil River). The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.08	-0.09	0.18	-0.23
Tributary Caa	0.02	-0.01	0.06	-0.07
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.08	-0.09	0.22	-0.32
Tributary Caa	0.00	0.00	0.02	-0.02

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is 0.05m (n+20%) and -0.05m (n-20%). Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows. The main channel is more sensitive than the tributary to changes in both roughness values and model inflows. Changes in roughness values result in average changes in water level of 0.08m. Increasing the model inflows by 20% has the most significant impact upstream of the bridge and lateral structure through the R132.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition point is at model cross section 8Ca611. As the model doesn't have tidal defences the differences in the water levels are +/-0.25m approximately in mostly of the affected areas.

The results indicate that the model is, on average, relatively insensitive to changes in both Manning's n and model inflows with notable increases and decreases occurring at structures with low conveyance capacity. As the model doesn't have a tidal defence, sensitivity to tidal level is along the tidal reaches of the river as expected.

'Without defences' scenario (ABDs)
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Corduff River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The flood maps indicate that there is limited flooding along the Corduff River. At the upstream extent of the modelled watercourse, near Gerrardstown, there is no flooding up to and including the 0.1% AEP fluvial event. Further downstream there are small pockets of flooding affecting agricultural land. Refer to map COR/HPW/EXT/CURS/001. Upstream of the N1 and the R127 road bridges there is some localised flooding affecting agricultural land which starts at the 4% AEP fluvial event. Refer to map COR/HPW/EXT/CURS/002.

Please refer to Section 5.10.1 for discussion on flooding around the Rogerstown estuary.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.09m and for the HEFS is 0.16m. The maximum difference is 0.34m and is located in the tidal reaches of the watercourse at the confluence with the Ballyboghil and the Rogerstown estuary and is primarily as a result of increases in mean sea levels. Along the fluvial reaches of the Corduff River there is a limited increase in flooding associated with the MRFS.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.09	0.16	0.34	0.99
Tributary Caa	0.00	0.00	0.02	0.03

5.11. Balbriggan North Stream

Introduction	
Water bodies	Balbriggan North Stream
APSRs	Balbriggan area

The Balbriggan North Stream has its source to the west of Balbriggan and flows in a south and then a north-easterly direction where it discharges to the Irish Sea at Balbriggan. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the hydraulic model. The catchment drains a small area of 3km² and is broken down into six sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 1.7km and there are two tributaries that have a combined length of 0.62km. The entire main channel and the 17Tab tributary are culverted. There are no gauging stations on this river. The model is entirely fluvially dominated as the last cross section of the model has an invert level higher than the maximum tidal event water level (based on the 0.1% AEP current scenario tidal levels).

Model Build

As the majority of this watercourse is culverted through an urban area, the most appropriate method for modelling this watercourse would be an urban drainage modelling tool such as InfoWorks CS. However, as urban drainage modelling is outside the scope of the project, a combined 1D-2D hydrodynamic model was selected to simulate the routing of fluvial flows.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	3	1 culvert on the main channel, 1 on the tributary and 1 bridge on the open channel. The bridge was modelled by an ORIFICE unit. Culvert modelled as a Conduit Unit.
Weir	0	
Gauging station	0	
Flood defences	0	
Other	0	

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
1	0	0	The entire model (grid size 5m).

Representative Manning's *n* values	
Channel. Manning's n of 0.03	*Floodplain. Manning's n varies between 0.03 and 1.0*
0.03: Minor stream, straight, full stage, no rifts or deep pools (node 18Taa8) 	0.06; dispersed bushes, weeds and few trees (node 18Taa284) 
Culverts. Colebrook-White friction of 0.006m	
0.006: Concrete culvert, monolithic construction against rough forms (node: 17Tab18) 	

Boundary condition
A normal head boundary is used as the downstream boundary unit for this model. This is because the last cross section in the model has an invert level higher than the maximum tidal event water level. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP fluvial event on the Balbriggan North Stream is 2.5 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

Sensitivity analysis to changes in model parameters (flow and Manning's n) was not possible due to the extent of the culverted reach of the water course. For the 1% AEP event (i.e. the event used for sensitivity testing), the flow through the culverted reach of the watercourse is pressurised and it was not possible to analyze the impact of adjusting model parameters on predicted water levels under these hydraulic conditions. In addition, as the model is entirely fluvially dominated, no tidal sensitivity has been carried out for this model.

'Without defences' scenario
No defences present in the model

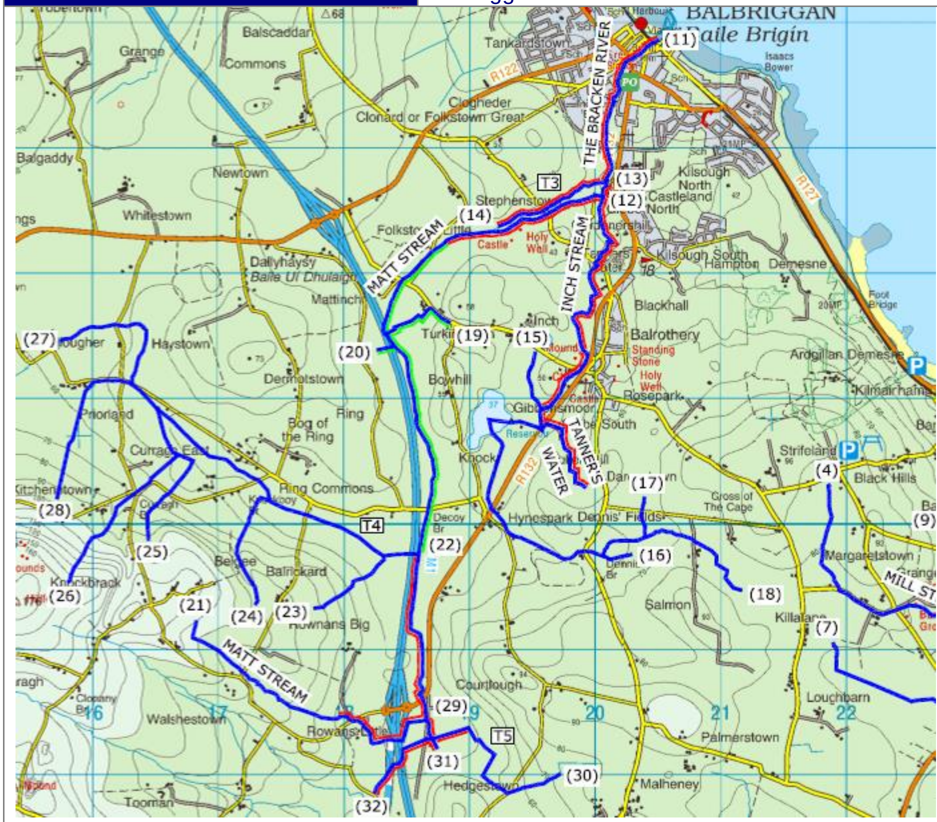
Model Results Summary
The following section provides a brief overview of the flood hazard along the Balbriggan North Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

There is no flooding along the Balbriggan North Stream for all events with the exception of the 0.1% AEP fluvial event. For the 0.1% AEP event, the surcharged culvert floods Drogheda Street via the manholes with the flood water flowing in an easterly direction to the beach in Balbriggan. The flooding puts a number of properties at risk along Drogheda Street in Balbriggan. Refer to map BNS/HPW/EXT/CURS/001.

For the MRFS, flooding along Drogheda Street starts at higher order AEP events. For the 0.1% AEP event, there is a sizeable increase in flooding with flooding extending to Moylaragh Cresent and Moylaragh Park.

5.12. Bracken River

Introduction	
Water bodies	Bracken River
APSR	Balbriggan area and Rowans Little area



The Bracken River has its source near Hedgestown and Rowans Little; it flows in a north-easterly direction where it discharges to the Irish Sea at Balbriggan Harbour. The map above provides an overview of the extent of the Bracken River and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model and elements of the hydraulic model build (e.g. 2D model domains). The catchment drains an area of 27.8km² and is broken down into 14 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 8km and there are five tributaries that have a combined length of 5.2km. There are no gauging stations on this river. The tidal/fluvial dominance transition point is at model cross section 16Ma34 based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised catchment of the Bracken River.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	54	The bridges were modelled using BRIDGE (ARCH), ORIFICE and VERTICAL SLUICE units.

Type	*Number*	*Summary*
Weir	6	All weirs on the main channel. 3 weirs were represented using a ROUND NOSE WEIR unit and 3 were represented by a SPILL unit due to their irregular shape.
Gauging station	0	
Flood defences	1*	**Refers to location.* Informal effective defences included at one location.
Other	1	1 crossing pipe represented by BERNOULLI LOSS unit on the main channel.

An assessment of culvert blockages was undertaken for two locations: the Bracken River at Decoy Bridge (a 36m long culvert at Decoy Bridge - cross section 16Ma5361) and the Bracken River at R132 Bridge (a 13.5m long stone bridge where it crosses Bridge Street in Balbriggan – cross section 16Ma244). Further details on the culvert blockage assessment are in Sections 9.2.15~~9.2.159.1.15~~ and 9.2.16~~9.2.169.1.16~~.

In Balbriggan, it was assumed, based on available information (refer to Section 4.4.6), that the walls along the left and right bank form a flood defence function. The defence consists of walls which form part of a building and free standing walls alongside the channel (the image opposite shows a sample of these defences). The walls form a continuous defence against flooding from nodes 16Ma270 to 16Ma162 and on both left and right banks. These defences were surveyed as part of the DAS and have been represented in the hydraulic model as ISIS HX lines. The elevation of the defences was obtained from the channel and structure cross sectional survey data. A map showing the location of the defences in the model is available in Appendix C3.

The impact of these defences on flood extents is discussed in the 'Without defence's scenario' section. An analysis of flood risk and flood hazard due to sudden failure of these defences is reported on in Chapter 8.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
283	10	1	Final 250m of the main channel (grid size 5m).

The pillars supporting the railway line which runs through Balbriggan are significant structures in the floodplain. The LiDAR filtering process removed these structures from the DTM. In order to model the impact of these structures on flood flows in the 2D model domain, the DTM was manually adjusted to raise ground levels in the vicinity of these pillars.

<table>
<tr><th colspan="2">Representative Manning's n values</th></tr>
<tr><th colspan="2">Channel. Manning's n varies between 0.035 and 0.040</th></tr>
<tr><td>0.035: Minor stream, clean, straight, full stage, no rifts or deep pools with stones and weeds (node 16Ma6047)
</td><td>0.04: Minor stream, clean, straight, full stage, no rifts or deep pools with more stones and weeds (node 16Ma178)
</td></tr>
<tr><th>Floodplain. Manning's n of between 0.03 and 1.0</th><th>Culverts. Colebrook-White friction varies between 0.002 and 0.006 m</th></tr>
<tr><td>0.06: dispersed bushes, weeds and few trees (node 16Ma5948)
</td><td>0.002: Concrete culvert, monolithic construction against rough forms (node 16Ma6983)
</td></tr>
<tr><th colspan="2">Boundary conditions</th></tr>
<tr><td colspan="2">A large tributary crosses the Bog of the Ring and the M1 before joining the main channel, at node 16Ma5726. This tributary was not surveyed, but had to be considered in the model as a hydrological input. It was therefore assumed in the 1D model, that its inflow would be directly linked to a reservoir unit representing the left floodplain of the main channel on the western side of the motorway. This reservoir flows into the main channel through a culvert that crosses the M1. A tidal boundary at the mouth of the river was used as the downstream boundary unit for this model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.</td></tr>
</table>

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP event on the Bracken River is 21 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.05	-0.06	0.13	-0.16
Loop branch	0.01	-0.01	0.05	-0.04
Tributary Mab	0.04	-0.05	0.11	-0.13
Tributary Mac	0.01	0.00	0.04	0.00
Tributary Mae	0.03	-0.05	0.05	-0.06
Tributary Maf	0.02	-0.03	0.04	-0.05

Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow -20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.11	-0.13	0.41	-0.65
Loop branch	0.01	-0.01	0.03	-0.04
Tributary Mab	0.10	-0.10	0.28	-0.38
Tributary Mac	0.00	0.00	0.00	0.00
Tributary Mae	0.10	-0.18	0.12	-0.30
Tributary Maf	0.16	-0.34	0.48	-0.78

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is +0.03m and -0.03m for increases and decreases in Manning's n. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows with water levels along all of the watercourses increasing by 0.08m (flows +20%) and decreasing by 0.13m (flows -20%). The most significant changes in water levels as a result of changes in the model parameters occur at structures. The Manning's n sensitivity analysis results indicate that the model is more sensitive to changes in the downstream section of the channel at 16Ma812 in and just upstream of the R132 Bridge in Balbriggan (16Ma244U). Changes to model inflows result in a notable impact on water levels on the 16Maf Tributary just upstream of the M1 long culvert. In the main river channel, the greatest impact occurs upstream of the M1 culvert (16Ma7600U) and at the disused online embankment structure just upstream of Balbriggan (16Ma433U).

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition starts to be significant downstream from the 16Ma162 weir in Balbriggan, on the main channel. As the model doesn't have tidal defence the differences in the water levels are +/-0.25m approximately in most of the affected area.

The results indicate that the model is, on average, relatively insensitive to changes in both Manning's n and model inflows with notable increases and decreases in water levels occurring locally at structures with low conveyance capacity. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the river as expected.

'Without defences' scenario
As described in the 'Summary of structures in the model' informal effective defences consisting of walls have been included in the model. As some of the defences consist of walls which form part of a building, the decision was made not to remove these for the 'without defences' model run. Only the free standing walls alongside the left bank of the channel between sections 16Ma244 and 16Ma162 were removed for the 'without defences' model run. The flood extent map, BRA/HPW/EXT/CURS/003, indicates that these walls provide protection to a limited area in Balbriggan town centre. However, two properties benefit from this wall with protection provided against the 10% AEP fluvial design event or greater and for the 0.5% AEP tidal event or greater.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Bracken River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

At the upstream extent of the modelled watercourse, there is a large area of land flooded in the vicinity of Rowanstown. Upstream of Decoy Bridge, between Hynespark and the M1 motorway, the left and right bank floodplains of the Bracken River floods for the 10% AEP fluvial design event or greater. Further downstream, on the western side of the M1 motorway, the Bog of the Ring area floods for all fluvial design events. Refer to map BRA/HPW/EXT/CURS/001.

At Glebe South, a large area of land along the left bank of the Mab tributary (Inch stream/Tanners Water) starts to floods for 10% AEP fluvial design event. Refer to map BRA/HPW/EXT/CURS/002.

In Balbriggan, properties are at risk of flooding for the 4% AEP fluvial design event or greater principally around Bridge Street and along Quay Street. Bridge Street is flooded for a 0.5% AEP fluvial design event or greater and for a 0.1% AEP tidal event. The car parks next to the harbour (between Mill Street and Quay Street) flood for a 4% AEP fluvial design events or greater and for a 0.2% AEP tidal event or greater. Refer to map BRA/HPW/EXT/CURS/003 and BRA/HPW/EXT/CURS/T/003.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.11m and the maximum difference is 0.49m. The average water level increase between the current scenario and HEFS for 1% AEP fluvial event is 0.12m. High differences occur principally at the downstream end of the model, within Balbriggan, where the water level is controlled by the tide. The largest difference occurs just upstream of the M1 culvert on the Maf tributary, where the constriction caused by the bridge creates a significant head loss and backwater effect upstream.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.12	0.19	0.47	1.09
Loop branch	0.01	0.02	0.04	0.05
Tributary Mab	0.10	0.16	0.29	0.39
Tributary Mac	0.00	0.00	0.00	0.00
Tributary Mae	0.10	0.14	0.12	0.17
Tributary Maf	0.16	0.19	0.49	0.51

5.13. Mill Stream

Introduction	
Water bodies	Mill Stream
APSRs	Skerries area

The Mill Stream flows through Skerries town in a north-easterly direction until it discharges to the Irish sea at Strand Road, Skerries. The map above provides an overview of the extent of the Mill Stream and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model and elements of the hydraulic model build (e.g. 2D model domains) The catchment drains an area of 8.19km² and is broken down into eight sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 3.19km and has four modelled tributaries that have a combined length of 0.8km. There are no gauging stations on this river. The tidal/fluvial dominance transition point is at model cross section 15Ma200 based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised catchment of the Mill Stream.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	25	19 culverts/bridges on the main channel and 6 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR), ORIFICE and VERTICAL SLUICE units.
Weir	0	
Gauging station	0	

Type	*Number*	*Summary*
Flood defences	2*	**Refers to location.* Informal effective defences included at two locations
Other	3	Control structure downstream of the railway embankment represented by 2 SLUICES and a SPILL unit. Reservoir outlet represented by a SLUICE unit. Pipe over the river represented by BERNOULLI LOSS unit.

An assessment of culvert blockages was undertaken for the following culvert; Mill Stream at Holmpatrick Road, where the main channel passes through a 13.5m stone bridge (cross section 15Ma222). Further details on the culvert blockage assessment are in Section 9.2.14~~9.2.14~~~~9.1.14~~.

According to the data made available by Fingal County Council, there are no surface water drains connected to the pond in Skerries Park, and therefore in the model there is no lateral inflow connected to the reservoir unit representing this pond. In addition, it was not possible to identify any drains flowing into the pond from the Greenlawns Estate using survey photographs and satellite images. The reservoir is generally full of water and the LiDAR survey measures to the top water level rather than the reservoir bed levels. The DTM levels were decreased by approximately 1.5m to represent the actual bed level of the reservoir using the surveyed bed levels.

There are two culverts under the railway embankment, one for the main channel (from the north) and one for the 15Maa tributary (from the south). At some point these culverts join another 80m culvert that flows into the control structure downstream of the railway underpass. The control structure diverts the flow between the river channel (with an on-line weir and a sluice) and the pond (with a second sluice). Access to this chamber was difficult so the exact dimensions and levels of the three structures were defined based on a sketch and photographs provided by the surveyors. The culverts have been modelled as orifices (with level control at the inlet) that outflow into the 80m long culvert. This 80m long culvert has been modelled using a conduit unit. As one of the orifices was unstable in the unit mode transition, it was modelled using a sluice unit as an alternative. This sluice unit represent the head loss in a similar manner to the culvert unit.

Based on available information (refer to Section 4.4.6), informal effective defences have been included at two locations in the model. The defences consist of walls which run along both banks of the Mill Stream at Miller's Lane and further downstream, prior to discharging to the Irish Sea (refer to sample image opposite). The defences were surveyed as part of the DAS have been represented in the hydraulic model as ISIS HX lines with the elevation of defences obtained from the channel and structure cross sectional survey data. A map showing the location of the defences in the model is available in Appendix C3.

The impact of these defences on flood extents is discussed in the 'Without defences scenario' section. An analysis of flood risk and flood hazard due to sudden failure of these defences is reported on in Chapter 8.

<table>
<tr><td colspan="4"></td></tr>
<tr><td colspan="4">Floodplain model build</td></tr>
<tr><td>Extended cross sections</td><td>Reservoir units</td><td>Parallel river sections</td><td>2D domain</td></tr>
<tr><td>48</td><td>1</td><td>0</td><td>Skerries town downstream of the railway embankment (grid size 5m).</td></tr>
<tr><td colspan="4">In order to accurately model the flooding mechanisms in the 2D model domain, ISIS Z lines were included in the model to represent structures which obstruct and divert the flow of water in the floodplain. Directly downstream of the railway underpass, walls alongside the R127 at the junction of Dublin Road and Miller’s Lane divert flood water along Miller’s Lane. Further downstream, walls to the rear of properties at Sherlock Park and along Miller’s Lane divert flood water within the park in Skerries. These walls are not considered as informal flood defences as they act to divert flood water within the floodplain as apposed to holding back significant volumes of water. These walls have therefore not been assessed for the without defences scenario. Further details on the location of these ISIS Z lines within the 2D model domain of the Mill Stream model are in Appendix C3.</td></tr>
<tr><td colspan="4">Representative Manning’s n values</td></tr>
<tr><td colspan="4">Channel. Manning’s n varies between 0.03 and 0.04</td></tr>
<tr><td colspan="2">0.03: Minor stream, clean, straight, full stage, no rifts or deep pools (node 15Ma211)
</td><td colspan="2">0.04: Natural channel, clay, lateral slopes and bed with irregularities and pasture on slopes (node15Ma3166)
</td></tr>
<tr><td colspan="2">Floodplain. Manning’s n of 0.06</td><td colspan="2">Culverts. Colebrook-White friction of 0.002m</td></tr>
<tr><td colspan="2">0.06: dispersed bushes, weeds and few trees (node 15Ma2330)</td><td colspan="2">0.002: Concrete culvert, monolithic construction against rough forms (node 15Ma3145)</td></tr>
</table>


Boundary conditions
A tidal boundary at the mouth of stream was used as the downstream boundary unit for this model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event on Mill Stream is 18 hours. The same critical storm duration was used for the other AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.04	-0.05	0.11	-0.17
Tributary Maa	0.02	-0.02	0.07	-0.07
Tributary Mab	0.03	-0.04	0.05	-0.05
Tributary Mac	0.02	-0.02	0.04	-0.05
Tributary Mad	0.01	0.00	0.02	-0.02

Model inflows

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.09	-0.10	0.28	-0.25
Tributary Maa	0.06	-0.08	0.09	-0.14
Tributary Mab	0.04	-0.05	0.06	-0.06
Tributary Mac	0.06	-0.05	0.13	-0.10
Tributary Mad	0.01	-0.01	0.02	-0.02

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is +0.02m and -0.02m for increases and decreases in

Manning's n. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows with water levels along all of the watercourses increasing by 0.05m (flows +20%) and decreasing by 0.06m (flows -20%). The greatest impact caused by the changes in roughness and inflow values occurs on the main river channel upstream of culverts. Changes in roughness values result in a maximum change in water level upstream of the constriction at section 15Ma612. The most significant impacts on water levels as a result of changes to model inflows occur on the reach between sections 15Ma706 and 15Ma236 where the bed level has a very flat slope. The impacts on water levels in the tributaries are relatively minor.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition point is at model cross section 15Ma632 approximately. No tidal defence is present in the model.

The results indicate that the model is, on average, relatively insensitive to changes in both Manning's n and model inflows with notable increases and decreases in water levels occurring locally at structures with low conveyance capacity. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the river as expected.

'Without defences' scenario

Informal effective defences have been included at two locations in the model; Miller's Lane and Brookville Lane to the beach. These defences consist of walls which run along the banks of the river channel, which were removed for the 'without defences' model runs.

The flood extent map, MIL/HPW/EXT/CURS/001, indicates that these informal defences provide no protection against flooding to Skerries. This is because water levels in the river do not exceed the bank levels of the river channel along the defended river reach. The flooding that occurs in the area of the defences is as a result of flood water spilling into the area from further upstream. .

Model Results Summary

The following section provides a brief overview of the flood hazard along the Mill Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The flood maps for Skerries show that a large area of urban land is at risk of flooding from the Mill Stream. Flooding in Skerries is primarily as a result of the poor capacity of existing culverts along the Mill Stream, particularly the culverts under the railway at the junction of Dublin Road and Miller's Lane. The capacity of these culverts is insufficient to convey large flows and results in flood waters ponding on land to the west of the railway embankment and surcharging of the culverts. This surcharging results in spilling of flood waters along the R127, Miller's Lane and Sherlock Park. Flooding begins for the 4% AEP fluvial event at Miller's Lane. A significant number of properties along Miller's Lane and Sherlock Park are flooded for the 1% AEP event. At the downstream extent of the model, out of bank flooding results in flood risk to a number of properties at Holmpatrick Road. Refer to map MIL/HPW/EXT/CURS/001.

The average increase in water level for a 1% AEP fluvial event between the current scenario and MRFS is 0.07m and for the HEFS is 0.12m. The MRFS flood map indicates that there is a sizeable increase in flooding towards the downstream end of Mill Stream along the R128 road. This is also where the maximum difference in water levels occurs between the current scenario and MRFS (0.43m).and is primarily as a result of increases in the mean sea level associated with the MRFS.

Watercourse	*Average Water Level Difference*	*Maximum Water Level Difference*

	(m)		*(m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.15	0.29	0.43	0.97
Tributary Maa	0.07	0.10	0.10	0.13
Tributary Mab	0.04	0.07	0.07	0.10
Tributary Mac	0.07	0.10	0.14	0.18
Tributary Mad	0.02	0.02	0.03	0.04

5.14. Gaybrook Stream

Introduction	
Water bodies	Gaybrook Stream
APSRs	Swords area and Malahide & Portmarnock area

The Gaybrook Stream has its source in Nevinstown West; it flows in an easterly direction through Airside Retail Park, the Hollywell Estate and the Yellow Walls area in Malahide where it discharges into the Broadmeadow Estuary. The map above provides an overview of the extent of the Gaybrook Stream and its tributaries. Please refer to Figure 1for more details on the extent of the hydraulic model. The catchment drains a small area of 3.9km² and is broken down into eight sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The catchment is heavily urbanised. The main channel length is 5.1km and the two tributaries have a combined length of 2.8km. There are no gauging stations on this river. The tidal/fluvial dominance transition point of the Gaybrook Stream is located close to the estuary, as the downstream reach of this stream is very steep (based on the 1% AEP fluvial and tidal event).

Model Build

The Gaybrook Stream has been modelled together with the Broadmeadow estuary, as the tidal boundary conditions were calculated at an offshore location near the mouth of the estuary. The western side of the Broadmeadow estuary is partly controlled by the viaduct constriction (Dublin to Belfast railway line). Therefore, a 50% AEP fluvial base flow from the other rivers discharging into the western side of the Broadmeadow estuary, i.e. the Broadmeadow River and the Lissenhall Stream, were considered. However, the Lissenhall Stream's 50% AEP fluvial base flow was considered negligible compared to the Broadmeadow and Ward Rivers and was therefore not included in the model. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised catchment of the Gaybrook Stream.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	17	10 culverts/bridges on the main channel and 7 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR) and ORIFICE units.
Weir	0	
Gauging station	0	
Flood defences	0	
Other	0	

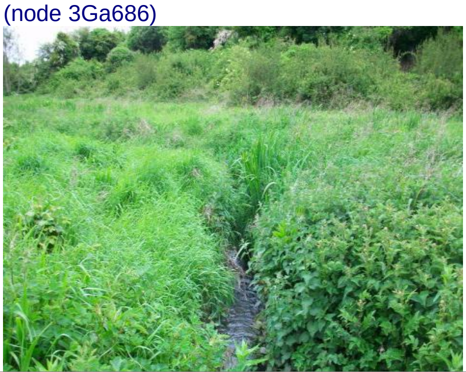
A 1.4km section of the Gaybrook Stream (north tributary) has been culverted under the M1 motorway. The details of this culvert are partly known and partly assumed as follows (refer to drawings in Appendix C5 for further details). To the north of the Hollywell Estate and. west of the M1 motorway, there is a double box culvert (2 parallel culverts of 1m x 1.6m). The culvert starts 80m upstream of the motorway at a manhole (reference CH0) before joining a single box culvert (2.3 x 1.25m) at manhole CH3 which is 200m downstream of the M1. A further 370m downstream, the culvert emerges at ground level.

On the main Gaybrook channel, upstream of the Holywell Estate, two online ponds outfall directly into a 1.05m diameter pipe. A drain, 0.3m in diameter, that bypasses the ponds also joins the 1.05m diameter pipe. It has been assumed that this 1.05m diameter pipe follows the line and slope of the original Gaybrook stream before it joins the M1 motorway's 1.2 m diameter collector pipe. The 1.2m pipe on the M1 also collects the road drainage from areas further west of the motorway before bending along Drynam Road and then joining the box culvert at the double/single junction (manhole CH3).

It is also assumed that the Gac tributary is culverted through a 0.9m diameter pipe and that this joins the double box culvert at CH0 where there is a 1.2 m diameter inlet.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
37	0	0	Nevinstown West, Drinan and Commons East (grid size 2m) and Yellow Walls and Coast (grid size 5m).

The entire coast line of Yellow Walls and Malahide has been modelled as a 2D domain and linked to the 1D estuary model using a 5m grid using LiDAR data for the topography. In Malahide, Bissets Strand Road crosses under the railway line through an underground tunnel. This tunnel is an important inland flow path at high tides between Malahide and Yellow Walls. As this feature is not represented in the LiDAR data, it was necessary to manually alter the DTM to represent this structure in the 2D domain. This was achieved by breaching the railway embankment at the location of the tunnel.

Representative Manning's *n* values	
Channel. Manning's n of 0.035	*Floodplain. Manning's n varies between 0.03 and 1.*
0.035: Minor stream, clean, straight, full stage, no rifts or deep pools with stones and weeds (node 3Ga686) 	0.06: Floodplain, dispersed bushes, weeds and few trees (node 3Ga474) 
Culverts. Colebrook-White friction varies between 0.002 and 0.006m	
0.002m: Concrete culvert, monolithic construction against rough forms (node 3Ga1037) 	0.006m: Old concrete culvert, monolithic construction against rough forms (node 3Ga4867) 

Boundary conditions

A tidal boundary at the mouth of the Broadmeadow estuary was used as the downstream boundary unit for the Gaybrook Stream model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP event for the Gaybrook Stream is 4 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.02	-0.03	0.12	-0.13
Tributary Gab	0.02	-0.03	0.06	-0.07
Tributary Gac	0.02	-0.02	0.12	-0.06

Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.15	-0.09	1.32	-0.51
Tributary Gab	0.03	-0.03	0.05	-0.05
Tributary Gac	0.09	-0.08	0.44	-0.17

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is +0.02m and -0.03m for increases and decreases in Manning's n. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the inflows with water levels along all of the watercourses increasing by 0.09m (flows +20%) and decreasing by 0.07m (flows -20%). The greatest impact caused by the changes in inflow values occurs on the main river channel and tributary Gac upstream of culverts. The most significant impact occurs upstream of the long M1 culvert (around chainage 3000, where the long culvert follows the M1).

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition starts to be significant upstream from the Old Yellow Walls Road bridge, around 3Ga417, on the main channel. As the model doesn't have tidal defences the differences in the water levels are +/-0.25m approximately in most of the affected area.

The results of the sensitivity analyses demonstrate that the model is not particularly sensitive to changes in roughness values with maximum differences in water levels of 0.13m at section 3Ga1426 near Yellow Walls. In terms of inflows, the results indicate that the model is, on average, relatively insensitive to changes in inflows with notable increases and decreases in water levels occurring locally at structures with low conveyance capacity. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the river as expected.

'Without defences' scenario
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Gaybrook Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

At the upstream extent of the modelled watercourse, west of the M1 motorway, the Gaybrook tributary floods in the southern part of the Airside Retail Park due to several surcharged culverts. Flooding starts at the 4% AEP fluvial design event with flood water flowing along the R125 before flowing back into the main channel's side drain near the long culvert inlet. Flooding of the Hollywell Estate occurs for the 0.5% and 0.1% AEP fluvial design events. On the northern part of the estate, at the Lake Shore Drive's culvert on the Gac tributary, flooding occurs for a 20% AEP fluvial design event or greater. The M1 motorway floods when the long culvert surcharges locally, for a 0.1% AEP fluvial design event. In Drinan, some properties on Aspen Drive and Aspen Park flood for a 1% AEP fluvial design event or greater. Refer to map GAY/HPW/EXT/CURS/001.

For the 0.5% AEP fluvial design event or greater, some of the flow runs from the Gaybrook Stream catchment into the Sluice River catchment through the Kettles Lane side drain. Refer to map GAY/HPW/EXT/CURS/001. Refer also to Section 5.16).

Towards the downstream extent of the modelled watercourse, there is limited flooding. There are no properties at risk of flooding for the fluvial design events within Yellow Walls, however for a 0.1% AEP tidal design event, properties are at risk of flooding at Ide Drive including a school. Refer to map GAY/HPW/EXT/CURS/T/002**.**

For further discussion on flooding around the Broadmeadow estuary, please refer to Section 5.2.1.

The average water level increase between the current scenario and the MRFS for a 1% AEP fluvial event is 0.28m and for the HEFS it is 0.37m. The maximum difference between the current scenario and the MRFS is 2.19m. The large differences in water levels between the current scenario and MRFS are as a result of surcharging of the culvert during high flows. This surcharging of the culvert forces water into the culvert invert slot which is used to improve model stability (please refer to Section 4.4.8). The levels reported on in the table below compare the pressurised water levels in the culvert inlet slot (for the MRFS and HEFS) which are significantly greater than the current scenario water levels as there is minimal surcharging of this culvert for this scenario.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.50	0.62	2.19	2.35
Tributary Gab	0.07	0.10	0.14	0.21
Tributary Gac	0.27	0.39	0.84	1.76

5.15. Mayne River

Introduction	
Water bodies	Mayne River and Cuckoo Stream
APSRs	Dublin airport, Belcamp & Balgriffin area,

The Mayne River has its source near Dublin airport; it flows in an easterly direction until it reaches the Baldoyle estuary. The map above provides an overview of the extent of the Mayne River and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model and elements of the hydraulic model build (e.g. 2D model domains). The catchment drains an area of 19.91km² and is broken down into thirteen sub-catchments (refer to FEM FRAMS Hydrology Report, 2010). The main channel length is 8.4km and has four tributaries that have a combined length of 8.6km. There are no gauging stations on this river. The Mayne River has been modelled together with the Baldoyle estuary. The tidal/fluvial dominance transition point in the river channel is almost indistinguishable due to the presence of a flapped outfall at the mouth of the river where it discharges to the estuary (based on the 1% AEP fluvial and tidal event).

Model Build

The Mayne River has been modelled together with the Baldoyle estuary, as the tidal boundary conditions were calculated at an offshore location at the mouth of the estuary. The 50% AEP fluvial base flow of the Sluice River (into the estuary) was not included in the model as the contribution of these river flows compared to the volume of the tidal flood waters is considered negligible. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised catchment of the Mayne River.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	47	20 culverts/bridges on the main channel and 27 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR), ORIFICE and VERTICAL SLUICE units.
Weir	6	6 weirs on the tributaries. Each structure was represented by a SPILL unit due to its irregular shape.
Gauging station	0	
Flood defences	1	Tidal sluice represented by a flapped ORIFICE unit

Type	*Number*	*Summary*
Other	3	Flow Control Diversion chamber represented by 2 ORIFICE units and one SLUICE unit. Footbridge represented by a BERNOULLI LOSS unit. Sediment Control System represented by 3 SYPHON units.

An assessment of culvert blockages was undertaken for the following culverts: Mayne River at the Swords Road and the Cuckoo Stream (Mac tributary of the Mayne River) at Wellfield Bridge. At Swords Road the river passes through a long culvert under the R132 (node 1Ma7268) while at Wellfield Bridge (node 1Mac258) the culvert is 119m long. Further details on the culvert blockage assessment are in Sections 9.2.1~~9.2.19.1.1~~ and 9.2.2~~9.2.29.1.2~~.

The flapped outfalls (image opposite) at the downstream end of the model are considered a formal flood defences. The flapped outfalls prevent the high tides from propagating upstream at any % AEP event. Operating rules within the ISIS model unit set the gate to open or close based on the upstream and downstream water level (e.g. the gate is set to close when the downstream tidal water level is higher than the upstream fluvial water level). The impact of this defence on flood extents is discussed in the 'without defences scenario' section.

The Dublin Airport Drainage and Pollution System Control structure was included in the model using details provided by the DAA (refer to Appendix C5 for more details). The attenuation tank (50 separate parallel culverts) located on the Cuckoo Stream was modelled as one big culvert with an equivalent area. The roughness of the culvert was assumed to have a Colebrook-White friction of 0.002m. This Colebrook-White friction factor was not modified for the purposes of calibrating the attenuation in the tank as no data was available to calibrate this structure.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
154 (without Baldoyle estuary)	2 (There is a third reservoir that represents the pollution storage area).	0	N32 road from M1 highway to Darndale Park. Between Balgriffin to Baldoyle estuary (grid size 5m).

The pollution storage area, which is part of the Dublin Airport Drainage and Pollution System Control, was modelled as a reservoir unit.

Representative Manning's *n* values

Channel. Manning's n varies between 0.030 and 0.035

0.03: Minor stream, clean, straight, full stage, no rifts or deep pools (node 1Ma6912)

0.035: Minor stream, clean, straight, full stage, no rifts or deep pools with more stones and weeds (node 1Ma3367)

Floodplain. Manning's n of 0.06

0.06: Dispersed bushes, weeds and few trees (node 1Ma5029)

Culverts. Colebrook-White friction varies between 0.002m and 0.03m

0.03m: Corrugated metal culvert (node 1Ma6653)

Boundary conditions

A tidal boundary at the mouth of the Baldoyle estuary was used as the downstream boundary unit for the Mayne River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for 1% AEP event on the Mayne River is 5.5 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.04	-0.05	0.15	-0.28
Tributary Maa	0.01	-0.02	0.01	-0.02
Tributary Mab	0.05	-0.05	0.09	-0.09
Tributary Mac	0.07	-0.08	0.19	-0.33
Tributary Mae	0.02	-0.02	0.05	-0.06
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.08	-0.22	0.52	-0.90
Tributary Maa	0.13	-0.11	0.13	-0.11
Tributary Mab	0.08	-0.08	0.22	-0.20
Tributary Mac	0.10	-0.14	0.61	-0.50
Tributary Mae	0.03	-0.03	0.05	-0.05

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is +0.04m and -0.04m for increases and decreases in Manning's n. Changes to model inflows results in an average increase and decrease in water levels along all watercourses of +0.08m and -0.12m. The main river channel and Mac tributary near Balgriffin experience the most significant impacts on water levels as a result of changes in roughness values. Changes in inflows results in a more noticeable impact on average water levels with the biggest differences downstream of the M1 motorway near Clonshaugh in the main river channel and upstream of the culvert through the M1 motorway on the Mac tributary.

The +/-0.25m tidal sensitivity analysis was carried out for the 0.5% tidal event. The tidal/fluvial dominance transition point (approximately 700m upstream of the coastline) remains almost invariable. The differences in water levels in that area are +0.08m and -0.09m respectively in comparison with the current scenario due to the influence of the flapped outfall (i.e. increases in water level are due to river flows being unable to discharge to the sea when the flap is closed rather than directly responding to sea level rise).

The results indicate that the model is, on average, relatively insensitive to changes in Manning's n and more sensitive to changes in model inflows. Notable increases and decreases in water levels occur locally at structures with low conveyance capacity. As the water levels in the river are controlled by a flapped outfall, the river channel is sensitive to changes in downstream tide levels.

'Without defences' scenario

The Mayne river has a flapped outfall that acts as a defence against tidal events. The flapped gate was removed in the 'without defences' model to determine the areas benefiting from this defence. The tidal flood extent map (MAY/HPW/EXT/CURS/T/003) indicates that flooding at Maynestown and Stapolin is increased without the tidal defence but no urban areas are affected. The fluvial flood extent map (MAY/HPW/EXT/CURS/003) indicates that the flapped outfall has no affect on the fluvial flood extents.

The difference between the defended areas on the tidal and fluvial maps is as a result of the JPA tide and river flow combinations (refer to Section 4.4.4 for further details on the JPA combinations). For the fluvial scenarios, the fluvial component is more dominant than the tidal component. In this scenario, the high tides prevent the discharge of the river flows to the Baldoyle Estuary resulting in flooding upstream of the flapped outfall. For the tidal events, the tidal component is more dominant. The fluvial flows for the tidally dominant scenario, although lower than the fluvially dominant scenario, are large and result in flooding upstream of the outfall where high tides prevent flows discharging to the estuary. Without the flap valve in place (i.e. undefended), the maps show that a 0.1% AEP tidal water level with 2% AEP flow results in additional flooding when compared to flood extends for the defended scenario.

The attenuation tank and pollution storage areas at Dublin Airport are not considered as formal defences and have not been removed in the 'without defences' model.

Model Results Summary

The following section provides a brief overview of the flood hazard along the Mayne River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

Towards the upstream extent of the modelled watercourse, the Swords Road (R132) is flooded at two different locations: when it crosses the Cuckoo Stream (flooding starts for the 4% AEP fluvial event) and when it crosses the main river channel (flooding starts for the 1% AEP fluvial event). The culvert located at section 1Ma6020 causes flooding for a 4% AEP fluvial event or greater. For a 0.1% AEP fluvial event this flooding extends to onto the N32 near Bewleys Hotel and floods the Belcamp urban area on the right bank. Refer to map MAY/HPW/EXT/CURS/001.

The Mac tributary, upstream of Balgriffin Road (R123), starts to flood on right bank for a 2% AEP fluvial event. This flooding spills over the R123 and flows into the housing development located downstream of the R123. At the downstream extent of the model there is sizeable area of undeveloped lands flooded in Snugborough both upstream and downstream of the railway line Refer to map MAY/HPW/EXT/CURS/003

The average water level increase between the current scenario and the MRFS for a 1% AEP fluvial event is 0.21m and for the HEFS it is 0.39m. The maximum difference between the current scenario and the MRFS is 1.07m and is located just upstream of a dual culvert at section 1Ma4685. This is due to a combination of the constraint on flows caused by the dimensions of the structure and a high spill level (i.e. the level at which the flow will overtop the structure).

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.17	0.27	1.07	1.35
Tributary Maa	0.39	0.75	0.39	0.75
Tributary Mab	0.22	0.50	0.64	1.56
Tributary Mac	0.22	0.30	0.95	1.05
Tributary Mae	0.09	0.12	0.13	0.19

5.16. Sluice River

Introduction

Water bodies	Sluice River
APSRs	Portmarnock and Malahide area, Kinsaley Lane area, Ballynacartle area and Dublin airport, Belcamp & Balgriffin area.

The Sluice River has its source near Barberstown and Pickardstown to the north of Dublin Airport and flows in a south-easterly direction until it discharges to the Baldoyle Estuary at Portmarnock Bridge. The map above provides an overview of the extent of the Sluice River and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model and elements of the hydraulic model build (e.g. 2D model domains). The catchment drains an area of 21.78km² and is broken down into 16 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 11.7km and has three tributaries with a combined length of 9.5km. The Kinsaley Hall gauging station is located downstream of the bridge at node 2Sa3017. The Sluice River has been modelled together with the Baldoyle estuary. The tidal/fluvial dominance transition point in the river channel is almost indistinguishable due to the presence of a flapped outfall at Portmarnock Bridge (based on the 1% AEP fluvial and tidal event).

Model Build

The Sluice River has been modelled together with the Baldoyle estuary, as the tidal boundary conditions were calculated at an offshore location at the mouth of the estuary. The 50% AEP fluvial baseflow of the Mayne River was not included in the model as the contribution of the river flow compared to the volume of the tidal flood waters is considered negligible. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the urbanised catchment of the Sluice River.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	79	44 culverts/bridges on the main channel and 35 on the tributaries. Structures modelled using BRIDGE (ARCH and USBPR), ORIFICE and VERTICAL SLUICE units.

Type	*Number*	*Summary*
Weir	12	4 weirs on the main channel and 8 on the tributaries. Each structure was represented by a SPILL unit due to its irregular shape.
Gauging station	1	Kinsaley Hall gauging station.
Flood defences	1	Tidal sluice represented by a Flapped ORIFICE unit.
Other	4	Golf pond outlet represented by a SLUICE unit. Trash screen located in the middle of the river represented by a SPILL unit.

An assessment of culvert blockages was undertaken for the following culvert; Sluice River at Portmarnock Trotting Track where the Sluice River passes through a 39m long culvert (near node 2Sa2300). Further details on the culvert blockage assessment are in Section 9.2.3~~9.2.39.1.3~~.

The flapped outfall at the downstream end of the model (Portmarnock Bridge – refer to image opposite) is considered a formal flood defence. The flapped outfall prevents the high tides from propagating upstream for any AEP event. Operating rules within the ISIS model unit set the gate to open or close based on the upstream and downstream water level (e.g. the gate is set to close when the downstream tidal water level is higher than the upstream fluvial water level). The impact of this defence on flood extents is discussed in the 'without defences scenario' section.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
255 (including Baldoyle estuary)	4 (There is a fifth 'online' reservoir).	0	Dublin Airport northern area; Between M1 highway and R108 road and from Streamstown to Baldoyle Estuary (grid size 5m).

<table>
<tr><td colspan="2">Representative Manning's *n* values</td></tr>
<tr><td colspan="2">*Channel. Manning's n varies between 0.030 and 0.035 (reduced to 0.024 and 0.028 to calibrate the model)*</td></tr>
<tr><td>0.03: Minor stream, clean, straight, full stage, no rifts or deep pools. (2Sa3386)
</td><td>0.035: Minor stream, clean, straight, full stage, no rifts or deep pools with stones and weeds. (2Sa9961)
</td></tr>
<tr><td>*Floodplain. Manning's n of 0.06*</td><td>*Culverts. Colebrook-White friction varies between 0.002m and 0.025m*</td></tr>
<tr><td>Floodplain, dispersed bushes, weeds and few trees. (2Sa8103)
</td><td>0.002m: Concrete culvert, monolithic construction against rough forms. (2Saaa511)
</td></tr>
<tr><td colspan="2">Boundary conditions</td></tr>
<tr><td colspan="2">There is an open quarry located in the upstream catchment of the 2Saa tributary which encompasses up to 50% of the SLU_02_B1 sub-catchment. The operation of the surface water pumps in this area was discussed with the operators of the quarry who confirmed that they only operate the pumps when necessary and not during a storm event. It has therefore been assumed that the inflows for this subcatchment should be based on using only 50% of the total subcatchment area. This assumption concurs with the observed flood extent in this area by Local Authority Engineers.

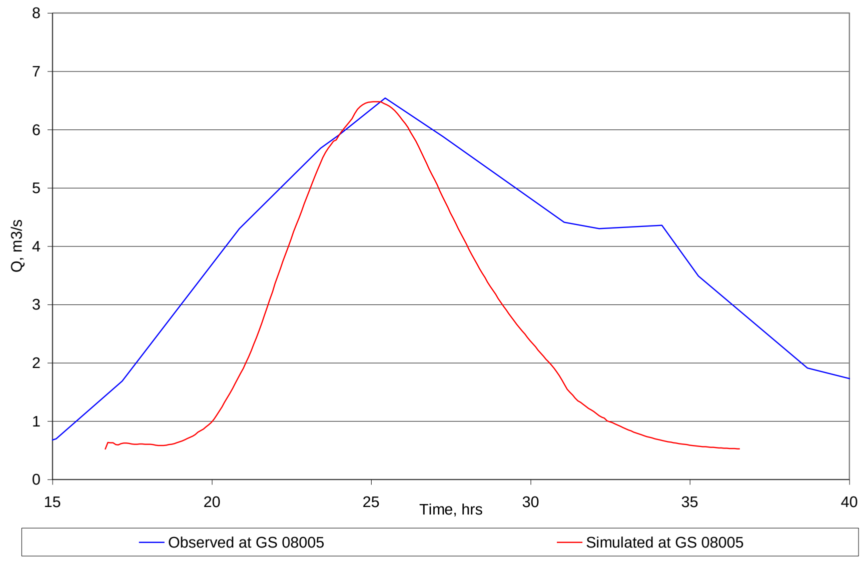
At low frequency AEP events (e.g. 0.1% AEP) flow from the upstream part of the Gaybrook Stream overspills into the Sluice River catchment. This required some minor modification to the original Sluice River hydraulic model by adding point inflows to the reservoir unit at node Sa7136LSD.

A tidal boundary at the mouth of the Baldoyle estuary was used as the downstream boundary unit for the Sluice River model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.</td></tr>
</table>

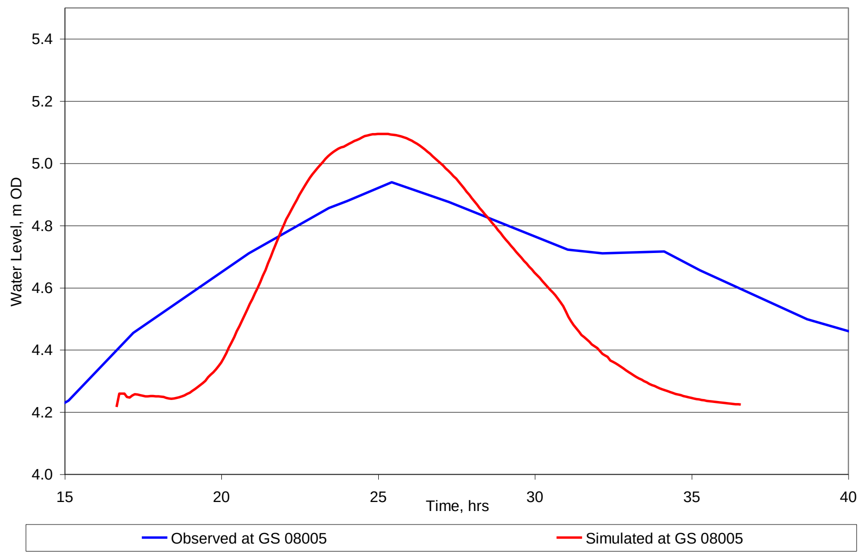
Model Calibration

The June 1986 historic flood event was used for the calibration of the model. As far as we know, there was no significant flood defence/construction works carried out along the river between the events and the time of the topographical survey. Therefore the design model was used to calibrate the event.

June 1986



Comparison between the observed June 1986 flow hydrograph (blue) and the calibrated model flow hydrograph (red) at Sluice gauging station.



Comparison between the observed June 1986 stage (blue) and calibrated model stage (red) at Sluice gauging station.

There is good agreement between the observed and modelled peak flow values and acceptable agreement (~0.2m) between the observed and modelled peak level values. Any modification in the river channel and increased urbanisation u/s of the Kinsaley Hall GS (GS 08005 Sluice River) post 1986 could result in a peakier model produced hydrograph than the August 1986 observed hydrograph. As good calibration was achieved between observed and modelled peak flows and levels the model is considered to be performing appropriately.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event on Sluice River is 12 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.05	-0.05	0.16	-0.16
Tributary Saa	0.04	-0.03	0.11	-0.10
Tributary Sab	0.06	-0.06	0.13	-0.14
Tributary Sac	0.03	-0.03	0.06	-0.08

Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.09	-0.12	0.29	-0.49
Tributary Saa	0.09	-0.07	0.25	-0.15
Tributary Sab	0.06	-0.07	0.13	-0.14
Tributary Sac	0.04	-0.04	0.18	-0.14

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. The average increase and decrease in water levels along all watercourses is +0.05m and -0.04m for increases and decreases in Manning's n. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the inflows with water levels along all of the watercourses increasing by 0.08m (flows +20%) and decreasing by 0.13m (flows -20%). The greatest impact on water levels as a result of changes in roughness and inflow occurs on the main river channel. Changes in water levels resulting from changes in Manning's n values never exceed 0.20m. The impact of changes in model inflows is greater with the maximum change in water level occurring in the reach between sections 2Sa10154 and 2Sa9675, upstream of the structure located at section 2Sa7234 and near the culverts through the Portmarnock trotting track. The highest differences in the tributary Saa occur at cross section 2Saaz1030.

The +/-0.25m tidal sensitivity analysis was carried out for the 0.5% tidal event. The tidal/fluvial dominance transition point is located at cross section 2Sa2300 (approximately). The differences in water levels in that area are +0.15m and -0.06m respectively in comparison with the current scenario due to the influence of the flapped outfall (i.e. increases in water level are due to river flows being unable to discharge to the sea when the flap is closed rather than directly responding to sea level rise).

The results indicate that the model is, on average, relatively insensitive to changes in Manning's n and more sensitive to changes in model inflows. The significant increases and decreases in water levels occur locally at structures with low conveyance capacity. As the water levels in the river are controlled by a flapped outfall, the river channel is sensitive to changes in downstream tide levels.

'Without defences' scenario
The Sluice River has a flapped outfall that acts as a defence against tidal events. The flapped gate was removed in the 'without defences' model to determine the areas which benefit from this defence. The tidal flood extent map (SLU/HPW/EXT/CURS/T/004) shows that the flapped outfall provides protection to a significant area of land downstream of the railway embankment for the 0.1% AEP event. Both the Beechwood golf course and lands near the racecourse are affected by the removal of this defence. The fluvial flood extent map (SLU/HPW/EXT/CURS/004) shows that the flapped outfall has a minimal impact on flood extents with a small area of land east of the railway embankment, near the racecourse, protected by the flapped outfall.

The difference between the defended areas on the tidal and fluvial maps is as a result of the JPA tide and river flow combinations (refer to Section 4.4.4 for further details on the JPA combinations). For the fluvial scenarios, the fluvial component is more dominant than the tidal component. In this scenario, the high tides prevent the discharge of the river flows to the Baldoyle Estuary resulting in flooding upstream of the flapped outfall. For the tidal events, the tidal component is more dominant. The fluvial flows for the tidally dominant scenario, although lower than the fluvially dominant scenario, are large and result in flooding upstream of the outfall where high tides prevent flows discharging to the estuary. Without the flapped outfall in place (i.e. undefended), a 0.1% AEP tide with 2% AEP flow results in flooding of the golf course at Beechmount. For a 1% AEP and 0.1% AEP flow and 2% AEP and 20% AEP tide respectively, the flap valve does not prevent flooding of this area.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Sluice River. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

Towards the upstream extent of the modelled watercourse, some of the flow from the upstream part of the Gaybrook Stream overspills into the Sluice River catchment at low AEP events (e.g. 0.1% AEP). Refer to map SLU/HPW/EXT/CURS/002 and Section 5.14 for further details.

At Streamstown, the capacity of the culverts results in flooding which starts at the 4% AEP event. The flooding overtops the Malahide Road and bypasses two culverts before going back to the river downstream of Streamstown. The flooding results in flood risk to a number of properties in Streamstown. Refer to map SLU/HPW/EXT/CURS/003.

Further downstream, flooding of the Portmarnock trotting track occurs for the 0.1% AEP fluvial event and flooding of the Beechwood golf course occurs for the 10% AEP fluvial and tidal events. At Strand Road (R106) in the Baldoyle Estuary Natural Reserve out of bank flooding starts at the 10% AEP tidal event resulting in flood risk to a number of properties at Strand Road. Refer to maps SLU/HPW/EXT/CURS/004 andSLU/HPW/EXT/CURS/T/004.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.15m and for the HEFS is 0.21m. The maximum difference between the current scenario and MRFS is 0.49m and is located just upstream of a culvert at section 2Sa9885. Along the fluvial reaches of the watercourse there is a marginal increase in flood extents associated with the MRFS. Towards the downstream extent of the modelled watercourse, there is a more obvious increase in flood extents which is associated with the increase in mean sea levels.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.17	0.32	0.49	0.77
Tributary Saa	0.14	0.22	0.36	0.61
Tributary Sab	0.09	0.11	0.19	0.22
Tributary Sac	0.08	0.20	0.39	1.02

5.17. St. Catherine's Stream

Introduction	
Water bodies	St. Catherine's Stream
APSRs	Rush area

St. Catherine's Stream has its source near Balcunnin on the R127. It flows in an easterly direction until it reaches the Irish Sea. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the hydraulic model. The catchment drains an area of 4.25km² and is broken down into 4 sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The channel length is 2.4km and it has no tributaries. There are no gauging stations on this river. The model is entirely fluvially dominated due to the steepness of the channel (2% on the downstream half of the modelled reach). Also, the invert level of the last section of the model is higher than the most extreme tide level (based on the 0.1% AEP tidal event).

Model Build

A 1D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the catchment of St. Catherine's Stream. No 2D modelling was undertaken as the river passes mainly through rural areas and its hydraulic behaviour can be accurately modelled using 1D modelling techniques.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	6	The bridges were modelled using BRIDGE (ARCH) and ORIFICE units.
Weir	13	11 weirs were represented by SPILL units due to their irregular shape. 2 weirs were represented by round weir units.
Gauging station	0	
Flood defences	0	
Other	0	

<table>
<tr><td colspan="4">Floodplain model build</td></tr>
<tr><td>Extended cross sections</td><td>Reservoir units</td><td>Parallel river sections</td><td>2D domain</td></tr>
<tr><td>0</td><td>0</td><td>0</td><td>1 domain covers the entire model (grid size 5m).</td></tr>
<tr><td colspan="4">Representative Manning's n values</td></tr>
<tr><td colspan="2">Channel. Manning's n of 0.04</td><td colspan="2">Floodplain. Manning's n varies between 0.03 and 1.0</td></tr>
<tr><td colspan="2">0.04: Minor stream, clean, straight, full stage, no rifts or deep pools with more stones and weeds (node 13Na1410)
</td><td colspan="2">0.06: dispersed bushes, weeds and few trees (node 13Na1448)
</td></tr>
<tr><td colspan="2">Culverts. Colebrook-White friction 0.006m</td><td colspan="2"></td></tr>
<tr><td colspan="2">0.006: Concrete culvert, monolithic construction against rough forms (node 13Na834)
</td><td colspan="2"></td></tr>
<tr><td colspan="4">Boundary conditions</td></tr>
<tr><td colspan="4">A normal head downstream boundary was used as the downstream boundary unit for this model. This is because the invert level of the last cross section of the model is higher than the highest tide event. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.</td></tr>
<tr><td colspan="4">Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the St. Catherine's Stream is 21 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, and model inflows.</td></tr>
</table>

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.03	-0.04	0.10	-0.10
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.06	-0.06	0.12	-0.12

The results of the sensitivity analyses show that the model is not particularly sensitive to changes in roughness or inflow values. The biggest impact resulting from changes to Manning's n values occurs in the upstream part of the modelled reach, just downstream of the structure at section 13Na1950 The maximum water level difference resulting from changes to inflow values occurs just upstream of the St Catherine's Estate access road bridge at section 13Na834U.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event but results in no change in water levels as the most downstream section of the model is above the maximum tide level.

'Without defences' scenario
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along St. Catherine's Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

There is limited flooding along St. Catherine's Stream with only a small pocket of localised flooding at node 13Na568. Refer to map CAT/HPW/EXT/CURS/001.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.08m and for the HEFS is 0.13m. The flood maps indicate that there is no increase in flood risk associated with the MRFS.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.08	0.13	0.35	1.00

5.18. Baleally Stream

Introduction	
Water bodies	Baleally Stream
APSR	Lusk area

The Baleally Stream has its source near Lusk town; it flows in a southerly direction until it reaches the Rogerstown estuary. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the hydraulic model. The catchment drains a small area of 4.2km² and is broken down into six sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 4.4km and has no tributaries. There are no gauging stations on this river. The tidal/fluvial dominance transition point is at model cross section 9Ba655 based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the catchment of the Baleally Stream.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	10	Structures modelled using BRIDGE (ARCH) and ORIFICE units.
Weir	1	Each structure was represented by a SPILL unit due to its irregular shape.
Gauging station	0	
Flood defences	0	
Other	0	

An assessment of culvert blockages was undertaken at one location on the Baleally Stream: a 316m long circular corrugated metal culvert at node 9Ba3030. Further details on the culvert blockage assessment are in Section 0.

Floodplain model build

Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
0	0	0	The entire model (grid size 5m).

Representative Manning's *n* values

Channel. Manning's n varies between 0.03 and 0.05

0.03: Minor stream, clean, straight, full stage, no rifts or deep pools (node 9Ba323)

0.05: Natural channel, clay, lateral slopes and bed with irregularities, scrub and bushes on lateral slopes (node 9Ba3981)

Floodplain. Manning's n varies between 0.06 and 0.1

0.06: Floodplain, dispersed bushes, weeds and few trees (node 9Ba274)

Culverts. Colebrook-White friction varies between 0.0006m and 0.045m

0.045m: Corrugated metal culvert (node 9Ba9030)

Boundary conditions

A tidal boundary extracted from the Rogerstown estuary within the Ballyboghil and Corduff River model was used as the downstream boundary unit for this model. Please refer to Section 5.10.1 for more details on the modelling of the Rogerstown Estuary. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for 1% AEP event on Baleally Stream is 6.5 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.03	-0.03	0.12	-0.11

Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.06	-0.07	0.21	-0.22

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the inflows. The results show that the model is more sensitive to changes in flows than the changes in roughness. The most significant impact on water levels as a result if changing the inflows occurs upstream of the structure at section 9Ba1801.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition point is at model cross section 9Ba949. As the model doesn't have tidal defences the differences in the water levels are +/-0.25m approximately in most of the affected area.

The results indicate that the model is, on average, relatively insensitive to changes in Manning's n and model inflows. The significant increases and decreases in water levels occur locally at structures with low conveyance capacity. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the river as expected.

'Without defences' scenario
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along the Baleally Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

There is limited flooding along the Baleally Stream. The river passes through the urban area of Lusk before it flows into the Rogerstown estuary. Within the urban area there are two long culverts with an open channel section in the middle as follows:

- 9Ba3905 to 9Ba3566, 340m culvert
- 9Ba3566 to 9Ba3030, 536m open channel

- 9Ba3030 to 9Ba2714, 315m culvert

Although the culverts are surcharged for a flood event, the capacity of the open channel section fully contains all of the current scenario flood events and no flooding occurs (refer to map BAY/HPW/EXT/CURS/001). For discussion on flooding around the Rogerstown estuary please refer to Section 5.10.1.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.23m and for the HEFS is 0.43m. The maximum difference between the current scenario and the MRFS is 0.68m and is located just upstream of a long culvert between sections 9Ba3030 to 9Ba2714 The increase in levels results in an increased flood risk in Lusk with the flood maps indicating flooding in Lusk for the 0.1% AEP MRFS event.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.23	0.43	0.68	1.06

5.19. Bride’s and Jone’s Streams

5.19.1. Bride’s Stream

Introduction	
Water bodies	Bride’s Stream
APSR	Lusk area and Rush area

The Bride’s Stream has its source at Ballymaguire (North of Lusk). It flows in a south-easterly direction where it discharges to the Rogerstown estuary near Rush. Before discharging to the Rogerstown Estuary, the river is joined by a large tributary, Jone’s Stream (near node 10La500 at the R128). The map above provides an overview of the extent of the Bride’s Stream and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model. The catchment drains an area of 4.9km² and is broken down into four sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). There are no gauging stations along this water course. The tidal/fluvial dominance transition point is at model cross section 10La495D based on the 1% AEP fluvial and tidal event.

Model Build

The Bride’s Stream forms part of the larger Bride’s and Jone’s Stream model. The two rivers were modelled as one river model to ensure that any interaction in flood flows between the rivers is accurately captured. This section provides details of the Bride’s Stream element of the model. Information on the Jone’s Stream is detailed in Section 5.19.2. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the this catchment.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	17	17 culverts/bridges on the main channel. The structures were modelled using BRIDGE (ARCH), ORIFICE, VERTICAL SLUICE and CONDUIT units as appropriate.
Weir	1	1 weir on the main channel. The structure was represented by a SPILL unit due to its irregular shape.
Gauging station	0	
Flood defences	0	
Other	4	3 sudden bed level drops caused by stones represented by SPILL units. 1 river narrowing (wall) represented by a SPILL unit.

At the outfall of the watercourse, there is an arch bridge with a tidal flap gate in place within the bridge arch. In order to model this scenario realistically, two orifices have been used, one to represent the small flap gate within the bridge arch and the second to represent the gap between the horizontal top of the flap gate and the opening for the arch bridge near the soffit (refer to image opposite). In addition, a spill unit has been used to represent the flow over and around the top of the bridge. This flapped outfall is not considered as a flood defence, with further discussion later in this section of the report in the 'without defences scenario' section.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
-	-	-	The floodplain was modelled using 2D approach for the whole area (grid size 5m).

Representative Manning's *n* values	
Channel. Manning's n varies between 0.030 and 0.045	
0.030: Open channel, clean, straight, full stage, no rifts or deep pools (node 10La142)	0.045: Minor stream, clean, winding, some pools and shoals with some weeds and stones (node 10La1650)

Floodplain. Manning's n varies between 0.06 and 0.1	*Culverts. Colebrook-White friction of 0.006m*
0.06: Floodplain, dispersed bushes, weeds and few trees 	0.006: Concrete culvert, monolithic construction against steel forms, normal (node 10La495) 
Boundary conditions	
A tidal boundary extracted from the Rogerstown estuary within the Ballyboghil and Corduff River model was used as the downstream boundary unit for this model. Please refer to Section 5.10.1 for more details on the modelling of the Rogerstown Estuary. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.	

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the Bride's Stream is 15 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

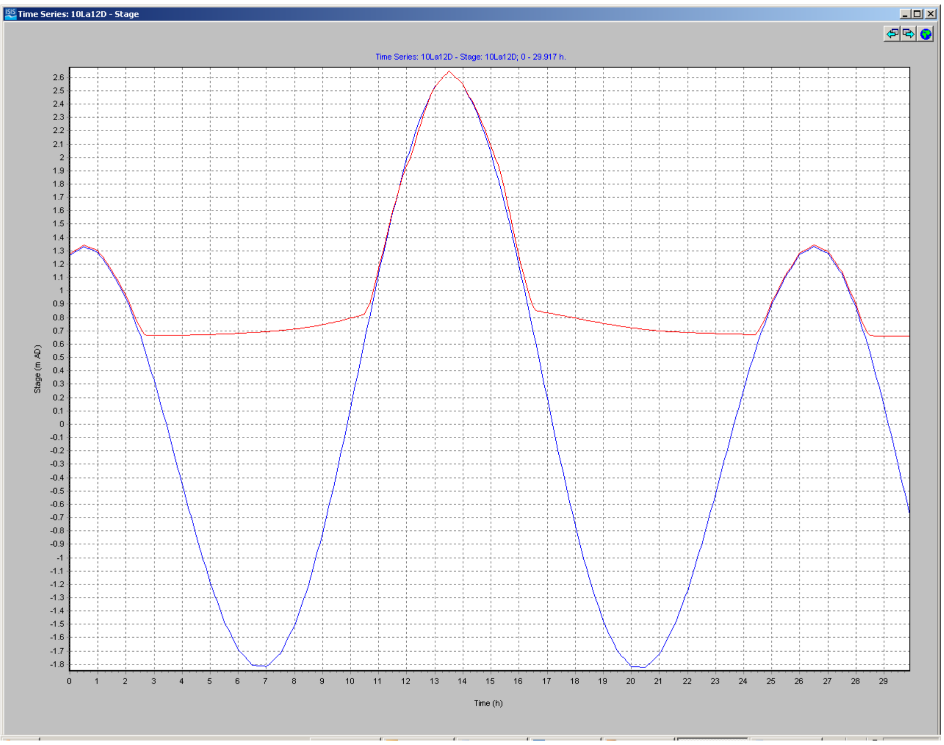
Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.06	-0.06	0.11	-0.12
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.08	-0.09	0.26	-0.22

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when the inflows are changed. The results indicate that the model is more sensitive to variations in flow than to roughness variations. The greatest impact resulting from changes in flow occurs upstream of the structure at section 10La3409.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the tidal/fluvial dominance transition point is at model cross section 10La495D. As the model doesn't have effective tidal defences the differences in the water levels are +/-0.25m approximately in mostly of the affected area.

The results indicate that the model is, on average, relatively insensitive to changes in Manning's n and model inflows. Notable increases and decreases in water levels occur locally at structures with low conveyance capacity. As the model doesn't have a tidal defence which form a tidal defence function, sensitivity to tidal levels is along the tidal reaches of the river as expected.

'Without defences' scenario
As described in the 'Summary of structures in the model', the outfall has a flapped gate with an opening above the top of the gate. The figure below shows that at cross section 10La26, the water level (red line) and the tide level (blue line) are the same for high tide levels. This demonstrates that the flapped gate does not work as a tidal defence and therefore no 'without defences' model runs have been undertaken for the Bride's Stream.



Model Results Summary
The following section provides a brief overview of the flood hazard along Bride's Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The flood maps indicate that there is a limited extent of flooding along the Bride's Stream. For a 2% AEP fluvial event, the culvert at section 10La811, just upstream of the junction with the Jone's Stream, is overtopped resulting in flood risk to a small number of properties.

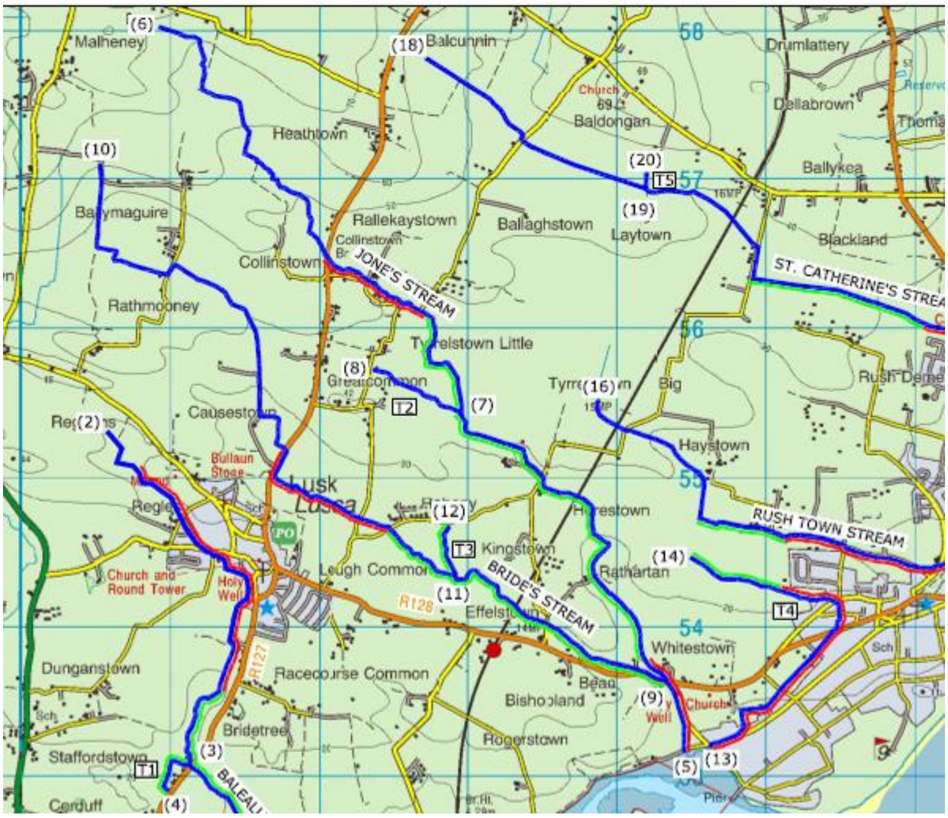
Further downstream near the Rogerstown estuary, Spout Road is overtopped for the 4% AEP event resulting in flood risk to a small number of properties. Refer to map BRI/HPW/EXT/CURS/001 and BRI/HPW/EXT/CURS/T/001. For discussion on flooding around the Rogerstown estuary please refer to Section 5.10.1.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.14m and for the HEFS is 0.29m. The maximum difference between the current scenario and the MRFS is 0.36m and is located just upstream of a structure with a low conveyance capacity at section 10La3409. The MRFS maps indicate that the largest increase in flood risk is at the downstream extent of the model as a result of the increase in mean sea levels associated with the MRFS.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.14	0.29	0.36	1.00

5.19.2. Jone's Stream

Introduction	
Water bodies	Jone's Stream
APSRs	Rush area



The Jone's Stream has its source at Malheney (North of Lusk); it flows in a south-easterly direction until it joins the Bride's Stream near node 10La500 at the R128. The map above provides an overview of the extent of the Jone's Stream and its tributaries. Please refer to Figure 1 for more details on the extent of the hydraulic model. The catchment drains an area of 5.71km² and is broken down into four sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The main channel length is 3.5km long. There are no gauging stations along this watercourse. It is a fluvially-dominated watercourse to its confluence with the Bride's Stream based on the 1% AEP fluvial and tidal event.

Model Build

The Jone's Stream forms part of the larger Bride's and Jone's Stream model. The two rivers were modelled as one hydraulic model to ensure that any interaction in flood flows between the rivers is accurately modelled. This section provides details of the Jone's Stream element of the model. Information on the Bride's Stream is detailed in Section 5.19.2. A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through this catchment.

<table>
<tr><th colspan="4">Summary of structures in the model</th></tr>
<tr><th>Type</th><th>Number</th><th colspan="2">Summary</th></tr>
<tr><td>Culvert/Bridge</td><td>8</td><td colspan="2">8 bridges on the main channel. The structures were modelled using BRIDGE (ARCH), ORIFICE and VERTICAL SLUICE units as appropriate.</td></tr>
<tr><td>Weir</td><td>0</td><td colspan="2"></td></tr>
<tr><td>Gauging station</td><td>0</td><td colspan="2"></td></tr>
<tr><td>Flood defences</td><td>0</td><td colspan="2"></td></tr>
<tr><td>Other</td><td>6</td><td colspan="2">6 sudden drops in bed level caused by stones represented by SPILL units.</td></tr>
<tr><th colspan="4">Floodplain model build</th></tr>
<tr><th>Extended cross sections</th><th>Reservoir units</th><th>Parallel river sections</th><th>2D domain</th></tr>
<tr><td>-</td><td>-</td><td>-</td><td>The floodplain was modelled using 2D approach for the whole area (grid size 5m).</td></tr>
<tr><th colspan="4">Representative Manning’s n values</th></tr>
<tr><th colspan="4">Channel. Manning’s n varies between 0.030 and 0.045</th></tr>
<tr><td colspan="2">0.030: Minor stream, clean, straight, full stage, no rifts or deep pools (node 10Ta173)
</td><td colspan="2">0.045: Minor stream, clean, winding, some pools and shoals with some weeds and stones (node 10Ta1851)
</td></tr>
<tr><th colspan="4">Floodplain. Manning’s n varies between 0.06 and 0.08</th></tr>
<tr><td colspan="2">0.06 to 0.08. Floodplain, dispersed bushes, weeds and few trees
</td><td colspan="2"></td></tr>
<tr><th colspan="4">Boundary conditions</th></tr>
<tr><td colspan="4">A tidal boundary extracted from the Rogerstown estuary within the Ballyboghil and Corduff River model was used as the downstream boundary unit for this model. Please refer to</td></tr>
</table>

Section 5.10.1 for more details on the modelling of the Rogerstown Estuary. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the Jone's Stream is 15 hours for the 1% AEP event. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n and model inflows.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.04	-0.06	0.06	-0.09
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.02	-0.03	0.13	-0.17

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when the inflows are changed. The most significant impact on water levels resulting from changes in model inflows occurs upstream of the structure at 10Ta2108.

The Jone's Stream joins the Bride Stream upstream of the area tidal influence; therefore the sensitivity analysis to tidal levels had no impact on this section of the model.

'Without defences' scenario

No defences present in the model.

Model Results Summary

The following section provides a brief overview of the flood hazard along Jone's Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

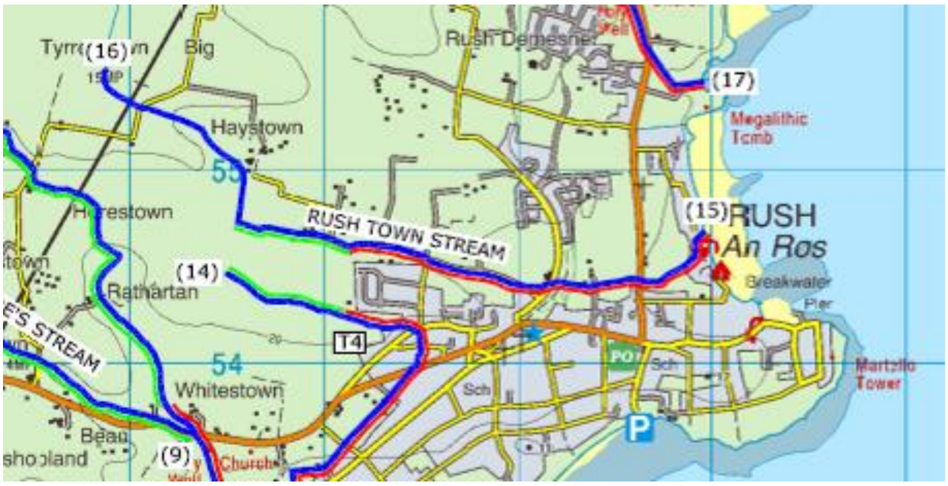
There is a limited extent of flooding along the Jone's Stream with localised pockets of flooding affecting undeveloped rural areas at the upstream extent of the modelled reach. Upstream of the junction with the Brides Stream, flooding poses a risk to a small number of properties. Refer to map JON/HPW/EXT/CURS/001.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.10m and for the HEFS is 0.18m. The maximum difference is 0.36m and is located just upstream of a long culvert with a low conveyance capacity at section 9Ba3030. The MRFS maps indicate that there is a minimal increase in flood extents along the Jone's Stream.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.10	0.18	0.36	1.00

5.20. Rush Town Stream

Introduction	
Water bodies	Rush Town Stream
APSR	Rush area



The Rush Town Stream has its source in Tyrrelstown, just north of Rush. It flows in an easterly direction before discharging to the Irish Sea at Rush. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the modelled watercourse. The catchment drains an area of 2.5km² and is broken down into four sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The channel length is 2.6km and it has no tributaries. There are no gauging stations on this river. The tidal/fluvial dominance transition point in the river channel is almost at the downstream extent of the model (100m upstream from the coast) due to the steep channel gradient (mean slope of 0.7%) based on the 1% AEP fluvial and tidal event.

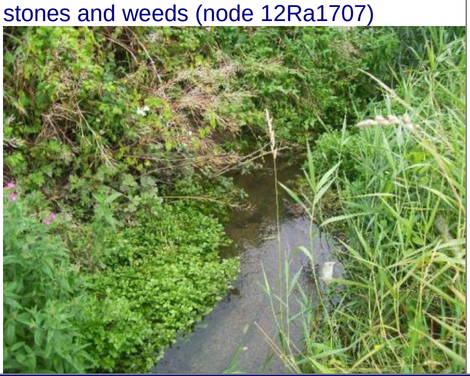
Model Build

A 1D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through the catchment of the Rush Town Stream. No 2D modelling was undertaken as the bed level is quite steep and the floodplain is narrow which means its hydraulic behaviour can be accurately modelled using 1D modelling techniques.

Summary of structures in the model		
Type	*Number*	*Summary*
Culvert/Bridge	30	The bridges were modelled using BRIDGE (ARCH), ORIFICE and VERTICAL SLUICE units.
Weir	1	Represented by a SPILL unit due to its irregular shape.
Gauging station	0	
Flood defences	0	
Other	0	

There is a history of flooding at some of the culverts on this river such as Skerries Road Bridge (R128) and Farran’s Lane culvert. This is due to blocking of the culverts and is not due to capacity issues with these culverts. These culverts were not included in the culvert blockage assessment as there is no history of properties flooding as a result of these culvert blockages.

Floodplain model build			
Extended cross sections	*Reservoir units*	*Parallel river sections*	*2D domain*
7	0	0	0

Representative Manning's *n* values	
Channel. Manning's n of 0.04	*Floodplain. Manning's n of 0.06*
0.045: Minor stream, clean, straight, full stage, no rifts or deep pools with more stones and weeds (node 12Ra1707) 	0.06: dispersed bushes, weeds and few trees (node 12Ra911) 
Colebrook-White friction varies between 0.001 and 0.006m	
0.001m: Concrete culvert, monolithic construction against steel forms (node 12Ra1239) 	0.006m: Old stone culvert (node 12Ra2157) 

Boundary conditions
A tidal boundary at the mouth of the Rush Town Stream was used as the downstream boundary unit for this model. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration
No calibration data was available to calibrate this model.

Critical Storm Duration
The critical storm duration calculated for the 1% AEP event for the Rush Town Stream is 17 hours. The same critical storm duration was used for all the AEP events.

Sensitivity
A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.03	-0.03	0.07	-0.08

Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow - 20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.06	-0.07	0.13	-0.13

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when the inflows are changed. The greatest impact on water levels occurs upstream of the Haystown Road bridge at section 12Ra2157U.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event and the impact on water levels is minimal as the tidal/fluvial dominance transition point is just 100m from the downstream boundary of the model. As the model doesn't have a tidal defence, sensitivity to tidal levels is along the tidal reaches of the river as expected

'Without defences' scenario
No defences present in the model.

Model Results Summary
The following section provides a brief overview of the flood hazard along Rush Town Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

The flood maps indicate that there is limited flooding along the Rush Town Stream for all AEP flood events. There is a small pocket of flooding on the shoreline next to the caravan park where the right bank is overtopped for a 1% AEP fluvial design event or greater and for a 10% AEP tidal event or greater. Refer to maps RUT/HPW/EXT/CURS/001 and RUT/HPW/EXT/CURS/T/001.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.10m and for the HEFS is 0.19m. The maximum difference is 0.35m and occurs at the downstream end of the model where the water level is directly controlled by the tide. A comparison of the flood extent maps between the current scenario and the MRFS indicates that there is a minimal increase in flood extents as a result of increases in mean sea levels and flows associated with the MRFS.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.10	0.19	0.35	1.00

5.21. Rush West Stream

Introduction

Water bodies	Rush West Stream
APSR	Rush area

Rush West Stream (marked (14) on the map above) has its source to the west of Rush. It flows in a south-easterly direction first, then in Rush town it turns and flows in a south-westerly direction before discharging into the mouth of the Rogerstown estuary. The map above provides an overview of the extent of the stream. Please refer to Figure 1 for more details on the extent of the hydraulic model of this watercourse. The catchment drains a small area of 1.25km². It is heavily urbanised and is broken down into two sub-catchments (refer to FEM FRAMS Final Hydrology Report, 2010). The channel length is 2.5km and it has no tributaries. There are no gauging stations on this river. The tidal/fluvial dominance transition point in the river channel is almost indistinguishable due to the steepness of the culvert before it discharges through the flapped outfall at the downstream end of the watercourse (i.e. on the shore line at Spout Road). This is based on the 1% AEP fluvial and tidal event.

Model Build

A combined 1D-2D hydrodynamic model was selected as the most appropriate method of simulating the routing of fluvial flows through this urbanised catchment.

Summary of structures in the model

Type	*Number*	*Summary*
Culvert/Bridge	12	The bridges were modelled using BRIDGE (ARCH) and ORIFICE units.
Weir	1	Represented by a SPILL unit due to its irregular shape.
Gauging station	0	
Flood defences	1	Flapped outfall
Other	0	

For the last 235m of the model, Rush West Stream is culverted (from Channel Road down to the coast line). The long culvert has one inlet and two apparent identical flapped outlets with the same invert level when viewing the survey. The survey photographs show clearly that water flows out of only one outlet.

An additional drawing of the long culvert was then provided showing that the culvert starts as one single pipe of 0.4m diameter with an inlet invert at 5.05 m AOD before changing into a 0.5m diameter 118m downstream. The 0.5m flapped outlet invert level is 1.83 m AOD. The other flapped outlet is the outfall of the local drainage system.

An assessment of culvert blockages was undertaken at the Channel Road culvert at node 11Wa267. Further details on the culvert blockage assessment are in Section 9.2.13~~9.2.13~~~~9.1.13~~.

Removing the downstream culvert flapped gate has no impact in terms of flooding, as Channel Road's culvert is very steep. Furthermore, there is no tidal wall defence to prevent tidal flooding bypassing the flapped outfall. Therefore, no 'without defences' scenario was undertaken.

<table>
<tr><th colspan="4">Floodplain model build</th></tr>
<tr><th>Extended cross sections</th><th>Reservoir units</th><th>Parallel river sections</th><th>2D domain</th></tr>
<tr><td>9</td><td>0</td><td>0</td><td>Final 750m of the channel (grid size 5m).</td></tr>
<tr><th colspan="4">Representative Manning's n values</th></tr>
<tr><th colspan="4">Channel. Manning's n varies between 0.03 and 0.04</th></tr>
<tr><td colspan="2">0.03: Excavated, dragged or man-made channel (node 11Wa1484)
</td><td colspan="2">0.04: Minor stream, clean, winding with some pools and sandbanks (node 11Wa1627)
</td></tr>
<tr><th colspan="2">Floodplain. Manning's n varies between 0.03 and 1.0.</th><th colspan="2">Colebrook-White friction of 0.006m</th></tr>
<tr><td colspan="2">0.06: dispersed bushes, weeds and few trees (node 12Ra1008)</td><td colspan="2">0.006: Concrete culvert, monolithic construction against steel forms, normal (node 11Wa1065)</td></tr>
</table>

Boundary conditions

A tidal boundary extracted from the Rogerstown estuary within the Ballyboghil and Corduff River model was used as the downstream boundary unit for this model. Please refer to Section 5.10.1 for more details on the modelling of the Rogerstown Estuary. Further information on the model boundary conditions is available in the FEM FRAMS Final Hydrology Report, 2010.

Model Calibration

No calibration data was available to calibrate this model.

Critical Storm Duration

The critical storm duration calculated for the 1% AEP event for the Rush West Stream is 6.5 hours. The same critical storm duration was used for all the AEP events.

Sensitivity

A sensitivity analysis was carried out in order to identify the dominant model parameters on predicted water levels. A sensitivity analysis was undertaken for changes in Manning's n, model inflows and the downstream boundary.

Manning's *n*				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Manning's n +20%	*Manning's n -20%*	*Manning's n +20%*	*Manning's n -20%*
Main channel	0.02	-0.02	0.06	-0.07
Model inflows				
Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	Model inflow +20%	*Model inflow -20%*	*Model inflow +20%*	*Model inflow -20%*
Main channel	0.03	-0.03	0.10	-0.06

In terms of roughness sensitivity, on average there is a minimal impact on the water levels when changing the Manning's n coefficient. Similarly for the flow sensitivity, on average there is a minimal impact on the water levels when changing the flows. The maximum impact results from increasing the flow by 20% and occurs upstream of a field access bridge at cross section 11Wa1616U.

The tidal sensitivity (+/-0.25m) was carried out for the 0.5% AEP tidal event but has almost no impact on water levels due to the steepness of the culvert discharging to the sea.

'Without defences' scenario

No defences present in the model

Model Results Summary

The following section provides a brief overview of the flood hazard along Rush West Stream. For further information on the flood risk, please refer to the FEM FRAMS Preliminary Options Report, 2010.

Flooding occurs at the downstream extent of the modelled reach, to the west of Rush town and around Channel Road. The flood maps indicate that a large urban area is at risk of flooding from a combination of both fluvial and tidal flooding. Surcharging of the culvert at channel road starts for a 4% AEP fluvial design event and results in flooding along Channel Road. Tidal flooding at Shore Road starts for the 0.5% AEP tidal design event extending inland to affect properties in Rush town. Refer to maps RSW/HPW/EXT/CURS/001 and RSW/HPW/EXT/CURS/T/001.

For discussion on flooding around the Rogerstown estuary, please refer to Section 5.10.1.

The average water level increase between the current scenario and the MRFS for 1% AEP fluvial event is 0.08m and for the HEFS is 0.17m. The maximum difference between the current scenario and the MRFS is 0.35m and occurs at the downstream end of the modelled watercourse where the water level is controlled by the outfall and the tide. A comparison of the fluvial and tidal current scenario and MRFS flood extent maps shows that the increase in mean sea level results in the largest increase in flood risk.

Watercourse	*Average Water Level Difference (m)*		*Maximum Water Level Difference (m)*	
	MRFS	*HEFS*	*MRFS*	*HEFS*
Main channel	0.08	0.17	0.35	1.00

6. Coastal modelling

6.1. Introduction

Modelling (coastal modelling) to simulate flooding from the sea has been undertaken to identify the flood hazard arising directly from coastal flooding. In addition the coastal modelling will support the development and appraisal of possible flood risk management measures, options and strategies to achieve the defined objectives for coastal areas and thence identify the most appropriate flood risk management strategy for the catchments in the study area.

The extreme sea levels derived from the Department of Agriculture, Fisheries and Food (DAFF) Strategic Coastal Flood Risk and Erosion Study have been used and as such coastal models to simulate off-shore or near-shore tide or surge dynamics or wave action, or extreme sea levels has not formed part of this work.

The modelling has considered the coastal defences (including high ground and coastal dunes) in place to protect the coastline.

The software used for this study was ISIS 2D, part of the ISIS suite which has been used for the other modelling elements of the study.

Limited model water level and flow boundary data, as would be derived from a regional model (Irish Sea) were not available to this study and the development of such was beyond the scope of the project. Hence model boundaries and the modelling approach has been based on readily available and derivable data.

Flooding due to wave action or overtopping has not been considered as part of the analysis.

This chapter of the report should to be read in conjunction with the coastal flood maps in Volume 2. These flood maps contain information water levels, flood outlines, confidence in levels and outlines, depths and hazards for a number of AEP events (please refer to Chapter 7 for a further details on the flood maps).

For technical readers of the report, this chapter should also be read in conjunction with the digital deliverables contained in Volume 3 of the report. This volume contains additional information for all AEP events and scenarios (current, MRFS, etc.).

6.2. Historical information and previous studies

The main source of information on historic flooding is the OPW National Flood Hazard Mapping website, www.floodmaps.ie. A record of at least 141 historic flood events in the study area since the 1940’s was made available by the OPW in GIS (MapInfo) layers. Of these 141 events, 22 records are related to coastal/tidal flooding (refer to Table B-3 in Appendix B of Hydrology Report).

The coastal flooding of 1st February 2002 at Portmarnock, Malahide, Baldoyle, Portrane, Swords, Skerries, Rush and Bettystown and that of the 1924 tidal flooding (anecdotal) at the Fingal and Meath coastal areas were the most significant tidal/coastal flooding events in the study area. Similarly, the November 2000 and November 2004 flooding at Skerries, Rush, Bettystown areas were among the major combined tidal and fluvial flooding events in the study area.

The principal previous studies undertaken that are relevant to the study area are:

- Mornington District Surface Water and Flood Protection Scheme, Final Preliminary Report, published in January 2004;
- Dublin Coastal Flooding Protection Project (DCFPP) Final Report, published in April 2005; and
- Irish Coastal Protection Strategy Study (ICPSS), Draft Final Technical Report - August 2008.

According to the Mornington District Surface Water and Flood Protection Scheme, Final Preliminary Report, published in January 2004 by Kirk McClure Morton for Meath County Council and OPW, the predicted spring tidal level at Boyne Estuary on 1st February 2002 was 2.28m OD (Malin Head). However, the actual highest tide that occurred on that day was 3.36m OD (Malin Head). According to the same report, this tidal flood approximates to a 1:100 year return period tide level.

The DCFPP, undertaken by Royal Haskoning for Dublin City Council and Fingal County Council, covered the Dublin City coastal area from Mortello Tower to the North of Portmarnock. According to the DCFPP Final Report, published in April 2005, the highest predicted tide at Dublin Port on 1st February 2002 was 1.93m OD (Malin Head). However, the actual highest tide that occurred on that day was 2.95m OD (Malin Head), which was around 1.02m higher than the highest predicted value on that day.

The DCFPP collated data on the February 2002 tidal flood event. The DCFPP recorded that flooding occurred of the Coast Road in the vicinity of the Mayne River and that two houses, adjacent to Baldoyle estuary, were flooded. The DCFPP also recorded that some flooding occurred at the junction of the Coast Road and Strand Road in Portmarnock (near the Sluice River) and that some land was also flooded in Portmarnock.

The DCFPP study also produced a set of flood hazard maps for the 0.5% AEP tidal event and a brief comparison of the results of the DCFPP study with the FEM FRAMS coastal results is provided in Section 6.6.

The ICPSS, Phase III, undertaken by RPS consulting for DAFF (now incorporated into the OPW) covered the coastline between Dalkey and Omeath. The Draft Final Technical Report - August 2008, which was made available to the project team, presents the work undertaken and the findings of Phase 3 of the ICPSS, Work Packages 2, 3 and 4A. Work Packages 2 and 3 provide the assessment of coastal flood risk at a strategic level.

The ICPSS used numerical modelling of combined storm surges and tide levels to obtain extreme water levels along the coastline. The application of extreme value analysis and joint probability analysis to both historic recorded tide gauge data and data generated by the numerical model allowed an estimation of the extreme water levels of defined exceedence probability to be established along the coastline.

A DTM developed by DAFF was used in the ICPSS to define the extent of the predictive floodplain. The predictive flood outlines were calculated by combining the results of the surge and tide level modelling, the statistical analysis, and the DTM using Geographic Information Systems (GIS) technology.

The resulting floodplain maps, including flood depth maps, are presented in the report and digital GIS layers have been made available for this study to allow comparisons between the studies. The following notable differences in approach and method need to be considered when comparing the results of the ICPSS and this study as they will lead to different results:

- The DTM used in this study is more recent and more accurate than that used in the ICPSS. The overall accuracy of the North East Coast DTM is between -0.35m to +0.52m at the 99% confidence limit. The accuracy, as specified by the procurement contract of the DTM used in this study, is ±0.20m at the 99% confidence limit;
- The model resolution in this study is significantly more detailed than that used in the ICPSS (ICPSS model resolution was used a maximum cell size of 200m along most of the Irish coastline, whereas in this study a 20m cell size has been used);
- This study has modelled the propagation of the flooding onto the land whereas the ICPSS projected the water levels and assumed that water levels remained constant between the coast and the landward limit of the floodplain. Such an approach does not consider flood paths and shows any area below the flood level as floodplain and will tend to over estimate the floodplain;
- The GIS processing techniques and DTM grid sizes used for mapping in this study lead to more detailed and refined assessment of the flood outlines when compared to the ICPSS. The ICPSS used 100m grid size for mapping flood outlines whereas this study has used a 5m grid size; and
- The DTM for this study has been augmented with data from the defence asset survey where relevant, whereas the ICPSS study has not considered any defence assets.

6.3. Data used to set up and undertake the analysis

6.3.1. Base DTM

The OPW provided the following 2008 DTM survey data of the region for use in the study:

- The 2m and 5m DTM of the FEM FRAMS area covering the HPWs, MPWs, APSRs, APMRs and the study area coastline; and
- The 2m low tide LiDAR DTM at the coastal and estuary area.

The DTM data set has been re-sampled and combined into a single 20m DTM to allow simulation of the entire coastline in a single model rather than separate models which could lead to boundary effects and greater uncertainty in results. A 20m resolution is considered highly detailed for a model of this application and is considered to be of sufficient detail to pick up local variations, such as high ground and coastal dunes, to a level of certainty commensurate with uncertainties of the DTM and boundary data. This DTM has been used for the 'without defences' model runs.

The DTM has been augmented with data from the defence asset survey where relevant, as detailed in Figure 6-1 for the 'with defences' model runs. Further details on the location of these defences are contained in Appendix C3. The process of adding the defence lines to the DTM was similar to adding ISIS Z-lines in the river models. However, due to the number of lines/points in the defence asset polyline and the format of the data, a new code was developed to add the defences to the DTM. A summary of this process is as follows:

- The code reads a text file containing the easting, northing and elevation of each survey point along the defences;

- Cells in the DTM that fall on a straight line connecting two adjacent defence survey points are identified;
- The elevations of these cells are modified by interpolation based on the elevations of the defence survey points; and
- The final output is a DTM with the elevation data of defences included.

Figure 6-1 Locations of defences added to the coastal model DTM

6.3.2. Tidal boundary data

DAFF provided water levels and flood outlines for design events for the exceedence probabilities of 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% as produced by the ICPSS. The OPW provided historic tide data at Dublin Port and at Port Oriel, Clogherhead.

Using the DAFF provided design event water levels and the Admiralty Tide Tables Volume 1, 2007: United Kingdom and Ireland, (United Kingdom Hydrographic Office, ISBN 0-70-771-5954), design event tide series were generated for range of AEPs, namely, 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1%, at 13 locations at the mouths of rivers and estuaries in the study area. Table 6-1~~Table 6-1Table 6-1~~ lists the locations, associated river/stream names and eastings and northings. The locations are presented visually in Figure 6-2. It is noted that the DAFF provided data did not include water levels at the design event of 4% AEP. For this purpose, the predicted water surface levels were plotted on a semi-log plot against the range of AEP for which tidal data was available from DAFF. The resulting plots were used to derive design water levels for the 4% AEP event.

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Table 6-1 Locations at which design event tidal boundaries were derived

Study area locations	River/stream names	Easting	Northing
SLU-MAY	Sluice and Mayne	325349	240309
WAR-BRO-GAY-LIS-TUR	Ward, Broadmeadow, Gaybrook, Lissenhall and Turvey	323693	246261
BAL-BAY-BRI-JON-RWS-COR	Ballyboghil, Baleally, Brides, Jones, Rush West, Corduff	325126	252698
RUT	Rush Town	326958	254685
CAT	St Catherine’s	326954	255454
RUR	Rush Road	326664	258547
MIL	Mill	325724	260081
BRA	Bracken	320509	263864
BNS	Balbriggan North	320149	264463
DEL	Delvin	318229	266337
MOS	Mosney (Bradden)	316951	269704
NAN	Nanny	316300	271147
BSS	Brookside	316192	273059

Figure 6-2 Locations along study area coastline at which design tidal boundaries were derived

Using linear interpolation, tide cycles incorporating surge have been generated with reference to OD Malin Head. The boundary condition with surge derived for Mosney is shown in Figure 6-3.

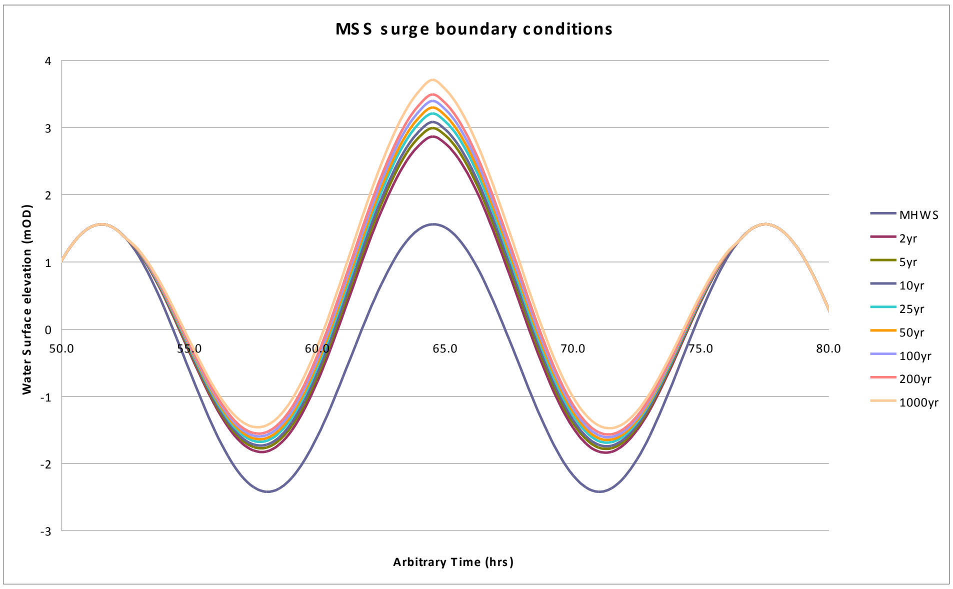


Figure 6-3 Surge boundary at Mosney (MOS)

6.3.3. Boundary

The water level boundaries were applied to the model as far offshore as was allowed by the available LiDAR data. Time varying boundary water levels were applied at each of the locations identified in Table 6-1~~Table 6-1Table 6-1~~ and Figure 6-2.

As the difference in water levels between adjacent locations was small (ranging between <0.1m) the same boundary conditions were applied up to the mid-point between each location.

6.4. Modelling set up

A single model of the entire coastline has been used rather than separate models which could lead to boundary effects and greater uncertainty in results.

The Alternating Direction Implicit (ADI) solver in ISIS 2D was used for all calculations. The ADI scheme discretises the shallow water equations over a regular grid of square cells, calculating water depths at the cell centres and discharges at the cell edges. A constant Manning's n value of 0.03 was used throughout the study area.

Eight independent models were established, one for each AEP event (50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1%). The time step for all of the models was set at 1 second. The total simulation period was 44.5 hours. This included a peak surge tidal cycle and an extended period before and after to allow for settling (refer to Figure 6-4 for a typical tidal cycle).

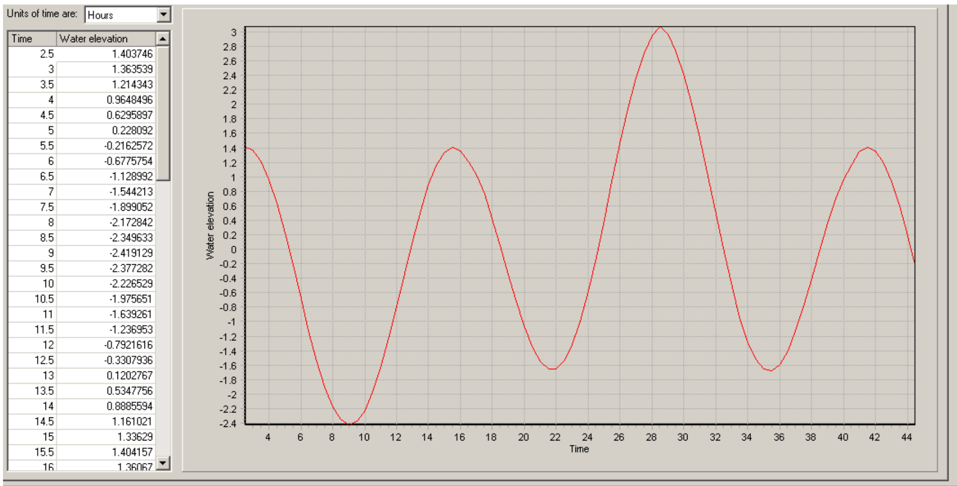


Figure 6-4 Tidal cycle for the 4% AEP at the Mill Stream

6.5. Model calibration and sensitivity analysis

6.5.1. Calibration

It was not possible to obtain suitable calibration data to undertake detailed calibration or validation of this model. Neither FCC nor MCC were able to provide any suitable data for the February 2002 tidal event which caused significant flood damage to the Dublin city coastline but only localised flooding to the Fingal and Meath coastline. According to the FCC Local Engineers report on the 2002 tidal event, only 14 houses, 1 shop and 1 pumping station were actually flooded internally although other properties were at risk. There was significant flooding of various roads including Strand Road (Malahide), Coast Road (Baldoyle), Estuary Road (Swords) and Crescent Road and South Shore Road in Rogerstown, Rush.

6.5.2. Sensitivity analysis

The sensitivity of the model to the global roughness value was tested. Manning's n was set at 0.03 for whole domain in the final model. For the sensitivity test the Manning's value was varied by n +/- 0.01 for the 1% AEP event model. The impact on water levels of the change in Manning's n values is shown in Table 6-2~~Table 6-2Table 6-2~~.

Table 6-2 Results of sensitivity analysis to Manning's n values

Location	1% AEP Water Depth (m) n = 0.03	Change in 1% AEP Water Depth (m) n = 0.02	n = 0.04
SLU-MAY	0.818	-0.002	0.000
WAR-BRO-GAY-LIS-TUR	2.237	-0.129	-0.037
BAL-BAY-BRI-JON-RWS-COR	2.094	-0.001	0.000
RUT	1.408	-0.034	-0.046
CAT	2.764	0.000	0.000

Location	1% AEP Water Depth (m) n = 0.03	Change in 1% AEP Water Depth (m)	
		n = 0.02	n = 0.04
RUR	1.439	-0.046	-0.003
MIL	1.607	0.000	-0.002
BRA	3.475	-0.001	-0.001
BNS	1.135	0.002	-0.001
DEL	1.874	0.000	0.000
MOS	1.070	-0.001	0.000
NAN	2.906	0.000	0.000
MSS	1.861	0.000	0.000

As the results in Table 6-2~~Table 6-2Table 6-2~~ demonstrate, changing Manning's n values by approximately +/- 33% result in minimal changes in water level (up to a maximum of 5%). Changes in water levels are within acceptable limits.

6.6. Modelling results

The peak elevations and velocities have been extracted from the model results data files and post processed to produce the flood maps. Results from the 20m model have to be transposed onto a 5m DTM so as to provide greater resolution in the final maps and allow the natural contour line to be followed more closely. The flood extents from the coastal model have been merged with those of the river models (tidally dominated runs) to produce flood extents for the coasts, estuaries and tidally dominated reaches of the rivers.

There is limited coastal flooding in the Fingal-East Meath study area, mainly due to the high level of land along the coast. Localised coastal flooding for lower probability AEP events (i.e. 1%, 0.5% and 0.1%) does occur in Bettystown, Laytown, Skerries, Rush, the Burrows Malahide and Portmarnock. Further discussion on the fluvial and tidal flood risk in Laytown, Bettystown, Skerries, Rush, Malahide and Portmarnock is detailed in the Chapter 5. There is an increase in the flood extent and hence the risk of coastal flooding for the MRFS, particularly in Balbriggan, Skerries, Malahide, Portmarnock and Baldoyle.

The results of the coastal modelling undertaken for this study have been compared to those of the ICPSS. At all locations the ICPSS shows more extensive flooding inland from the coast and this is due to the methodology used to project the water levels from the sea onto the land, as explained in Section 6.2.

Similarly the results of the coastal modelling undertaken for this study have been compared to those of the DCFPP in the vicinity of Portmarnock and Sutton cross. There is generally good correlation between the two sets of results. Additional flooding is shown in some areas (e.g. Portmarnock golf course and Beechwood golf course) but the notes on the DCFPP study indicate that they had limited survey data. Some areas are shown as being protected (i.e. the wetland behind the tidal defence valve on the Mayne River). The FEM FRAM study shows that this area is not protected as there is flooding due to fluvial influences. The DCFPP study shows significant flooding of Baldoyle & Sutton but note that this is 'worst case' scenario used to determine overtopping. The majority of the flood extent is due to wave overtopping through gaps. This flood extent is very similar to the FEM FRAM study results for the MRFS.

As discussed in Section 6.3.1 a 'without defences' coastal model was also run. The results show that the defences added to the coastal model have a limited impact on the flooding for the current scenario (identified as 'defended areas' on the coastal flood extent maps).

7. Flood hazard mapping

7.1. Introduction

This section describes the development of the detailed flood mapping formats for the FEM FRAM Study. Most of the formats for mapping were established during the Lee CFRAM study and only minor amendments have been made as part of this study.

7.2. Flood mapping formats

7.2.1. Identified end users of flood maps

Flood maps are one of the main outputs of the hydrologic and hydraulic assessments and are the medium by which model results are communicated. Results are being communicated differently depending on the end users role. Each user needs to understand and interpret the maps, therefore the method of communication needs to be consistent, clear and provide the information that they require to an appropriate level of detail. The following end user roles / requirements have been highlighted as part of the project:

- Planning and development management;
- Flood risk management planning and design;
- Public awareness and preparedness; and
- Emergency response planning.

The best method of transferring the information to the user has been discussed at a number of project meetings. End users have also been consulted to gauge their views on the proposed formats.

7.2.2. Flood map types

The FEM FRAM Study has produced a range of map output types from the hydrologic and hydraulic modelling process:

- Flood extent maps which show the area inundated by a flood event for a given AEP;
- Flood zone maps which show flood zones A, B and C, to facilitate implementation of the Guidelines on the Planning System and Flood Risk Management (DEHLG & OPW, November 2009);
- Flood depth and velocity maps which show the depths and velocities of the area inundated of a given AEP;
- Flood hazard function maps which show the hazard of a flood event of a given annual exceedence probability, as a function of the depth and velocity based on Defra's Flood Risk to People Phase 2 methodology; and
- Metadata which describes the digital datasets used in the creation of the flood maps.

These outputs are required to meet the needs of the end users and they will be in different formats, either hardcopy or electronically through the OPWs flood hazard mapping website (www.floodmaps.ie). The flood extent maps for the current scenario are available on the FEM FRAMS website for the remaining duration of the project (www.fingaleastmeathframs.ie).

Table 7-1

~~Table 7-1~~

~~Table 7-1~~ shows the types of output required and the method of communication to each end user.

Table 7-1 End user flood mapping format requirements

	Format		Type				
End user	**Hard copy**	**Web**	**Flood extents**	**Flood zone**	**Flood depth**	**Flood velocity**	**Flood hazard**
Planning and development management	✓	✓	✓	✓	✓		
Flood risk management, planning and design	✓	✓	✓		✓	✓	✓
Public awareness and preparedness		✓	✓				
Emergency response planning	✓				✓	✓	✓

7.2.3. Mapping produced

The mapping produced as part of the FEM FRAM Study is comprehensive. Table 7-2 shows the deliverables as part of the study for each type of map for the current and future scenarios.

Table 7-2 FEM FRAMS flood mapping deliverables

Requirement	Current	MRFS
Hardcopy		
Flood Extent, Node Points and Uncertainty	10, 1/0.5*, 0.1% AEPs	10, 1/0.5*, 0.1% AEPs
Flood Zone	1/0.5*, 0.1% AEPs	1/0.5*, 0.1% AEPs
Flood Depth	10, 1/0.5*, 0.1% AEPs	Not required
Flood Velocity	10, 1/0.5*, 0.1% AEPs	Not required

Requirement	Current	MRFS
Flood Hazard	10, 1/0.5*, 0.1% AEPs	Not required
Web		
Flood Extent and Node Points	All**	All**
Flood Zone	Not required	Not required
Flood Depth	All**	Not required
Flood Velocity	All**	Not required
Flood Hazard	All**	Not required

** 1/0.5% AEP means the 1% AEP fluvial and 0.5% AEP tidal. There is a requirement in the brief that the fluvial event to be modelled is the 1% AEP and not the 0.5% AEP and similarly that the tidal event to be modelled is the 0.5% AEP and not the 1% AEP. All other AEP events modelled are the same for both fluvial and tidal events.*

*** Where 'All' is stated in the table this means outputs for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.1% AEPs*

7.3. Hardcopy flood maps

7.3.1. Flood extent maps

The flood extent maps are designed to increase awareness among the public, local authorities and other organisations of the likelihood of flooding, and to encourage people living and working in areas prone to flooding to find out more and take appropriate action.

Flood extent mapping procedure – 1D models

In order to generate flood maps from 1D maps the following procedure is followed:

- Generation of a Triangulated Integrated Network (TIN) from the model – the TIN is created from points taken at the extremities of the river sections, lowest point of the river section, extremities of extended sections and the edge of reservoir units. These points correspond to a model node name and hence a water level from model results;
- Modification of TIN – once the TIN is created it is modified to ensure that it represents the flow paths accurately;
- Creation of water surface profile – Once the TIN is finalised, it is used to compute a water surface profile based on the link between the model nodes and the model results; and

- Creation of the flood extent – The flood extent is created by intersecting the water surface profile with the DTM. Where the water surface profile minus the DTM is greater than zero, flooding occurs. A GIS data set is created from this calculation and is displayed on the flood extent map.

ISIS Mapper and/or Arcview – ArcGIS – MapInfo have been used to undertake the flood extent creation process.

Flood extent mapping procedure – 2D models

Flood extent maps from 2D models are much simpler to create as the model outputs a water surface and/or a depth grid from the 2D domain. This is then used to create the flood extent by intersecting it with the DTM as per the 1D flood mapping process above or by using the depth grid directly.

Cleaning of flood extents

During the production of the flood extents a number of artefacts and islands are present initially from the intersection of the water surface profile with the DTM. A semi-automated cleaning routine has been developed to tidy the flood extents in MapInfo. This process is:

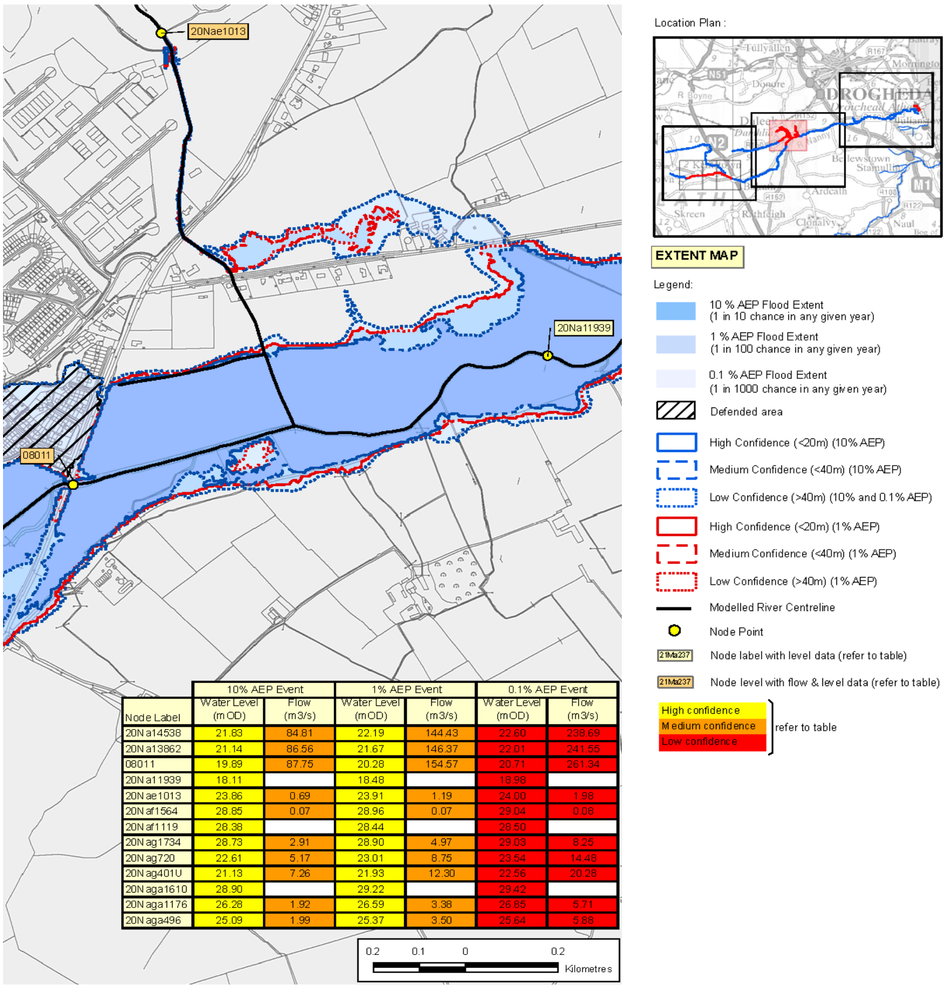
- Island filling of holes smaller than $100m^2$ (approx., this could change accordingly);
- Island removal of polygons smaller than $100m^2$ (approx., this could change accordingly);
- Node thinning of vertices in the polygon outline of less than 10m; and
- Bowtie removal in the polygon outline

Format of the fluvial flood extent maps

The format of the hardcopy fluvial flood extent map is shown in Figure 7-1 for the Lissenhall River. The key features are:

- Maps at 1:10,000 scale for APSRs and at 1:25000 and with background mapping at 1:50,000 for APMRs with the mapping in greyscale;
- Fluvial flood events are shown for 10%, 1% and 0.1% AEPs, coloured using a transparent fill from dark blue to light blue. Points along the river centreline with a table on the map showing the flow for 10%, 1%/0.5% and 0.1% AEP (in selected locations) and water level at each point for 10%, 1%/0.5% and 0.1% AEP;
- Fluvial and tidal maps are shown separately so that it is possible to see the source of flood risk;
- Areas benefiting from defences are shown by a grey hatched area;
- Uncertainty is shown by a changing flood extent outline: solid – high confidence; dashed – medium confidence; dotted – low confidence. Outlines are blue, except for the 1% AEP which is in red to make it more visible. Refer to Section 7.5.3 for further details;

- Tables of flows and water levels also contain the level of uncertainty of the information. This is shown in yellow, orange and red (high, medium and low confidence respectively); and
- Notes on the flood map to tell the user if the area covered by the map is also at risk from a second source of flooding (i.e. on a fluvial map, a note will identify if the area is also at risk of tidal flooding, and vice versa).



	10% AEP Event		1% AEP Event		0.1% AEP Event	
Node Label	Water Level (mOD)	Flow (m3/s)	Water Level (mOD)	Flow (m3/s)	Water Level (mOD)	Flow (m3/s)
20Na14538	21.83	84.81	22.19	144.43	22.60	238.69
20Na13862	21.14	86.56	21.67	146.37	22.01	241.55
08011	19.89	87.75	20.28	154.57	20.71	261.34
20Na11939	18.11		18.48		18.98	
20Nae1013	23.86	0.69	23.91	1.19	24.00	1.98
20Naf1564	28.85	0.07	28.96	0.07	29.04	0.08
20Naf1119	28.38		28.44		28.50	
20Nag1734	28.73	2.91	28.90	4.97	29.03	8.25
20Nag720	22.61	5.17	23.01	8.75	23.54	14.48
20Nag401U	21.13	7.26	21.93	12.30	22.56	20.28
20Naga1610	28.90		29.22		29.42	
20Naga1176	26.28	1.92	26.59	3.38	26.85	5.71
20Naga496	25.09	1.99	25.37	3.50	25.64	5.88

Figure 7-1 Extract from fluvial flood extent map

The user should note the full valley flows were not derived for all nodes in the model. There are two ways of deriving flow across the valley for a 1D-2D model as follows:

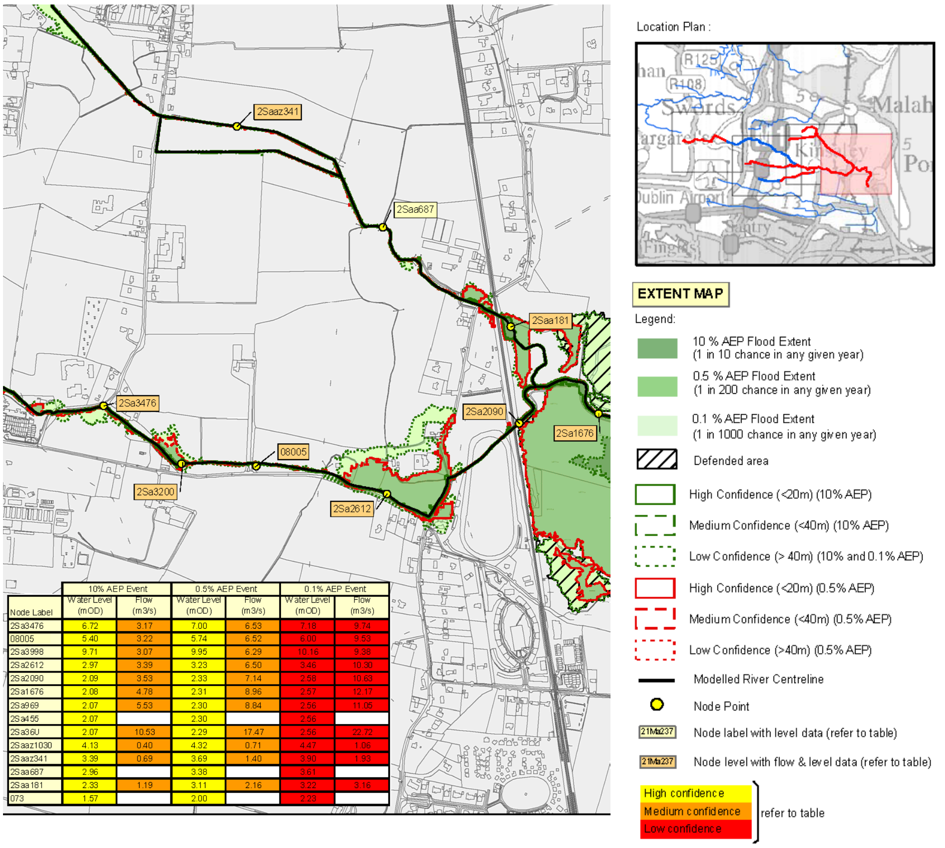
- Setup PO lines (shape file) before the runs at each location which automatically creates a result file; and
- Read manually the flows from the result grids at each location (left and right banks).

The 1D (river channel) and 2D (floodplain) flows have to be added for both approaches. The second method was adopted because the node locations were agreed after the model simulations finished. However, the flow result grids are available in digital format.

Format of the tidal flood extent maps

The format of the hardcopy tidal flood extent is shown in Figure 7-2 for the extent of tidal flooding on the Lissenhall River. The key features of the maps are similar to the fluvial flood maps, with the differences being:

- Tidal flood events are shown for 10%, 0.5% and 0.1% AEPs, coloured using a transparent fill from dark green to light green. Points along the edge of the flood extent at key locations; and
- Table on the map showing the water level for 10%, 0.5% and 0.1% AEP at each point and flow for 10%, 0.5% and 0.1% AEP only in selected locations.



Node Label	10% AEP Event		0.5% AEP Event		0.1% AEP Event	
	Water Level (mOD)	Flow (m3/s)	Water Level (mOD)	Flow (m3/s)	Water Level (mOD)	Flow (m3/s)
2Sa3476	6.72	3.17	7.00	6.53	7.18	9.74
08005	5.40	3.22	5.74	6.52	6.00	9.53
2Sa3998	9.71	3.07	9.95	6.29	10.16	9.38
2Sa2612	2.97	3.39	3.23	6.50	3.46	10.30
2Sa2090	2.09	3.53	2.33	7.14	2.58	10.63
2Sa1676	2.08	4.78	2.31	8.96	2.57	12.17
2Sa969	2.07	5.53	2.30	8.84	2.56	11.05
2Sa455	2.07		2.30		2.56	
2Sa36U	2.07	10.53	2.29	17.47	2.56	22.72
2Saaz1030	4.13	0.40	4.32	0.71	4.47	1.06
2Saaz341	3.39	0.69	3.69	1.40	3.90	1.93
2Saa687	2.96		3.38		3.61	
2Saa181	2.33	1.19	3.11	2.16	3.22	3.16
073	1.57		2.00		2.23	

Figure 7-2 Extract from tidal flood extent map

7.3.2. Zone

Flood zone maps show three zones depicting three zones, within the 1% AEP, between the 1% and 0.1% AEP and outside the 0.1% AEP. The maps have been developed following the publication of the Guidelines on Planning and Flood Risk Management.

Format of zone maps

The map borders, features and general components of the hardcopy zone maps are the same as the flood extent maps. The key features particular to the zone maps and as demonstrated in Figure 7-3~~Figure 7-3Figure 7-3~~, are:

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- The 1/0.5% and 0.1% only are shown the maps;
- Zone A, the area within the 1/0.5% AEP is shaded red;
- Zone B, the area between the 1/0.5% and 0.1% AEP is shaded orange; and
- Zone C, the area outside the 0.1% AEP is shaded yellow.

7.3.3. Depth

Flood depth maps show where the water would flow over time and how deep it would get in a given annual exceedence probability. The maps are useful in planning and design to understand the depth of flooding in area and they allow emergency responders to determine rescue areas, evacuation areas and potential evacuation routes.

Format of depth maps

The map borders, features and general components of the hardcopy depth maps are the same as the flood extent maps. The key features particular to the depth maps are:

- The 10%, 1/0.5% and 0.1% are shown on individual maps;
- Depth information only required for the current situation velocity maps; and
- Flood depths are shown on the map in six graduated classes, coloured light blue to purple for low to high depths respectively as shown in Figure 7-3~~Figure 7-3Figure 7-3~~. The classes used on the map are as follows:
  - 0 – 0.25m
  - 0.25 – 0.5m
  - 0.5 – 1.0m
  - 1.0 – 1.5m
  - 1.5 – 2.0m

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- > 2m

7.3.4. Velocity

Flood velocity maps show the speed of water flow for a given annual exceedence probability. These maps along with the flood depth maps help users to understand the flood risk in an area. Flood velocity maps are a component of the flood hazard maps described in 7.3.1 above.

Determining velocity from 1D models

Trying to determine velocities in the floodplain from a 1D model is not straightforward. We have developed an approximate method for establishing the velocity and hence hazard maps from the 1D models by using the Manning's equation.

$$V = \frac{1}{n}\frac{A^{2/3}}{P^{2/3}}S_o^{\ 1/2}$$

Equation 7-1

From the 1D models accurate depth grids were produced. Using these it was assumed that the area (A) is calculated as the grid cell size multiplied by the flood depth and the wetted perimeter (P). The slope component (S) was calculated in GIS from the DEM and approximated to a 50m cell resolution.

Determining velocity from 2D models

ISIS 2D outputs velocity to the 2D grid which has a 2-5m cell resolution.

Format of velocity maps

The map borders, features and general components of the hardcopy velocity maps are the same as the flood extent maps. The key features particular to the velocity maps are:

- The 10%, 1/0.5% and 0.1% are shown on individual maps;
- Velocity information only required for the current situation velocity maps; and
- Flood velocities are shown on the map in five graduated classes, coloured yellow to red for low to high velocities respectively as shown in Figure 7-3~~Figure 7-3Figure 7-3~~. The classes used on the map are as follows:
  - $0 – 0.25ms^{-1}$
  - $0.25 – 0.5ms^{-1}$
  - $0.5 – 1.0ms^{-1}$
  - $1.0 – 2.0ms^{-1}$
  - $> 2ms^{-1}$

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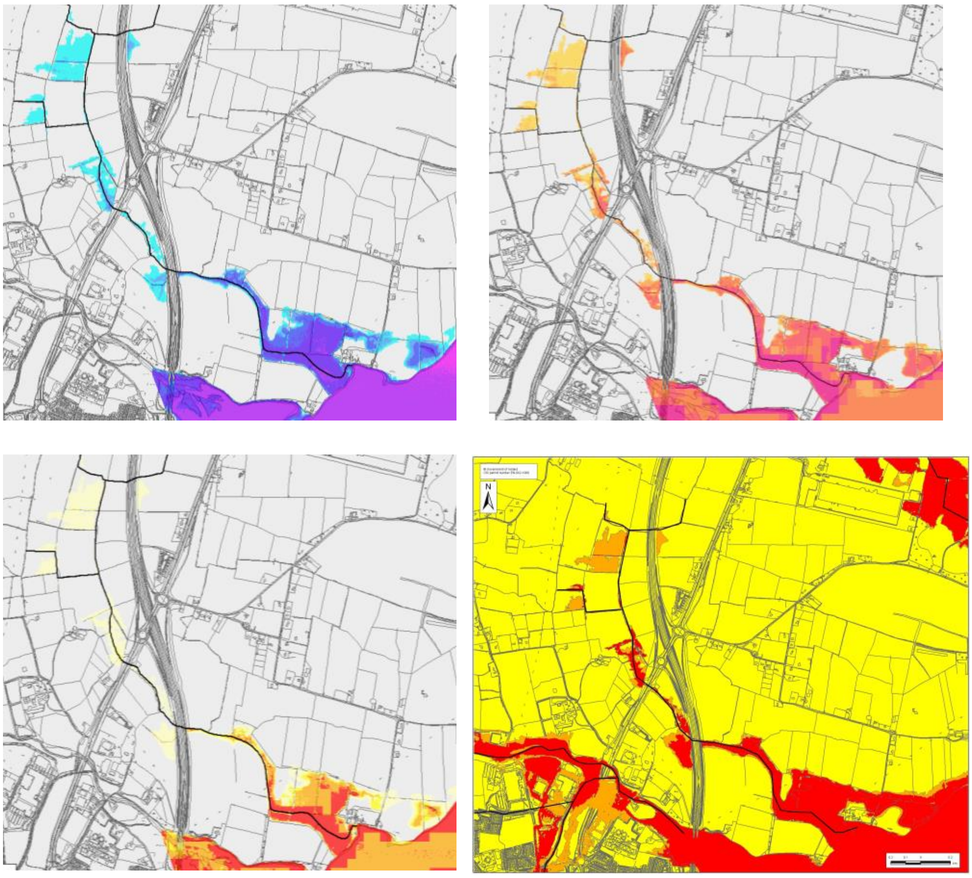

Figure 7-3 Top Left: flood depth map, Top Right: flood velocity map, Bottom Left: flood hazard map, Bottom Right: flood zone map

7.3.5. Hazard

Flood hazard maps show the risk which may be experienced by people for a particular AEP. This is calculated as a function of the depth and velocity of flood waters.

The FEM FRAM Study uses the methodology and concepts shown in the Defra / EA guidance Flood Risks to People Phase 2 to calculate flood hazard.

The flood hazard maps are created by calculating the hazard from the depth and velocity grids from Sections 7.3.3 and 7.3.4. The formula adopted for the FEM FRAM Study is:

$$hazard = d \times (0.5 + v) \qquad \textit{Equation 7-2}$$

The classifications of the degree of flood hazard are shown in Table 7-3 along with the graduated colours used to display the flood hazard on the maps. An example of a flood hazard map is shown in Figure 7-3~~Figure 7-3Figure 7-3~~.

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Table 7-3 Flood hazard map classifications

d x (0.5 + v)	Degree of flood hazard	Colour on map
< 0.75	Caution	Light yellow
0.75 – 1.25	Moderately dangerous for some people	Yellow
1.25 – 2.5	Significant danger for most people	Orange
> 2.5	Extreme danger for all	Red

7.4. Digital flood maps

7.4.1. Requirements

As part of the FEM FRAM Study project requirements, digital copies of the flood extents, velocity, depth and hazard maps are to be provided. In the future, it is intended to display these outputs on the OPW's flood mapping website (www.floodmaps.ie).

The OPW's flood mapping website currently contains historical flooding information, but the intention is to enable this website to host the flood mapping outputs of the National FRAM studies. The specification for how the flood maps will be shown on this website is outside the scope of this project. The digital deliverable outputs required are shown in Table 7-2.

7.4.2. Format of the digital deliverables

The digital deliverables to be provided are the GIS outputs that have been created for the hardcopy flood mapping formats. No formatting of the digital deliverables is required as these will be set when the files are uploaded to the flood mapping website.

7.4.3. Attribute data

Flood extent outlines

Flood extent outlines are merged so that a single polygon represents an AEP flood event. The current and mid-range future scenario event polygons are all merged into one GIS dataset. The attribute data contains details of what each polygon shows.

Node points

Flow and water level information for a node point (river cross sections) is placed on record line in the attribute data. This enables the user of the data to click on the node and obtain a table of flows and water levels for each AEP flood event and for both the current and mid range future scenario.

Flood depth, velocity and hazard grids

No attribute data is needed for the flood depth, velocity and hazard grids as it is inherent in the structure of the GIS deliverable.

7.5. Uncertainty analysis

As part of the LEE CFRAM Study, a research and development project was undertaken to provide guidance for the development of uncertainty estimates in flood risk maps. This report is entitled “Uncertainty Estimation Research and Development” (issued July 2008).

The report describes the use of a scoring method for estimating uncertainty in the flood levels predicted by hydraulic models, and how to transform this level uncertainty estimates into flood outline uncertainty estimates.

7.5.1. Application of uncertainty

The method used to estimate uncertainty in flood outlines is shown in Figure 7-4. First, the uncertainty in water levels is estimated from scores assigned to the hydrological accuracy, model complexity and peak flow, through use of a Microsoft Excel spreadsheet. The water level is then transformed into a horizontal uncertainty in the flood outline location using a tool called “UMap” (Uncertainty Mapping), which takes into account the water level uncertainty and the floodplain topography.

The method has been developed and calibrated using twelve catchments in Ireland and the UK. The application of this method is described in the next sections.

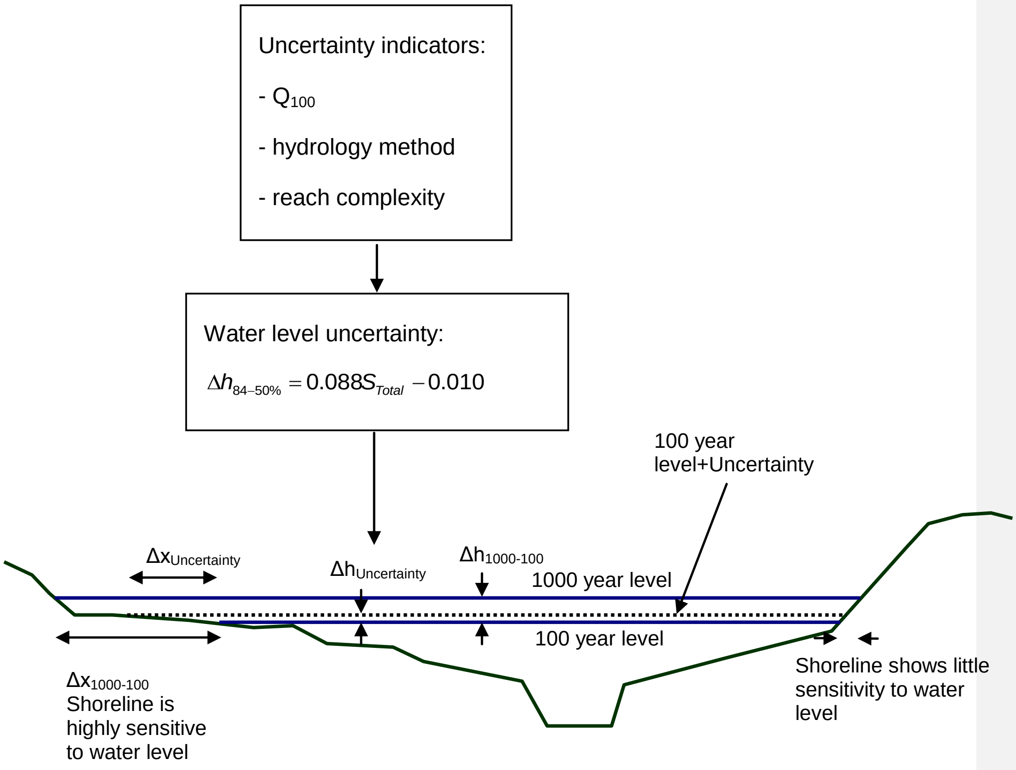

Figure 7-4 Schematic of method estimating uncertainty in 1% and 0.1% AEP outlines

Requirements

The uncertainty estimation method requires:

- Information on the hydrological analysis used to derive design flows used as boundary conditions for the hydraulic model;
- Information on the complexity of the hydraulic model itself. The method is not specific to any one model; and
- Flood outlines in the form of ESRI shape files.

Flood outlines are not required for an estimate of water level uncertainty alone.

Water level uncertainty

Uncertainty in predicted water levels is estimated from three sources:

- Hydrology;
- Hydraulic model complexity; and
- Peak of the design flow hydrograph.

The user assigns a score to each of these factors, with these scores expressing the uncertainty in each. Scores assigned for these factors are summarised in Table 7-4.

Table 7-4 Scores used to estimate uncertainty in water levels

Index Flood Method		**Score**
Catchment Descriptors (Flood Studies Report)		4
[1]*Catchment Descriptors (Flood Estimation Handbook)*		*3*
Short Record ≤ 10 years		2
Long Record > 10 years		1
[1]*Not used for Irish catchments*		
Model Complexity	**Units/km**	
Complex - reservoir units, many inflows, and/or many branches	>40	2
Medium - some inflows, branches and structures	20-40	1
Simple - few inflows, branches and structures	<20	0

The hydrology score is based on the method used to estimate the index flood (the mean or median annual flood) from either catchment descriptors or gauged measurements. For estimates relying on both these methods (e.g. where a catchment descriptors method is used to scale gauged flows for an ungauged site), a score somewhere between the catchment descriptors and gauged flow scores would be appropriate. For example, where a short gauged record is used to estimate design flows at an ungauged site, a score of 2.5 might be used to represent the extra uncertainty introduced in transferring the flows from the gauged site.

The model complexity is a subjective assessment, but Table 7-4 also includes a guide to the number of hydraulic units (cross sections, weirs, junctions *etc*.) per km of reach length for typical models of different complexities. While this can be a useful guide to assigning a model complexity, the user should use their judgment in determining this parameter. For example, a model with many cross sections and few other hydraulic units, while having a large number of units per km, might be assigned a lower complexity factor as a relatively simple model.

The water level uncertainty can be described as a probability distribution. These factors are combined in the following formulae (also implemented in the spreadsheet), to calculate the uncertainty in water levels:

$$S_{Total} = S_{Hydrology} + S_{Complexity} + \frac{Q}{200} \qquad \text{Equation 7-3}$$

$$\Delta h_{84-50\%} = 0.088 S_{Total} - 0.010 \qquad \text{Equation 7-4}$$

$$\Delta h_{95-50\%} = 0.16 S_{Total} - 0.018 \qquad \text{Equation 7-5}$$

Equation 7-4 gives the difference between the 84%ile and the median of the uncertainty (corresponding to +1 standard deviation), and Equation 7-5 gives the 95%ile difference (+1.65 standard deviations). The user needs to decide which of these percentiles to use to represent the uncertainty, with the 95%ile giving a wider uncertainty bound than the 84%ile.

The user should note that this method gives a single estimate of uncertainty for each model reach to which it is applied. For larger models, it may be worthwhile treating the model as several reaches, and calculating an uncertainty for each reach. This may be useful in situations where a model has several distinct reaches of different complexities, for example where a complex model is used to represent flow in an urban area, with simpler representations elsewhere.

Flood outline uncertainty

Once an uncertainty in water levels has been estimated, the UMap tool can be used to transform this into an uncertainty in flood outline location. UMap requires the following data:

- Digital Terrain Model (DTM) in ESRI ascii raster format;
- Shapefile of the flood outline for which the location uncertainty is to be calculated;
- Another shapefile flood outline, typically from another design flow, produced by same model and mapping method. The exact nature of this outline is not important, although it should lie outside (i.e. represent a larger flood extent) the outline for which uncertainty is required;
- Water level uncertainty. The UMap tool can only accept a single uncertainty value, so if separate water level uncertainties are estimated for different reaches, the shapefile needs to be split into corresponding regions in GIS; and

- The discretisation levels. These control how UMap splits the outline into lengths of different uncertainty. Three classes are produced, and these are specified by giving the two values dividing these classes. For example, if set to 20m and 40m, UMap will output polylines with uncertainty <20m, 20m-40m and >40m.

UMap is a command line tool, and should be run from a MSDOS command window. Output is in the form of a shapefile, with the uncertainty class for each polyline listed in the attribute table.

7.5.2. Classification of the uncertainty

There are two ways to determine the uncertainty classification parameters.

- Method 1 - by trial and error to get equal numbers of lines in each class. The number of shapefile points in each class is displayed by UMap; and
- Method 2 - by determining low, medium and high classes of uncertainty with reference to how the uncertainty mapping output will be used. Typical values might be < 20m (high accuracy), 20-40m (medium accuracy) and > 40m (low accuracy).

Method 1 allows the comparison of uncertainty between parts of the same model, but when comparing the results across multiple models and areas, it is important to consider the uncertainty as a common metric, which is what method 2 allows. Method 2 adopts an approach where the end user determines what is acceptable in terms of horizontal flood extent accuracy.

Method 2 has been adopted for the FEM FRAM Study. The classification of uncertainty has been determined as:

- High confidence is described as the flood extent having a horizontal distance uncertainty measure of less than 20m;
- Medium confidence is described as having a horizontal distance uncertainty measure between 20 – 40m; and
- Low confidence is described as the flood extent having a horizontal distance uncertainty measure of greater than 40m.

7.5.3. Display of uncertainty on flood extent maps

Each part of the flood outline output by UMap is associated with a horizontal distance uncertainty measure, which is displayed using different line styles around the flood extent.

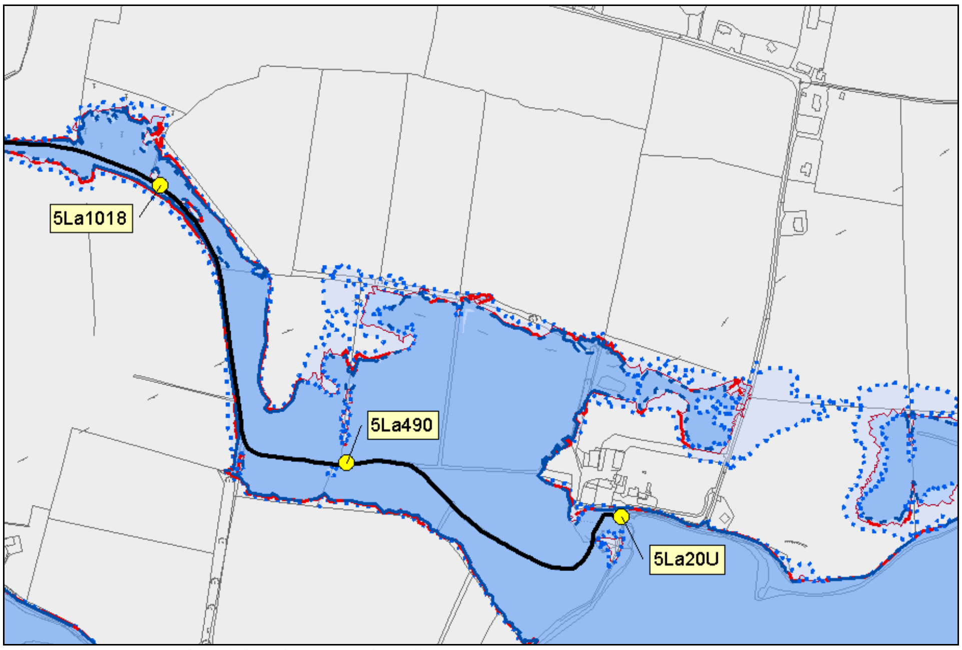

Figure 7-5 Example of flood uncertainty lines

Figure 7-5 shows an example of how uncertainty is shown on the flood extent maps. Uncertainty is shown on the flood extent maps in the following ways:

- Line type:
  - Solid lines show a high confidence in the horizontal distance uncertainty measure
  - Dashed lines show a medium confidence in the horizontal distance uncertainty measure
  - Dotted lines show a low confidence in the horizontal distance uncertainty measure
  - Outlines are shown in blue on 10% AEP and 1% AEP flood extents. For the 1% AEP (fluvial) and 0.5% AEP (tidal) the outline is shown in red to enable the map to be clearer.
- 0.1% AEP uncertainty lines are classified as low confidence. This is primarily due to the lack of a higher order event to carry out an uncertainty estimate, but in mitigation, such a high order event will be inherently uncertain;
- Uncertainty estimates are only required for the AEPs listed above; and
- All MRFS uncertainty lines are classified as low confidence. By definition all future scenarios have a high level of uncertainty.

The flood extent maps include tables of flow and water level for the 10%, 1% and 0.1% AEP. Confidence levels and flows have been colour coded as follows:

- High confidence figures are shown in yellow;

- Medium confidence figures are shown in orange; and
- Low confidence figures are shown in red.

Water Level confidence

The uncertainty in water levels for fluvial scenarios is estimated from scores assigned to the hydrological accuracy, model complexity and peak flow. These factors are combined in the equations 7.3 and 7.4 as described in Section 7.5.1. The confidence in levels has been classified as follows:

- High confidence is described as the water level having a vertical distance uncertainty measure of less than 0.40m;
- Medium confidence is described as the water level having a vertical distance uncertainty measure between 0.40 – 0.70m; and
- Low confidence is described as the water level having a vertical distance uncertainty measure of greater than 0.70m.

The 0.1% AEP for current situation and all the future scenarios have been considered as Low confidence.

The tidal water levels have been considered as “High confidence” because the 95%ile uncertainty value in water level for the tidal results is 0.15m which is less than 0.4m.

Flow confidence

The uncertainty in flows for fluvial scenarios is estimated from scores assigned to the hydrological accuracy. These factors are presented in Table 7-4 as it was presented in Section 7.5.1. The confidence in flows has been classified as follows:

- High confidence is described as the Index Flood Method score of equal to 1 (where the gauging station records are long, > 10 years);
- Medium confidence is described as the Index Flood Method score of equal to 2 (where the gauging station records are short, ≤10 years) ; and
- Low confidence is described as the Index Flood Method score of equal to 3 or 4 (where there are no gauging station records and Catchment Descriptors from the Flood Studies Report have been used).

The 0.1% AEP for current situation and all the future scenarios have been considered as Low confidence.

Flows in tables

It has been agreed with the client to present flows in tables for some locations. The criteria for selecting the nodes are listed below:

- The node at the upstream boundary of hydraulic models;
- The node at the centre of each APSR (Area of Potential Significant Risk), and the nodes immediately upstream and downstream of the APSR, (i.e. 3 node locations per APSR);

- The nodes at all the hydrometric gauging stations;
- The nodes upstream and downstream of the confluences of all tributaries that potentially contribute more than 10% of the flow of the main channel; and
- Other nodes at suitable locations to ensure that there is at least one node every 5km along reaches of all modelled rivers.

8. Defence failure scenarios

8.1. Introduction

As part of the FEM FRAM Study, it is required to investigate flood risk and flood hazard due to sudden failure of defences. Additional model runs were carried out to determine the impact of failure of defences at a number of locations along the watercourses.

Sections 8.3 and 8.4 provide information on the results of the defence failure scenario for a number of locations identified in the study area. The information provided focuses on the increase in flood extents and the flood hazard associated with the failure of defences. For technical readers of the report, this chapter should also be read in conjunction with the digital deliverables contained in Volume 3 of the report. This volume contains additional information on water levels, velocities and hazards for the defence failure scenarios.

8.2. Modelling approach

It is likely that a breach would occur when water levels in the river are high. For the defence failure scenario, the breach is induced instantaneously at the peak water level. Two possible modelling approaches were considered as appropriate:

a. 2D Approach - The breach was modelled using the maximum water levels as initial condition of the 2D model. The existing model results (with defences) were used to generate the initial conditions. A breach was setup and the 2D model was rerun.

b. 1D-2D Approach – The 1D-2D link in the current design model was removed along the defence failure area. An ISIS Breach unit was attached to the 1D model along the proposed defence failure location which was linked to the 2D model of the floodplain.

Approach ‘a.’ was used for the Nanny River at Duleek’s Millrace Estate and approach ‘b.’ for the other models. This decision was taken as the Duleek’s Millrace Estate earth embankment is represented by the LiDAR data in the 2D domain (refer to discussion on defences in Section 5.3 and Appendix C3 for further details).

The models have been run for the following AEPs:

- 10% AEP fluvial;
- 1% AEP fluvial; and
- 0.1% AEP fluvial.

The defence failure for the Bracken River in Balbriggan, the Mill Stream in Skerries and the Mayne River at the Coast Road were run using the 10%, 0.5% and 0.1% AEP tidal events as these defences were located in tidally dominated areas.

8.3. River defence failure locations

The river defence failure locations were determined following a review of existing defences, the results of the defence asset survey condition assessment and the properties benefiting from the defences. An initial list was provided to the client for consideration. Table 8-1~~Table 8-1Table 8-1~~ lists the agreed defence failure locations and coordinates within the study area.

Table 8-1 River defence failure locations

N°	River	Defences	Coordinates	
			Easting	Northing
1	Broadmeadow River, Ratoath	Raised defence between 4Ba21573 and 4Ba21100	301420	251606
2	Broadmeadow tributary at Ashbourne	Garden/property walls upstream Brookville Street	306365	252680
3	River Nanny at Duleek	Raised earth embankment on left floodplain at Duleek along the Millrace Estate.	305230	268565
4	Paramadden tributary at Duleek	Concrete wall defence on left bank along Nanny's Paramadden tributary.	304990	268510
5	Bracken River, Balbriggan	Skerries, garden/property walls along the downstream reach LB d/s R132 bridge	320300	263720
6	Mill Stream, Skerries	LB & RB walls u/s of Holmpatrick Road along Millers Lane.	325705	260080
7	Mayne River, Coast Road	Flapped outfall at the downstream extent of the model	323960	241485

8.3.1. Broadmeadow River, Ratoath

The defence failure was setup on the left bank at cross-section 4Ba21228 in Ratoath. Please refer to Section 5.2.1 and Appendix C3 for further details on these defences. The map in Figure 8-1 shows the breach location with the flood outlines for 1% AEP current scenario fluvial event shown in grey and the defence failure scenario shown in blue.

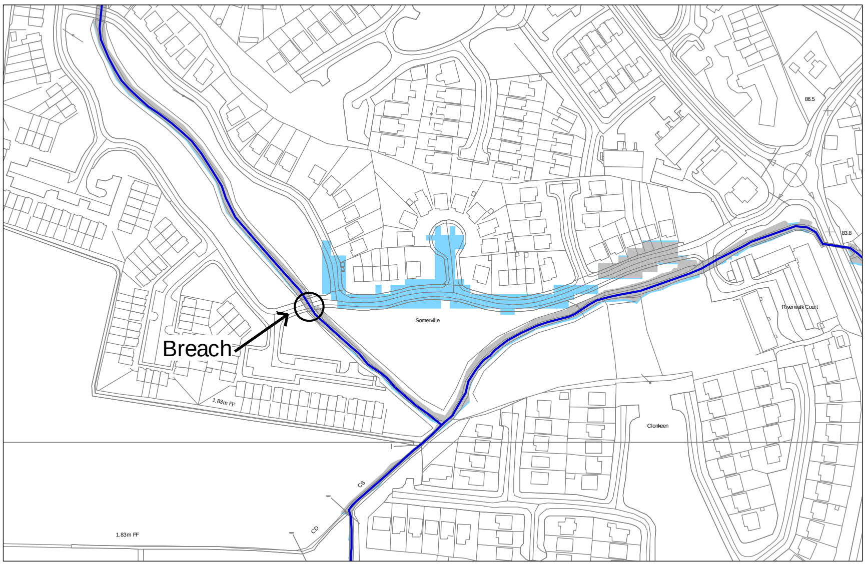

Figure 8-1 Broadmeadow breach location

The flood map indicates that failure of the defences in Ratoath increases the flood risk to a number of properties in the Somerville housing estate. The flood extent in the defence failure scenario is 0.5 hectares more extensive than that in the current scenario for 1% AEP fluvial event.

Figure 8-2~~Figure 8-2~~~~Figure 8-2~~ and Figure 8-3~~Figure 8-3~~~~Figure 8-3~~ show the hazard maps for the current and defence failure scenarios respectively for a 1% AEP fluvial event.

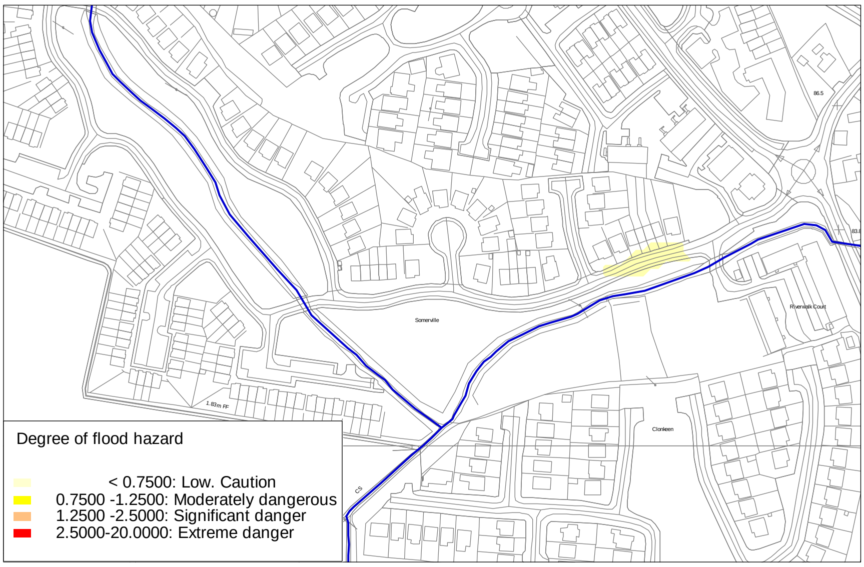

Figure 8-2 Hazard maps for the current scenario

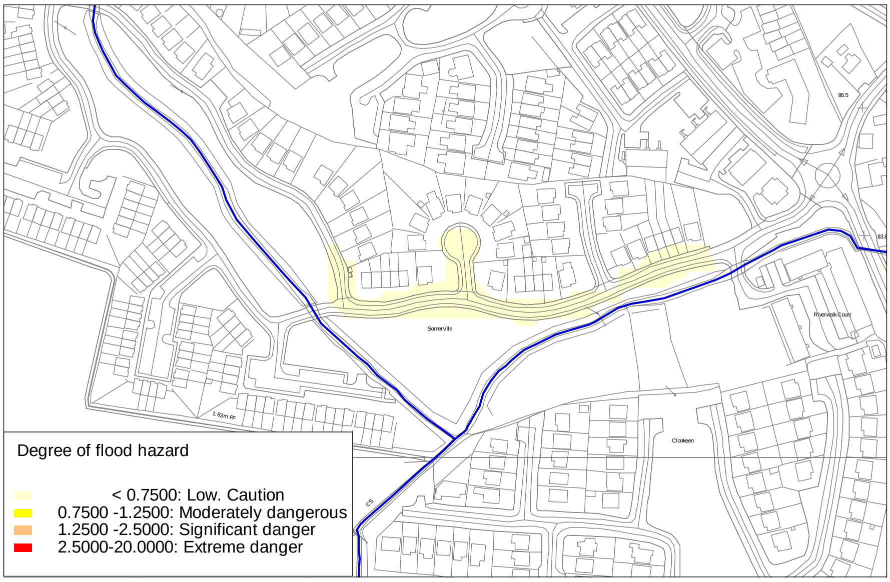

Figure 8-3 Hazard maps for the defence failure scenario

The maximum velocities and depths are of the order of 0.35 m/s and 0.36 m, respectively, and the associated hazard (Table 7-3), would fall under the classification of 'caution' for the 1% AEP fluvial extent.

8.3.2. Broadmeadow tributary at Ashbourne

The defence failure was setup on the right bank at cross-section 4Bau1610 in Ashbourne. Please refer to Section 5.2.1 and Appendix C3 for further details on these defences. The map in Figure 8-4~~Figure 8-4Figure 8-4~~ shows the breach location with the flood outlines for 1% AEP current scenario fluvial event shown in grey and the defence failure scenario shown in blue.

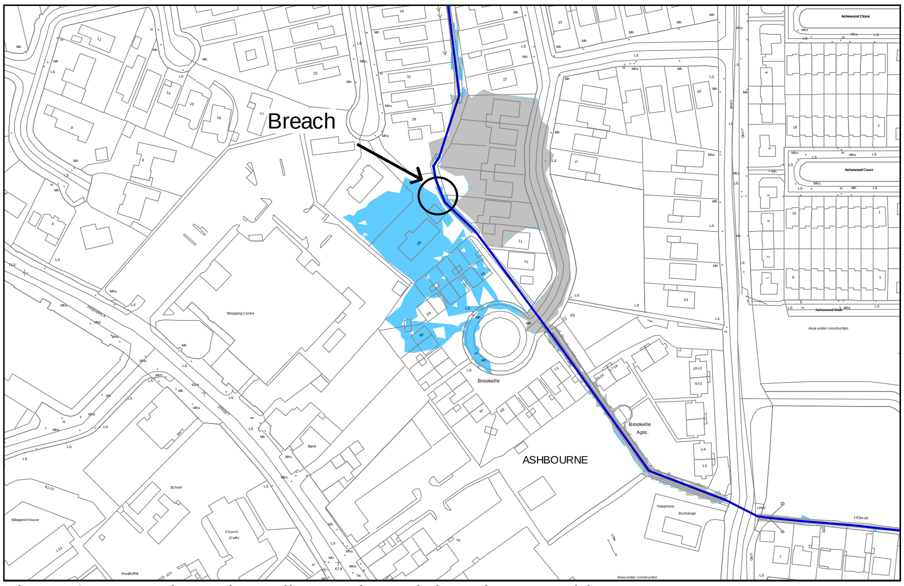

Figure 8-4 Broadmeadow tributary breach location at Ashbourne

The flood map indicates that the failure of the defences in Ashbourne results in an increase in flooding along the left bank of the Bau tributary with a number of additional properties at risk of flooding near Brookville. The flood extent for the defence failure scenario is 0.45 hectares greater than the flood extent for the 1% AEP current scenario fluvial event.

Figure 8-5~~Figure 8-5Figure 8-5~~ and Figure 8-6 show the hazard maps for the current and defence failure scenarios respectively for a 1% AEP fluvial event. Both the velocities and depths never exceed 0.4m/s and 0.6m respectively. Therefore, the Hazard values are never higher than 0.75 and using the Flood Hazard Map Classifications (Table 7-3), the degree of flood hazard would fall under the classification of 'caution' for the 1% AEP fluvial extent.

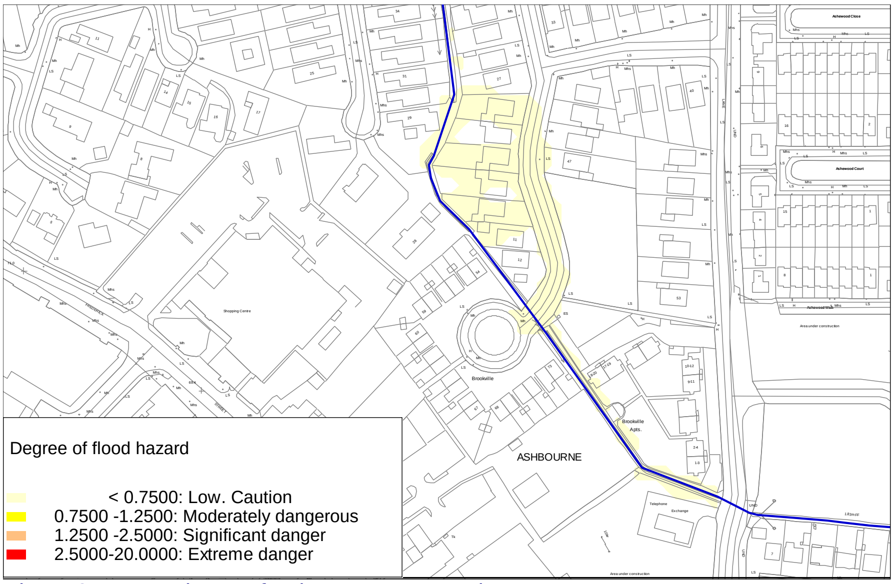

Figure 8-5 Hazard maps for the current scenario

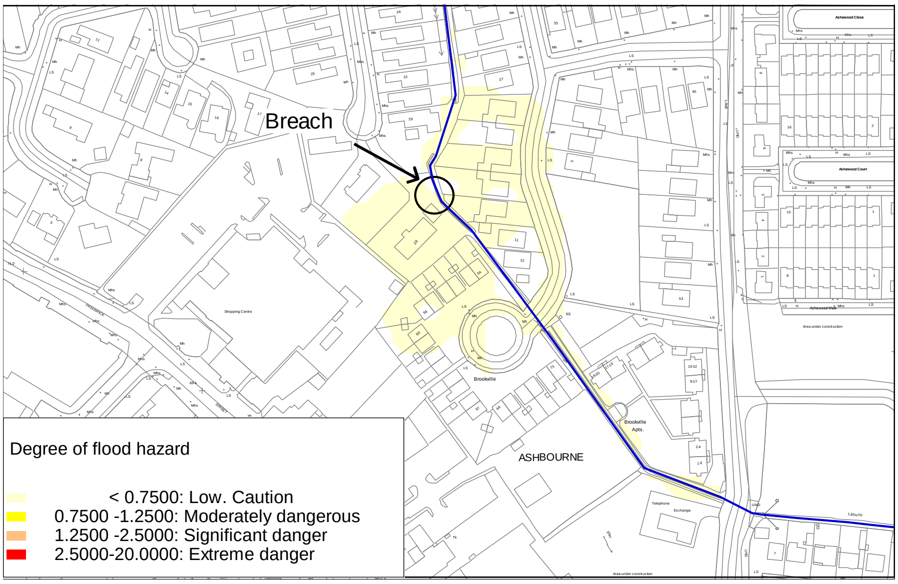

Figure 8-6 Hazard maps for the defence failure scenario

8.3.3. Nanny at Duleek

The breach in the defences along the Nanny River has been set up at the flood embankment along the left floodplain at cross-section 20Na13152 at Duleek's Millrace Estate. . Please refer to Section 5.3 and Appendix C3 for further details on these defences. The map in Figure 8-7 shows the breach location with the flood outlines for 1% AEP current scenario fluvial event shown in grey and the defence failure scenario shown in the colour shaded area.

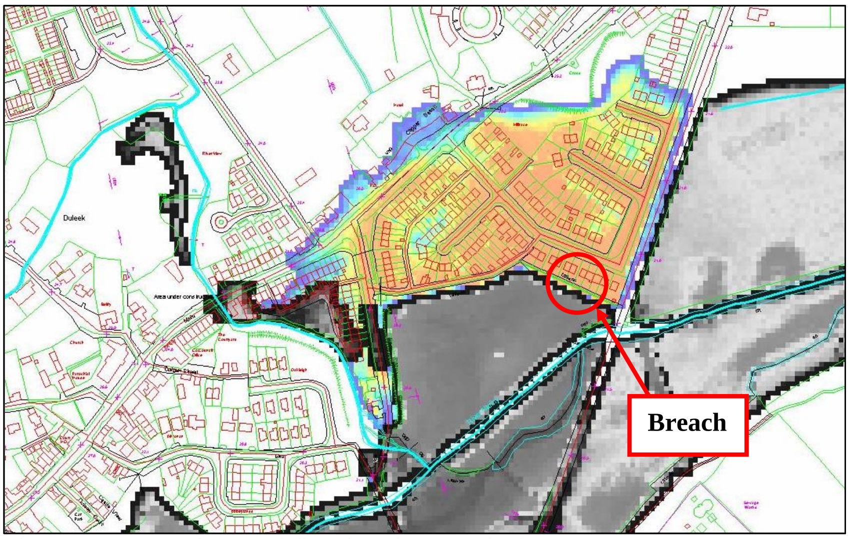

Figure 8-7 Nanny defence failure location at Duleek

The flood extent map indicates that the impact of a defence failure on the Millrace Estate would be significant. Figure 8-8 shows the hazard maps for the current and defence failure scenarios respectively for a 1% AEP fluvial event. Velocities up to 3m/s occur at the location of the failure and up to 1.15 m/s on the streets around the estate, for a 1% AEP fluvial. In terms of water depth, almost the entire estate would be under 1m – 2m of water.

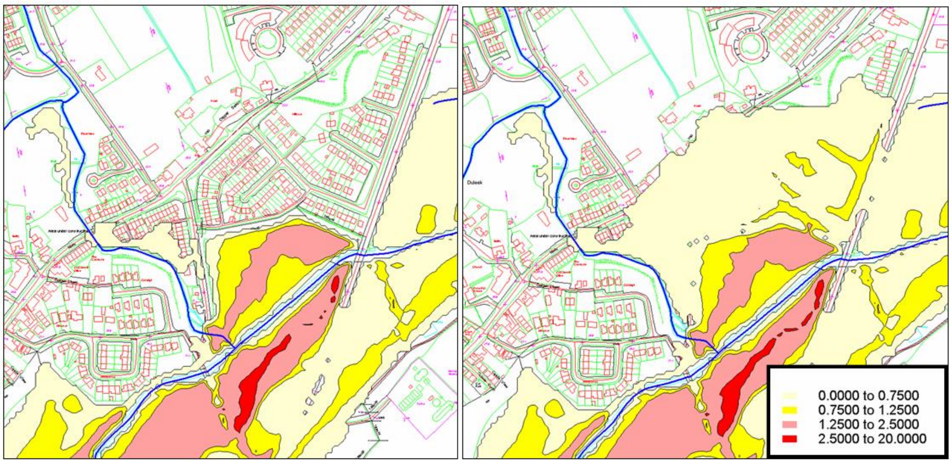

Figure 8-8 Hazard maps for the current and defence failure scenarios

Using the flood hazard map classifications (Table 7-3), the hazard classification in the majority of areas in the Millrace Estate is generally “Low – caution”. The maximum hazard level is 1.5, just next to the breach, which has a hazard classification of ‘significant danger to most people’.

8.3.4. Paramadden at Duleek

The breach has been setup on the Paramadden River in Duleek which is a tributary of the Nanny River. Please refer to Section 5.3 and Appendix C3 for further details on these defences. The breach is located on the left bank defences (flood wall) at cross-section 20Na13152 in Duleek (Millrace Estate). The map in Figure 8-9 shows the breach location with the flood outlines for 1% AEP current scenario fluvial event shown in grey and the defence failure scenario shown in the colour shaded area.

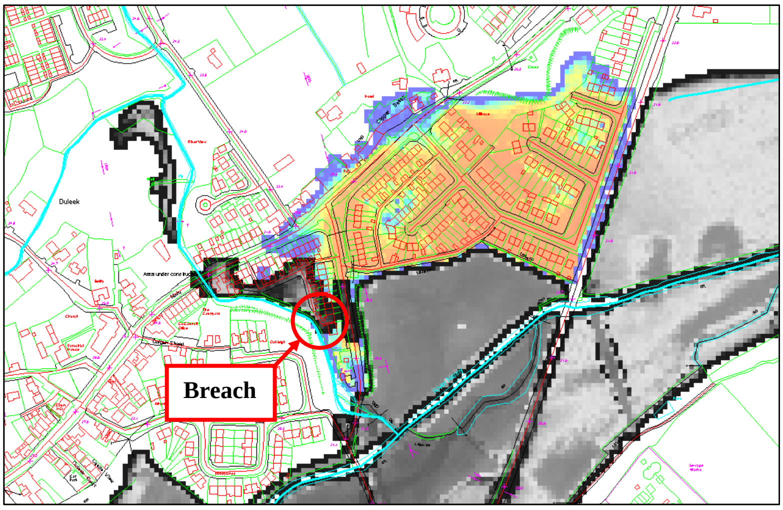

Figure 8-9 Paramadden breach location at Duleek

The flood extent map indicates that the impact of a defence failure on the Millrace Estate would be significant. The map shows that the extent of flooding resulting from the failure of the defences on the Paramadden tributary is similar to the extent of flooding resulting from defence failure on the Nanny River (refer to Figure 8-7). This is because at the location of the defence failure on the Paramadden tributary, the water level is controlled by the levels in the Nanny River (the Nanny River backs up into the downstream end of the Paramadden tributary).

The impact of the failure of the defences in terms of depth and velocity will be slightly less than with a breach occurring along the embankments on the Nanny River (refer to Section 8.3.3~~8.3.38.2.3~~). Velocities would be up to 0.95m/s on the streets around the Millrace Estate, for a 1% AEP fluvial event. In terms of water depth, almost the entire estate would be under 0.8m to 2m deep water.

Figure 8-10 shows the hazard maps for the current and defence failure scenarios respectively for a 1% AEP fluvial event. Using the Flood Hazard Map Classifications (Table 7-3), the hazard classification in the majority of areas in the Millrace Eatate is generally “Low – caution”. The maximum hazard level is 0.9, which has a hazard classification of ‘moderately dangerous for some people’.

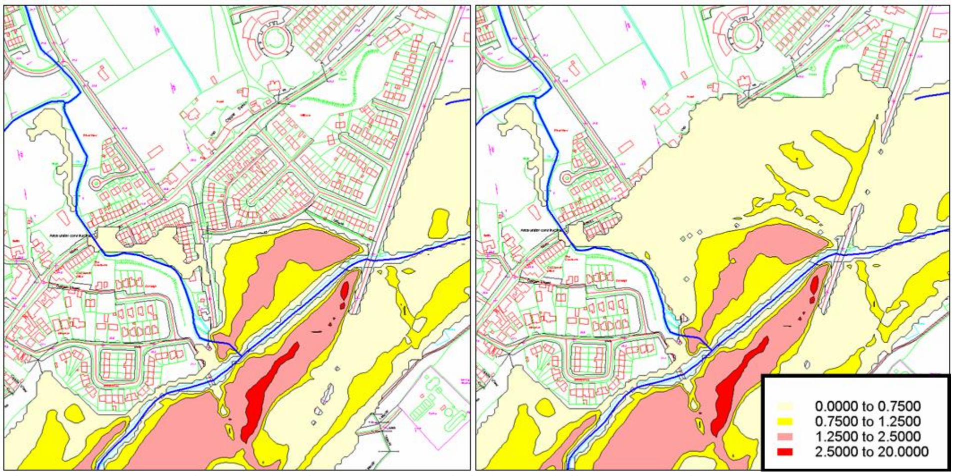

Figure 8-10 Hazard maps for the current and defence failure scenarios

8.3.5. Bracken River

The defence failure has been setup at the flood walls along the left bank of the Bracken River in Balbriggan at cross-section 16Ma217. Please refer to Section 5.12 and Appendix C3 for further details on these defences. The map in Figure 8-11 shows the defence failure location with the flood outlines for 0.5% AEP current scenario fluvial event shown in grey and the defence failure scenario shown in the colour shaded area.

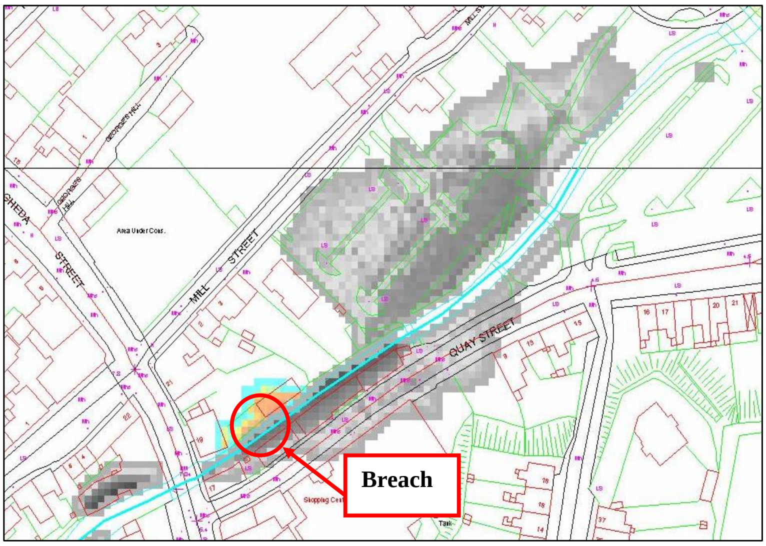

Figure 8-11 Bracken River breach location

Figure 8-11 indicates that the failure of the defences results in a small localised increase in flood extents along the left bank of the river. Figure 8-12 shows the hazard maps for the current and defence failure scenarios respectively for a 0.5% AEP tidal event. Both velocities and depths never exceed 0.1m/s and 0.3m respectively. This results in a maximum hazard value of 0.75 which equates to a hazard classification of ‘Low - caution’ (refer to Table 7-3).

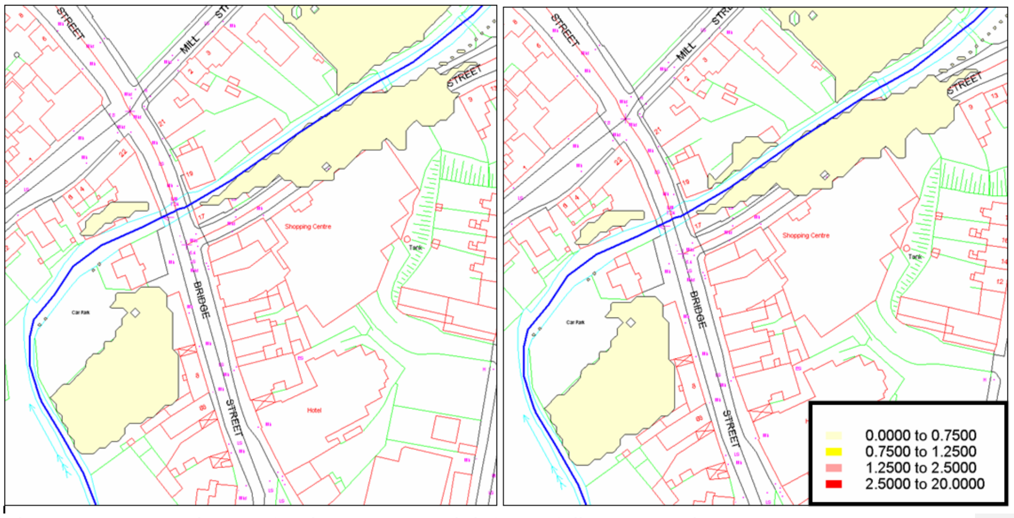

Figure 8-12 Hazard maps for the current and defence failure scenarios

8.3.6. Mill Stream

The defence failure has been set at the flood walls along the left bank of the Mill Stream in Skerries at cross-section 15Ma56D. Please refer to Section 5.13 and Appendix C3 for further details on these defences. The map in Figure 8-13 shows the breach location with the flood outlines for 0.5% AEP current scenario tidal event shown in grey and the defence failure scenario shown in the colour shaded area

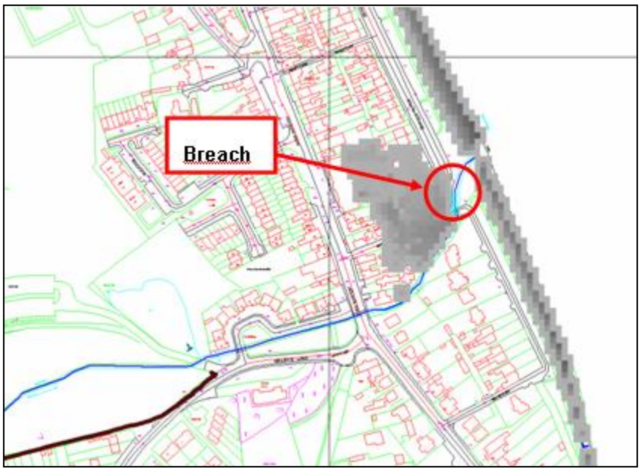

Figure 8-13 Mill Stream breach location

Figure 8-13 shows that the flood extent in both cases is the same. At the location of the defence failure (on left bank), the area is already flooded from flows spilling from upstream. When the defence fails at the maximum tidal level, the water in the floodplain flows back in to the river as the level in the floodplain is higher than the level in the main channel.

Figure 8-14 shows the hazard maps for the current and defence failure scenarios respectively for a 0.5% AEP tidal event. Both velocities and depths never exceed 0.1m/s and 0.3m respectively. This results in a maximum hazard value of 0.75 which equates to a hazard classification of ‘caution’ (refer to Table 7-3).

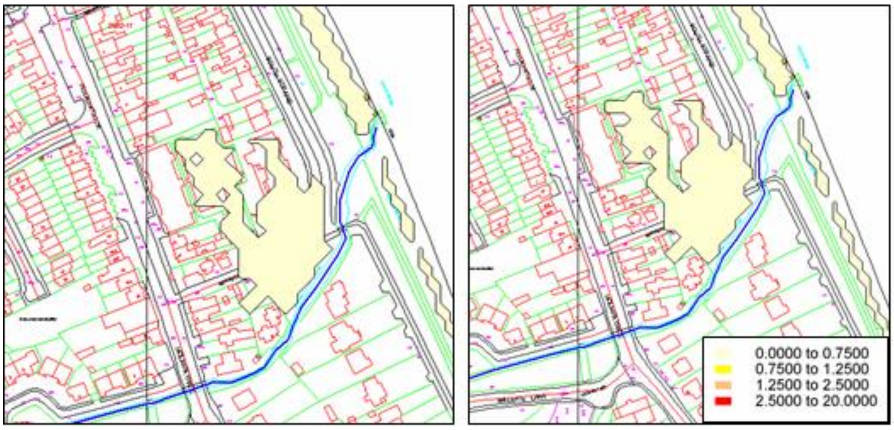

Figure 8-14 Hazard maps for the current and defence failure scenarios

8.3.7. Mayne River, Coast Road

The Mayne River has a flapped outfall that acts as a defence against tidal events. The flapped outfall is located at the downstream end of the model and prevents high tides from propagating upstream at any % AEP event. The Mayne River model was run assuming that the flap valve failed (i.e. open during a tidal event). This assessment is the same as the ‘without defences’ model run described in Section 5.15. The resulting flooding is shown in Figure 8-15~~Figure 8-15Figure 8-15~~ and on the tidal flood extent map, MAY/HPW.EXT/CURS/T/003, as an ‘area benefiting from defences’. The flood extents in Maynestown and Stapolin are increased when the flap valve remains open; however no urban area is affected. The fluvial flood extent map, MAY/HPW/EXT/CURS/003, indicates that the flapped outfall has no affect on the fluvial flood extents.

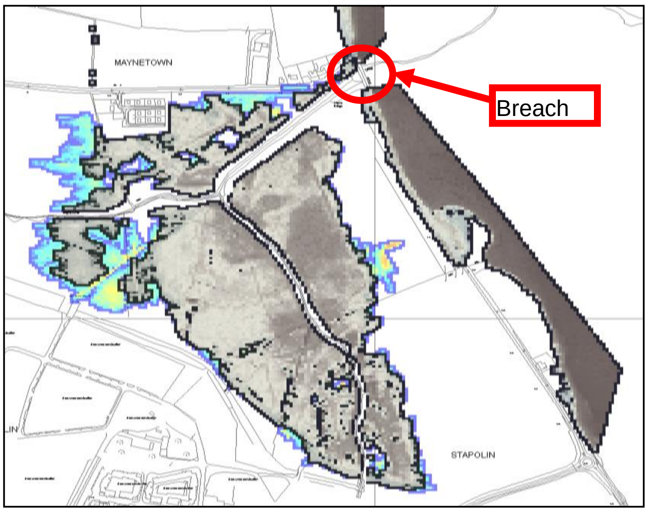

Figure 8-15 Mayne River breach location and flood extent map for the current scenario and defence failure scenario (0.5% AEP tidal event)

Figure 8-16~~Figure 8-16Figure 8-16~~ and Figure 8-17 show the hazard maps for the current and defence failure scenario respectively for a 0.5% AEP tidal event. For the majority of the flooded area the maximum hazard value is 0.75 which equates to a hazard classification of 'caution' (refer to Table 7-3). There are some small pockets of flooding for the defence failure scenario where the hazard values is greater than 0.75 and has a hazard classification of 'moderately dangerous for some people'.

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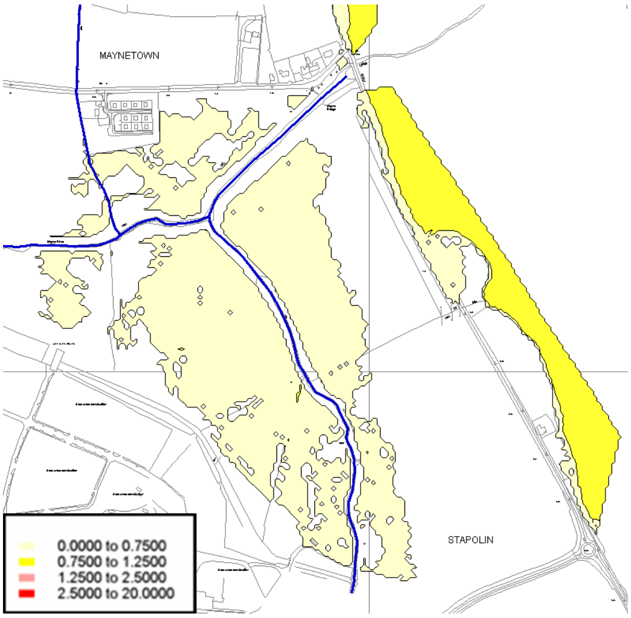

Figure 8-16 Hazard map for the current situation

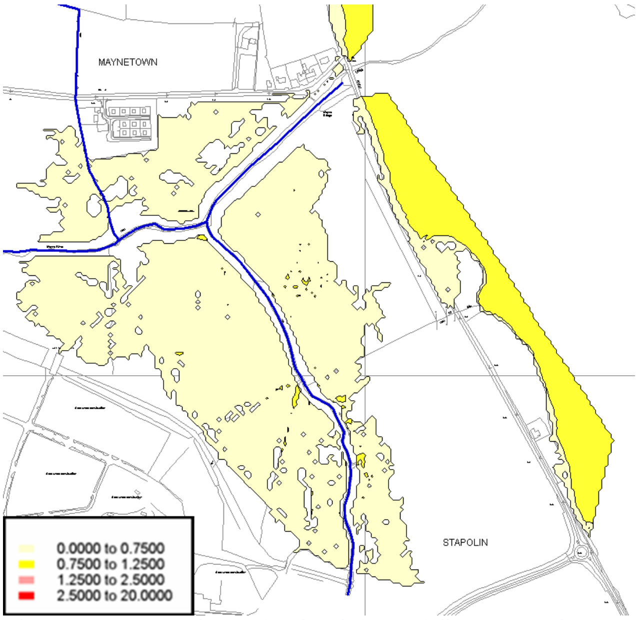

Figure 8-17 Hazard map for the defence failure scenario

8.4. Coastal defence failures

The coastal defence failure locations were determined following a review of existing coastal defences, the results of the defence asset survey condition assessment and a review of the helicopter photographic coastal survey provided by the OPW. An initial list was provided to the client for consideration. It was noted at the time that as the coastline is generally quite high it was difficult to determine the locations of potential defence failure locations and it is anticipated that the extent of flooding as a result of these breaches would be minor. The three locations chosen for coastal defence failure locations are as follows (the locations of these breaches are shown in Figure 8-18);

- Malahide (near Marina Village and Strand Street);
- Rush (Harbour Road area); and
- Bettystown (hotel car park behind Strandview Terrace).

Figure 8-18 Breach locations in coastal defences

The three coastal defence failure scenarios were modelled by adding breaches to the ISIS 2D coastal model. The breaches were modelled by adding a polyline shapefile to the model with a polyline to represent each breach. The breach polylines are drawn perpendicular to the defence line and the fields in the shapefile 'Height1' and 'Height2' are used to control the ground level at the start and end of the breach line to be modelled. The polylines added to the model result in a breach of one model cell width, i.e. 20m wide, which was exactly the breach width specified for the breach modelling. A wider breach could be modelled if required by using a polygon to define the breach.

The breaches are specified under the topography grid in the topography section for the Domains tab in the XML control file. The items in this list are read in sequence so when the breach shapefile is specified after the DTM the breaches are imprinted into the DTM. The functioning of the breaches can then be checked by looking at the 'chk.zmod.asc' grid.

Due to the very limited nature of coastal flooding and the high level of natural ground along the coast there was no significant additional flooding as a result of breaching the defences in these locations. None of the modelled breaches resulted in any flooding, for the reasons outlined below:

Breach 1 (Malahide)

- Ground level behind breach: ~3.45m on spit (and 8m and rising on land off the spit to the south). Refer to

Figure 8-19

- ~~Figure 8-19~~Figure 8-19 for further details;
- Modelled breach level: ~3.4m;
- 1% AEP water level: ~2.6m; and
- 0.1% AEP water level: ~3m.

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Figure 8-19 Breach location in Malahide

No flooding occurs as a result of the breach as the 1% and 0.1% AEP water levels are not high enough to cause flooding. The spit may be inundated as a result of the HEFS 0.5% AEP event but land south of spit is too high to flood.

Breach 2 (Rush)

- Ground level behind breach: ~5.2m (and rising on land behind). Refer to

Figure 8-20

- ~~Figure 8-20Figure 8-20~~ for further details;
- Modelled breach level: ~5.2m;
- 1% AEP water level: ~3.3m; and
- 0.1% AEP water level: ~3.6m.

No flooding occurs as a result of this breach as the land behind the breach location is very high and higher than the 0.1% AEP water level.

Figure 8-20 Breach location in Rush

Breach 3 (Bettystown)

- Ground level behind breach: ~5m @ 80m behind peak defence level and ~3.5/2.5m @ 200m behind peak defence level (refer to

Figure *8-21*

- ~~Figure *8-21*Figure 8-21~~ for further details);

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- Modelled breach level: ~5m (currently have ~80m long breach);
- 100yr water level: ~3.4m; and
- 1000yr water level: ~3.7m.

Figure 8-21 Breach location in Bettystown

No flooding occurs as a result of this breach as the land behind the breach location is very high and higher than the 0.1% AEP water level.

8.5. Conclusion

To summarise, the river defence failure with the most significant impact are the ones located in Duleek and Ashbourne. Both breaches on the Paramadden tributary and along the main channel of the River Nanny would cause severe flooding of the Millrace Estate in Duleek. The highest velocities and depths are present with the Nanny's left bank failure but distribution of velocities is completely different between the two scenarios which make both dangerous in terms of the hazard classification.

Defence failures on both the Bracken River and Mill Stream have a very small localised impact for the 0.5% AEP tidal event. The effect of the defence failure on the Mill Stream is negligible and the impact in the Bracken River in terms of flood extent and flood hazard is not significant. The Mayne river flap valve was removed and the model rerun. Additional localised flooding was encountered as shown on the flood extent maps.

Due to the very limited nature of coastal flooding and the high level of natural ground along the coast there was no additional flooding as a result of breaching the coastal defences in the three locations specified.

9. Risk of blockage of structures

9.1. Introduction

A blockage scenario was undertaken to determinate the impact of structure blockage in the locations detailed below. This analysis was undertaken for 0.1%, 1% and 10% AEP events. The 10% AEP event provides an indication of the blockage effect for smaller and more frequent floods. The 1% and 0.1% AEP events provide an indication of the incremental effect that the blockage would have during a major flood.

The blockage assessments were carried out for two blockage scenarios; 30% and 70% blockage of the opening of the structure. The blockages were modelled by modifying the structures as follows:

- Orifice - the area of the inlet was reduced;
- Culvert - the diameter of the culvert was reduced;
- Sluice - the width of the opening was reduced; and
- Bridge - the width of the opening was reduced.

Section 9.2 provides information on the results of the blockage assessment for a number of locations identified in the study area. The information provided focuses on the increase in flood extents and the flood hazard associated with the blockage of culverts. For technical readers of the report, this chapter should also be read in conjunction with the digital deliverables contained in Volume 3 of the report. This volume contains additional information on water levels, velocities and hazards for the full range of events and blockage scenarios assessed.

9.2. Blockage locations

The locations for undertaking blockage scenarios were selected following review of the modelling results to determine which structures had potential for becoming blocked and where the consequence of any blockage could be significant. This list was then discussed and agreed with the client. Table 9-1 provides a summary of the culvert blockage locations with further details of the impact of these culvert blockages reported on in Sections 9.2.1~~9.2.1~~~~9.1.1~~ to 9.2.19~~9.2.19~~~~9.1.19~~.

Table 9-1 Locations of culvert blockage assessment

Model	Details of blockage location	Type of structure	Cross section location (label)
MAY	Mayne River at Swords Road	Culvert	1Ma7268
SLU	Cuckoo Stream (Mac tributary of the Mayne River) at Wellfield Bridge	Culvert	1Mac258
SLU	Sluice River at Portmarnock Trotting Track	Culvert	2Sa2300
WAR	Ward River at Balheary Road Bridge and Balheary Bridge	Culvert and bridge	4Wa102 and 4Wa953

Model	Details of blockage location	Type of structure	Cross section location (label)
BRO	Broadmeadow River at Moulden Bridge	Bridge	4Ba19220
BRO	Broadmeadow River at Ashbourne Bridge	Bridge	4Ba15420
BRO	Broadmeadow River at Robertstown Bridge	Bridge	4Ba12867
BRO	Broadmeadow River at Warblestown Bridge	Bridge	4Ba5770
BRO	Broadmeadow Tributary in Ashbourne	Culvert	4Bau2326
BAL	Ballyboghil at Ballyboghil Bridge	Bridge	7Ba7547U
COR	Corduff at R132	Culvert	8Ca1129
BAY	Baleally	Culvert	9Ba3030
RWS	Rush West at Channel Road	Culvert	11Wa267
MIL	Mill Stream at Holmpatrick Road, Skerries	Bridge	15Ma222
BRA	Bracken River at Decoy Bridge	Culvert	16Ma5361
BRA	Bracken River at R132 Bridge	Bridge	16Ma244
MOS	Mosney at Mosney Road	Culvert	19Maa548
NAN	River Nanny, Paramadden tributary at Bridge Street	Bridge	20Nag63
BSS	Brookside Stream at Laytown Road Bridge	Culvert	21Ma63

9.2.1. Mayne River at Swords Road

The Mayne River passes through a long culvert where it crosses the Swords Road (R132). This culvert (at node 1Ma7268) is 342m long and is of concrete construction. The inlet and outlet shapes are rectangular and circular respectively, so the conduit changes section at some point along its length. At the inlet of this culvert, there is a trash screen that can be easily blocked by debris.

Figure 9-1 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for 1% AEP fluvial event. The maps indicate that the blockage of this culvert results in a sizeable increase in the flood extent. For a 1% AEP event, water levels at the culvert inlet rise significantly with a 30% and 70% culvert blockage. The backup of water in the channel forces flows out of bank and into the Collinstown Business Park

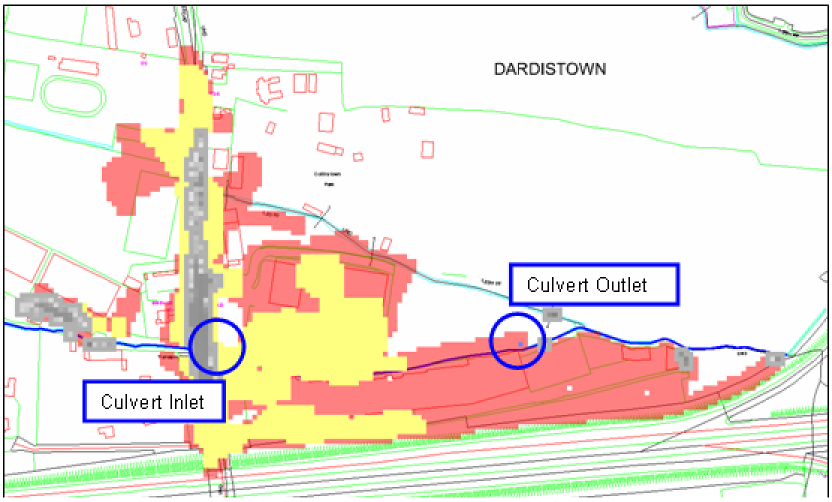

Figure 9-1 Mayne River flood extents at Swords Road for the current scenario, 30% blockage scenario and 70% blockage scenario

Figure 9-2 shows the hazard maps for the current scenario and the two blockage scenarios for a 1% AEP fluvial event.

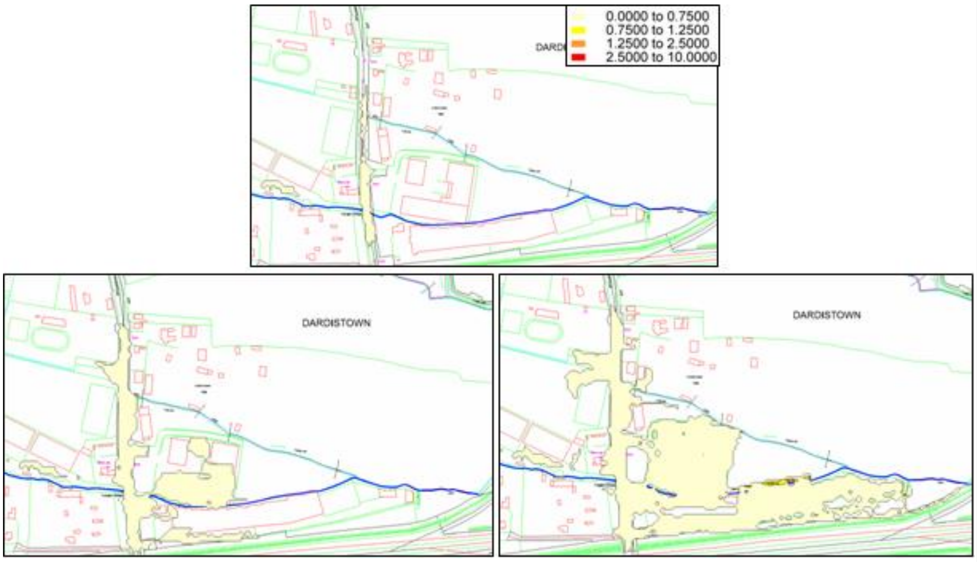

Figure 9-2 Mayne River hazard maps at Swords Road for the current scenario (top), 30% blockage scenario (bottom left) and 70% blockage scenario (bottom right)

For the 30% blockage scenario, the maximum depths and velocities are over the Swords Road. These depths and velocities never exceed 0.3m and 0.3m/s respectively. Based on the flood hazard classification (refer to Table 7-3), this equates to a hazard classification of “low - caution” for the whole 1% AEP fluvial extent.

Generally, for the 70% blockage scenario, the hazard values never exceed 0.75. The depths and velocities on the Swords Road are 0.4m and 0.35m/s respectively. The maximum hazard is located just upstream of the culvert outlet where the water come back into the river. At this location the hazard values vary between 1 and 1.30 (with 1.25m and 0.90m/s maximum depths and velocities respectively). Based on the flood hazard classification this equates to a hazard classification between “moderately dangerous for some people” and “significantly dangerous for most people”.

9.2.2. Cuckoo Stream (Mac tributary of the Mayne River) at Wellfield Bridge

The Cuckoo Stream passes through a long culvert where it crosses Wellfield Bridge (node 1Mac258). This culvert is 119m long and is of concrete construction. There is an arch conduit at the inlet and two rectangular conduits at the outlet. The conduit separates in two just downstream from the inlet. The channel has a lot of bushes along the embankments so a partial blockage of the culvert is possible due to debris carried by the flow. Figure 9-3 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for 1% AEP fluvial event

There is a small increase in the extent of flooding at Balgriffin Park, however, the predominant flow path is in an easterly direction with a more sizeable increase in flooding at Snugborough just upstream the railway embankment. Some properties beside Mayne Road are at risk of flooding for both blockage scenarios.

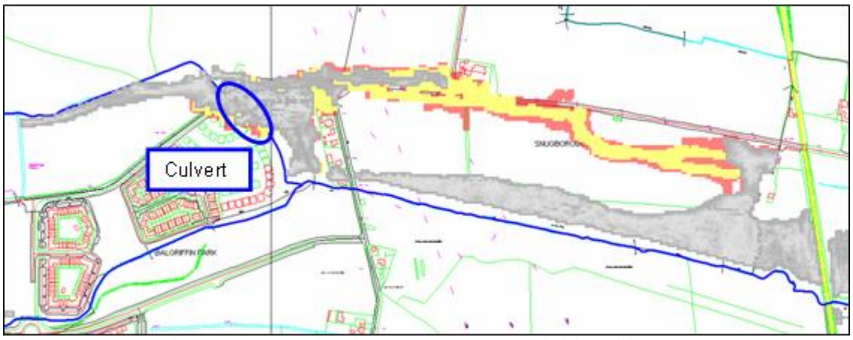

Figure 9-3 Cuckoo Stream flood extents at Wellfield Bridge for current, 30% and 70% blockage scenarios

Figure 9-4 shows hazard maps for the current scenario and the two blockage scenarios for a 1% AEP fluvial event.

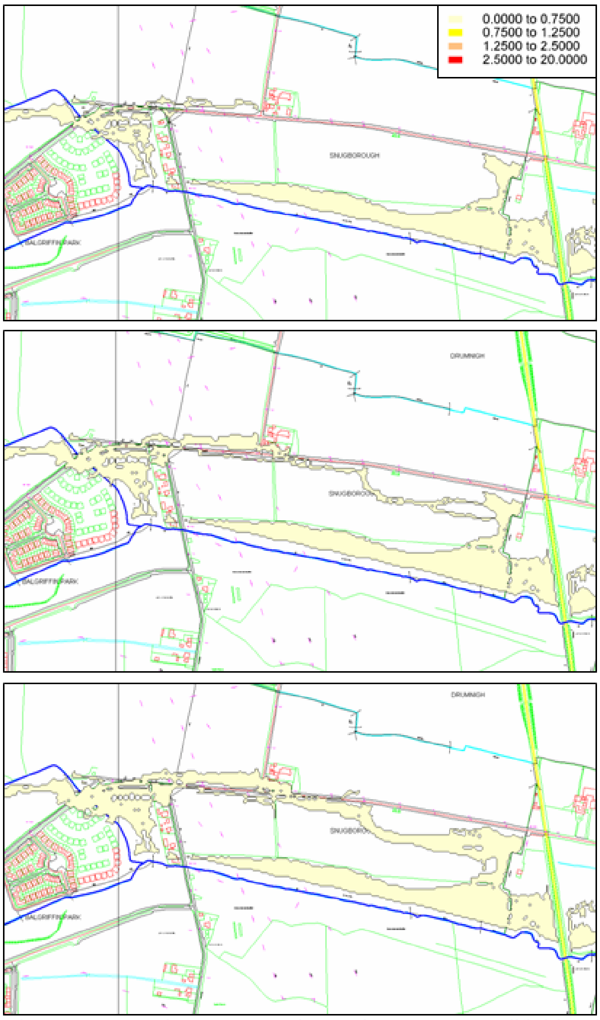

Figure 9-4 Cuckoo Stream hazard maps at Wellfield Bridge for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

The hazard values for the current scenario and two blockage scenarios are always lower than 0.75. The maximum hazard values are 0.25 and 0.35 for 30% and 70% blockage scenarios respectively. Therefore, a hazard classification of “caution” applies to all areas for all scenarios. The highest velocities are present where the water bypasses the culvert and flows in an easterly direction to the railway embankment. The maximum values are 0.55m/s, 0.85m/s and 1.1m/s for the current scenario, 30% blockage scenario and 70% blockage scenario respectively. The maximum depths values are 0.4m and 0.55m for 30% and 70% blockage scenarios respectively.

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9.2.3. Sluice River at Portmarnock Trotting Track

The Sluice River passes through a culvert where it crosses Portmarnock trotting track (near node 2Sa2300). This culvert is 39m long and is constructed of concrete. The inlet and outlet shapes are rectangular. The channel has a lot of scrub and bushes along the channel banks. A partial blockage of the culvert is possible due to debris carried by the flow.

Figure 9-5 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for 1% AEP fluvial event. The map shows that there isn't much difference between the current scenario and the 30% blockage scenario. For the 70% blockage scenario there is a sizeable increase in the flood extent with flood waters extending to the trotting track.

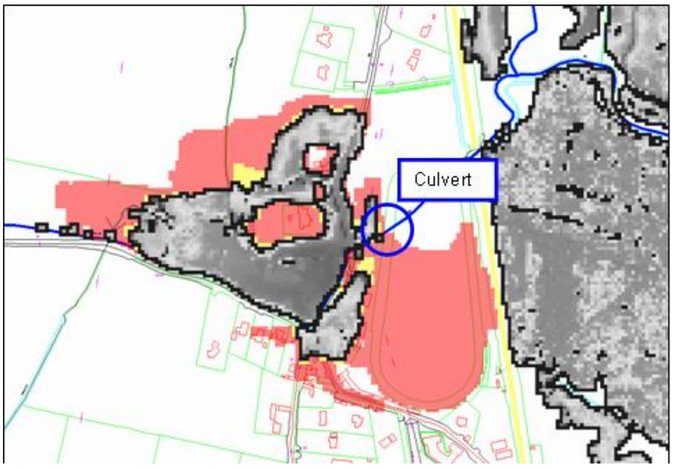

Figure 9-5 Sluice River flood extents at Portmarnock trotting track for current, 30% and 70% blockage scenarios

Figure 9-6 shows the hazard maps for the current scenario and two blockage scenarios respectively for a 1% AEP fluvial event.

For the current and the 30% blockage scenarios, most of the area has a hazard value less than 0.75 and the hazard classification is 'caution'. However, in certain locations near the main river channel upstream of the trotting track, the hazard value varies between 0.5 and 1.1 with maximum velocities of 0.5m/s. In these locations the hazard classification is "moderately dangerous for some people" (refer to Table 7-3).

For the 70% blockage scenario the water flows into the trotting track where the hazard is always lower than 0.75 (hazard classification is 'caution'). Upstream of the track, the hazard values vary between 0.5 and 1.5 with maximum velocities of 0.75m/s. A sizeable area of floodplain at this location has a hazard classification of 'moderately dangerous for some people' with some areas alongside the river classified as 'significant danger for most people'.

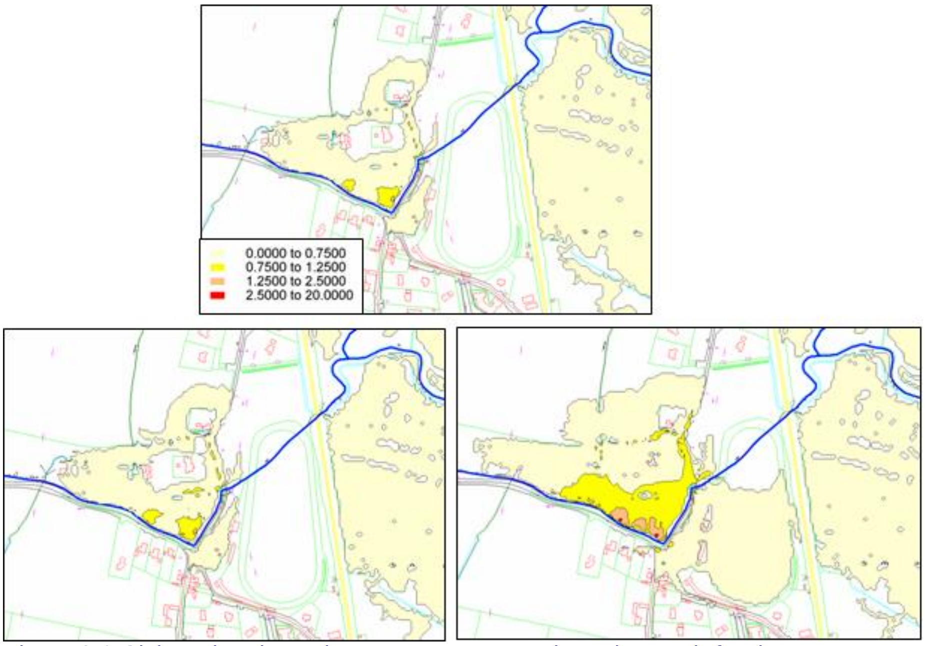

Figure 9-6 Sluice River hazard maps at Portmarnock trotting track for the current scenario (top), 30% blockage scenario (Bottom left) and 70% blockage scenario (bottom right)

9.2.4. Ward River at Balheary Road Bridge and Balheary Bridge

The following culverts were identified as potential culvert blockage locations along the Ward River:

- a 45m concrete culvert on the Ward River at cross section 4Wa102; and
- a bridge at cross section 4Wa953 (blockage risk is increased due to overhanging pipes.

The blockages at both culvert locations were modelled together in the same model. Figure 9-7 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded orange) for 1% AEP fluvial event when both culverts are modelled with blockages together. The map shows that there isn't much difference between the current scenario and the 30% blockage scenario. For the 70% blockage scenario there is a sizeable increase in the flood extent in Swords town centre.

Figure 9-8~~Figure 9-8~~Figure 9-8, Figure 9-9 and Figure 9-10 show the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario respectively for a 1% AEP fluvial event. The maps show that degree of flood hazard is classified as "caution" except in a small area at the confluence with the Broadmeadow River which has a hazard classification of 'moderately dangerous for some people'.

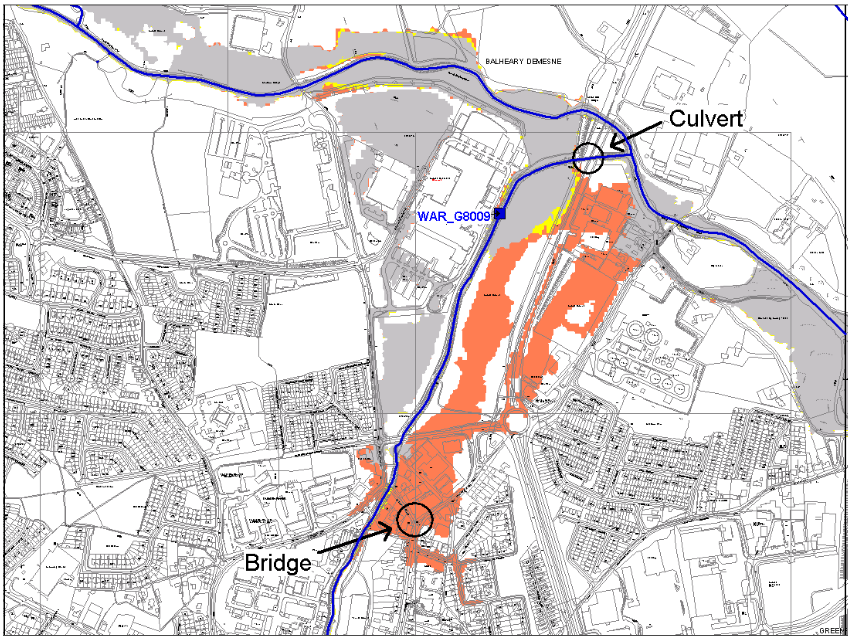

Figure 9-7 Ward River flood extents at Balheary Road Bridge for current scenario, 30% blockage scenario and 70% blockage scenario

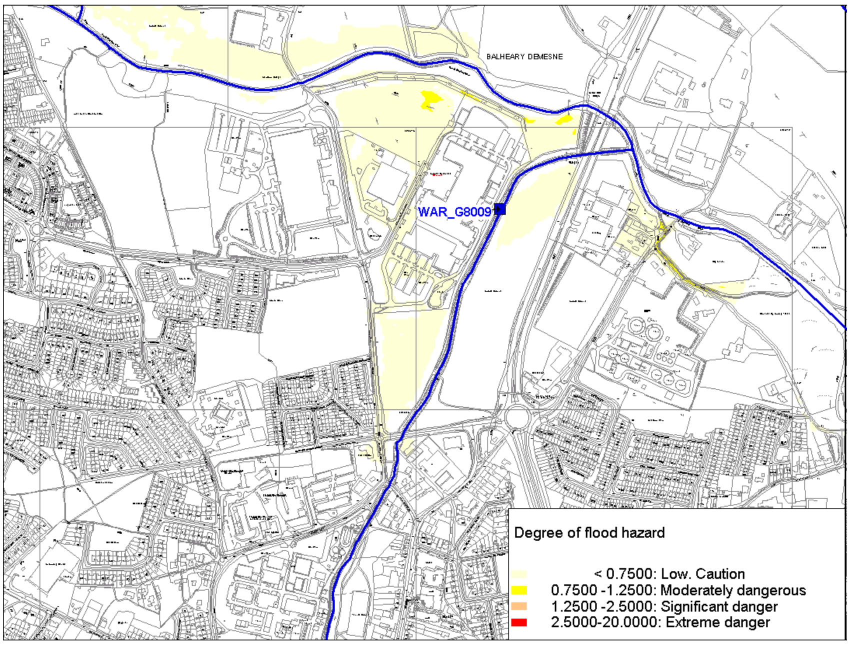

Figure 9-8 Ward River hazard map for the current scenario

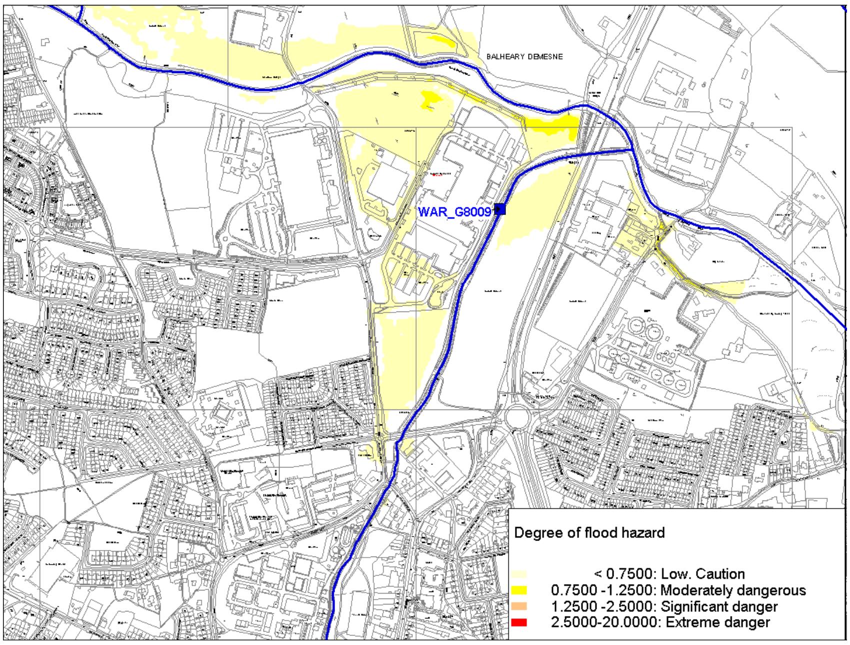

Figure 9-9 Ward River hazard map for the 30% blockage scenario

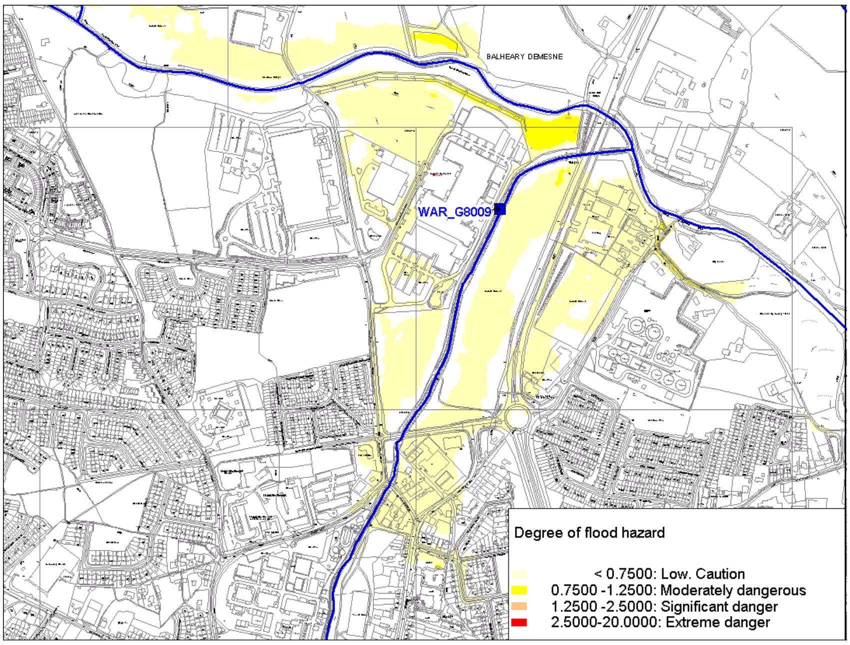

Figure 9-10 Ward River hazard map for the 70% blockage scenario

9.2.5. Broadmeadow River at Moulden Bridge

The main Broadmeadow channel passes through a stone bridge at node 4Ba19220 (Moulden Bridge in Ratoath). The soffit level of the bridge is relatively low so the opening can be blocked by debris carried by the flow.

Figure 9-11 shows the flood extents for the current scenario (shaded grey) and the 30% culvert blockage (shaded yellow). Figure 9-12 shows the flood extents for the current scenario (shaded grey) and the 70% culvert blockage (shaded orange). Both maps are for the 1% AEP fluvial event. The maps show that the most significant increase in flood risk is to agricultural land downstream of Moulden Bridge. Both the 30% and 70% blockage scenarios result in similar flood extents.

Figure 9-13, Figure 9-14 and Figure 9-15 show the hazard maps for the current scenario and two blockage scenarios for a 1% AEP fluvial event and show that degree of flood hazard is always less than 1.25 which has a hazard classification of 'moderately dangerous for some people'.

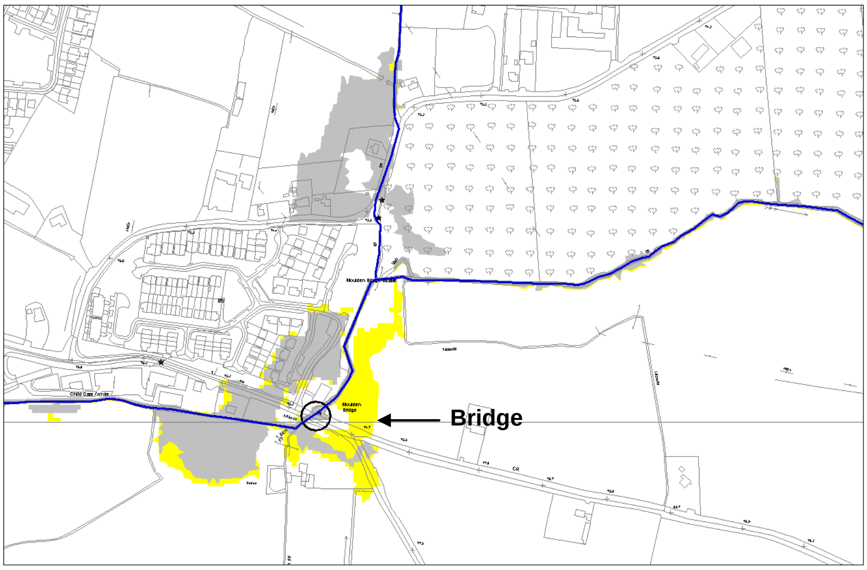

Figure 9-11 Broadmeadow River flood extents at Moulden Bridge for current scenario and 30% blockage scenario

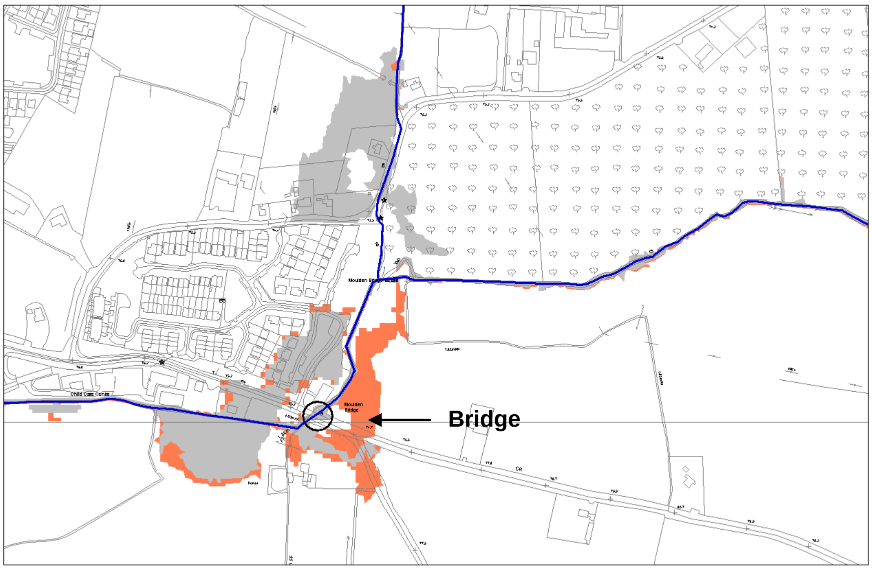

Figure 9-12 Broadmeadow River flood extents at Moulden Bridge for current scenario and 70% blockage scenario

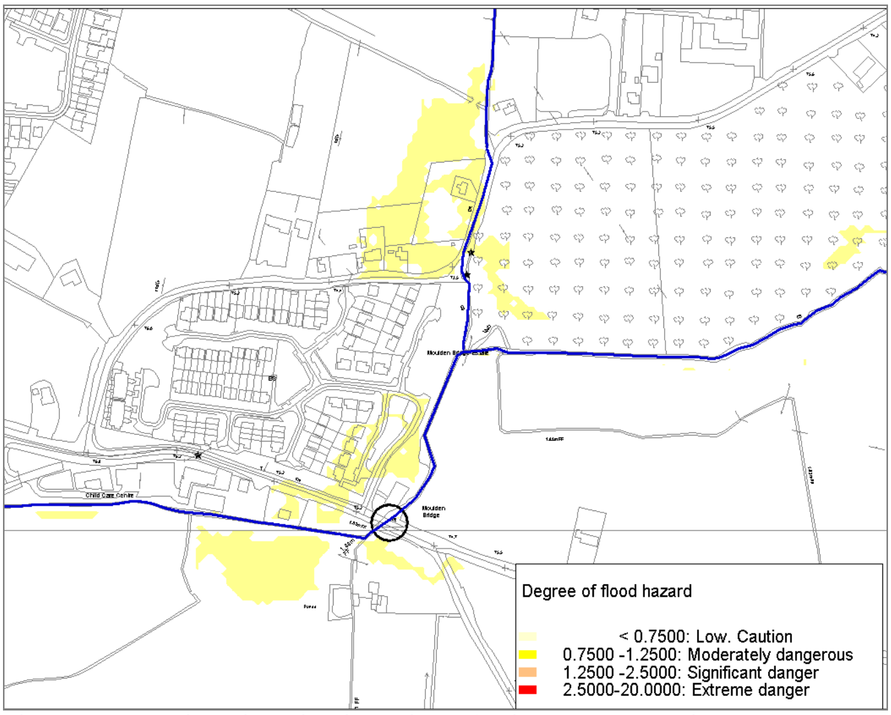

Figure 9-13 Broadmeadow River hazard map for the current scenario

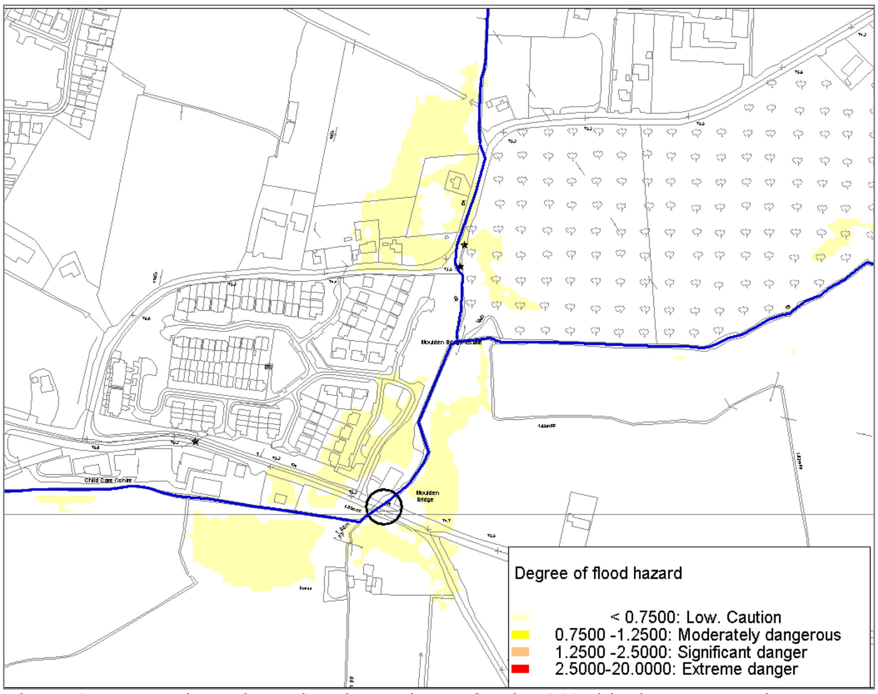

Figure 9-14 Broadmeadow River hazard map for the 30% blockage scenario

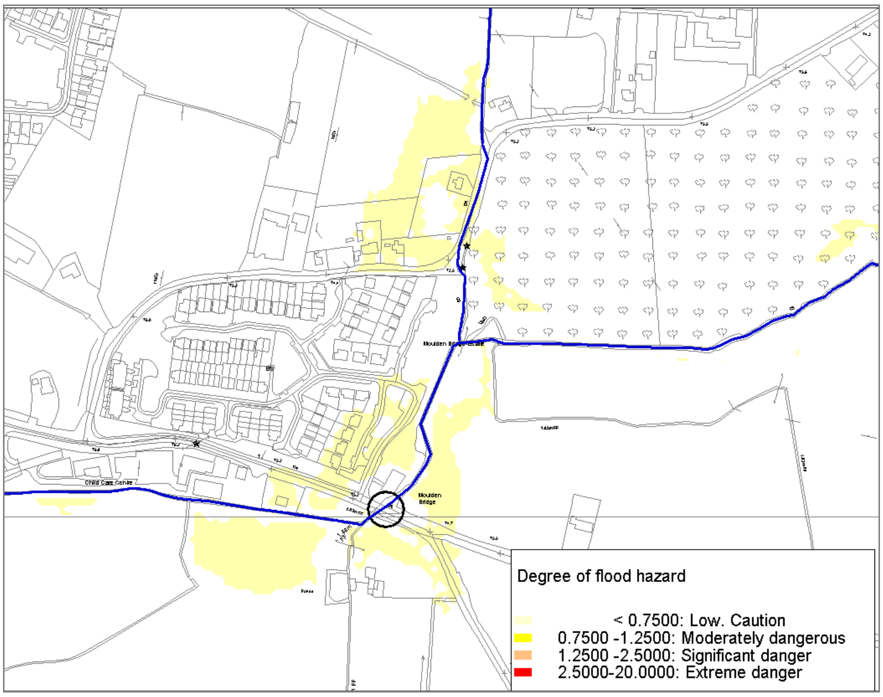

Figure 9-15 Broadmeadow River hazard map for the 70% blockage scenario

9.2.6. Broadmeadow River at Ashbourne Bridge

The main Broadmeadow River channel passes through a stone bridge where it crosses Bridge Street in Ashbourne (node 4Ba15420).

Figure 9-16 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded orange) for 1% AEP fluvial event. The map shows that there isn't much difference between the current scenario and the blockage scenarios with minor increases in flooding upstream of Ashbourne Bridge. Due to the limited increase in flooding, no assessment of the flood hazard was carried out.

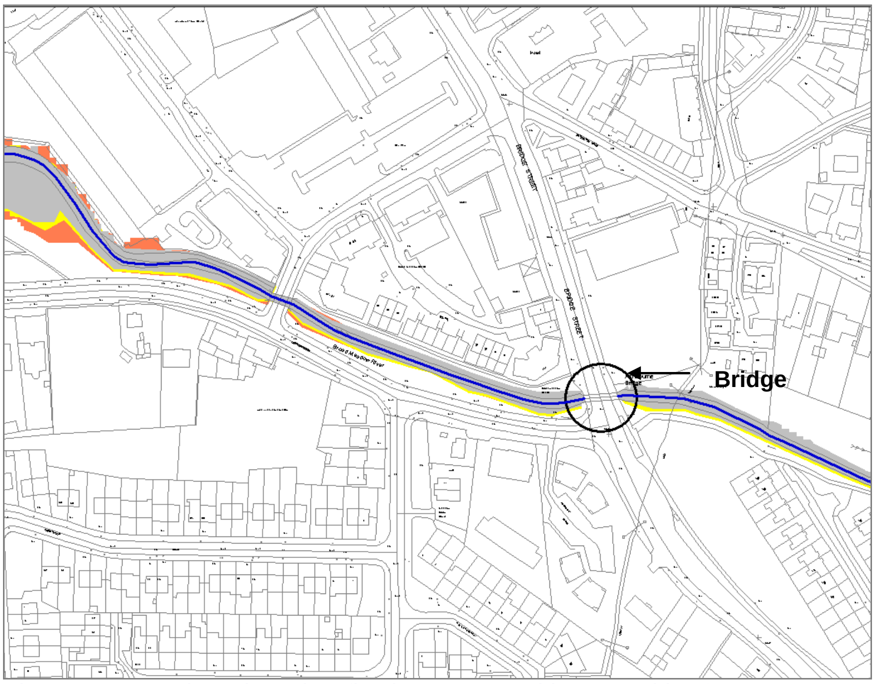

Figure 9-16 Broadmeadow River flood extents at Ashbourne Bridge for current scenario, 30% blockage scenario and 70% blockage scenario

9.2.7. Broadmeadow River at Robertstown Bridge

The main Broadmeadow River channel passes under Robertstown Bridge at node 4Ba12867. A blockage assessment was undertaken to determinate the impact of debris blocking this structure..

Figure 9-17 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded orange) for a 1% AEP fluvial event. The map shows that there is a negligible difference between the current scenario and the blockage scenarios. Because of the limited increase in flooding, no assessment of the flood hazard was carried out.

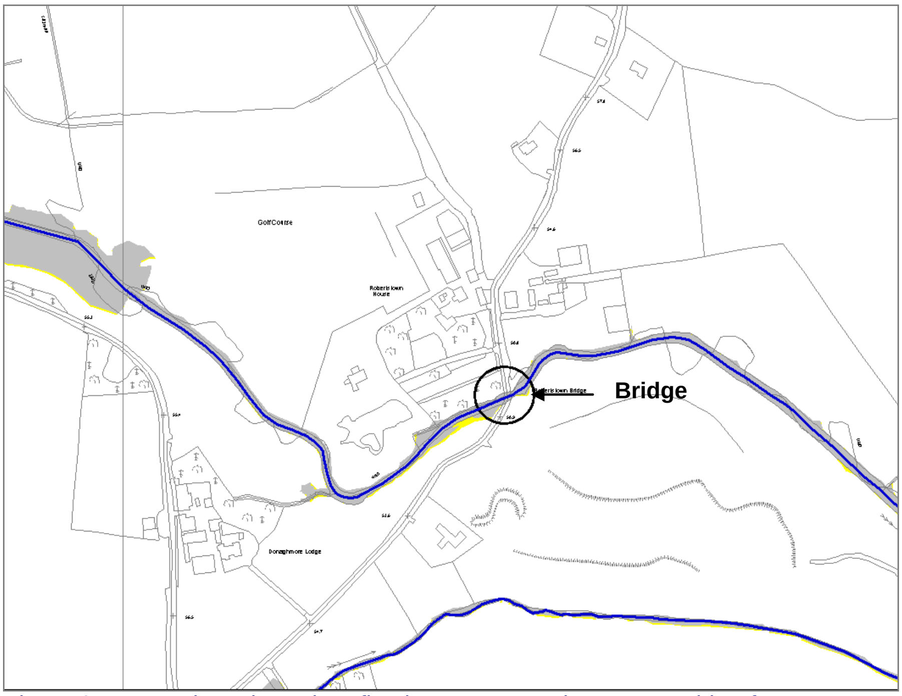

Figure 9-17 Broadmeadow River flood extents at Robertstown Bridge for current scenario, 30% blockage scenario and 70% blockage scenario

9.2.8. Broadmeadow River at Warblestown Bridge

The Broadmeadow River passes through Warblestown Bridge which lies between Lispopple and Roganstown, at cross section 4Ba5770. The opening of this bridge can be blocked by debris carried by the river.

Figure 9-18 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded orange) for 1% AEP fluvial event. The map indicates that there is a minimal increase in flood extents between the current scenario and the 30% blockage scenario. However, there is a sizeable increase in flood extents for the 70% blockage scenario with a number of properties at risk of flooding.

Figure 9-19, Figure 9-20 and Figure 9-21 show the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario for a 1% AEP fluvial event. The maps show that the degree of flood hazard is always less than 1.25 which has a hazard classification of ‘moderately dangerous for some people’.

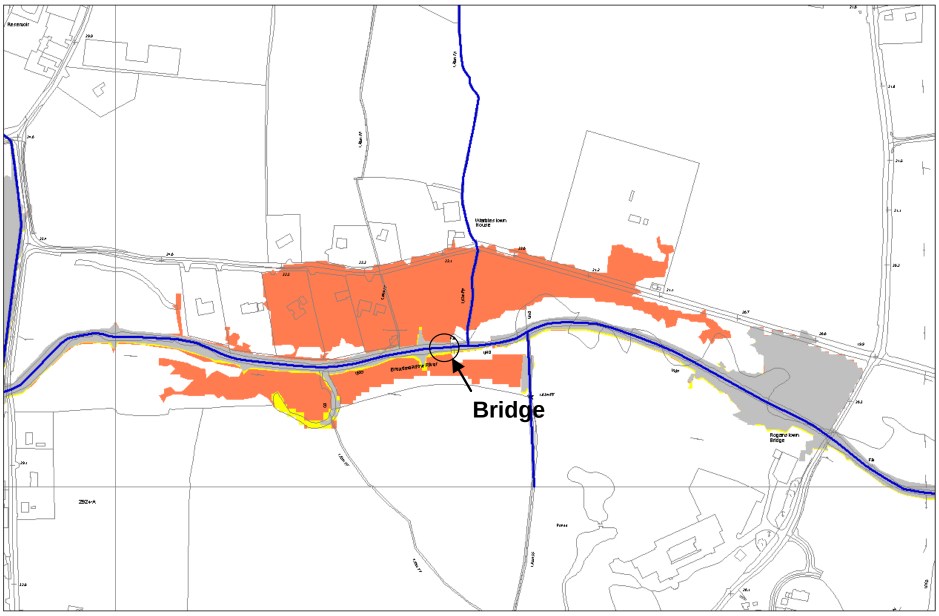

Figure 9-18 Broadmeadow River flood extents at Warblestown Bridge for current scenario, 30% blockage scenario and 70% blockage scenario

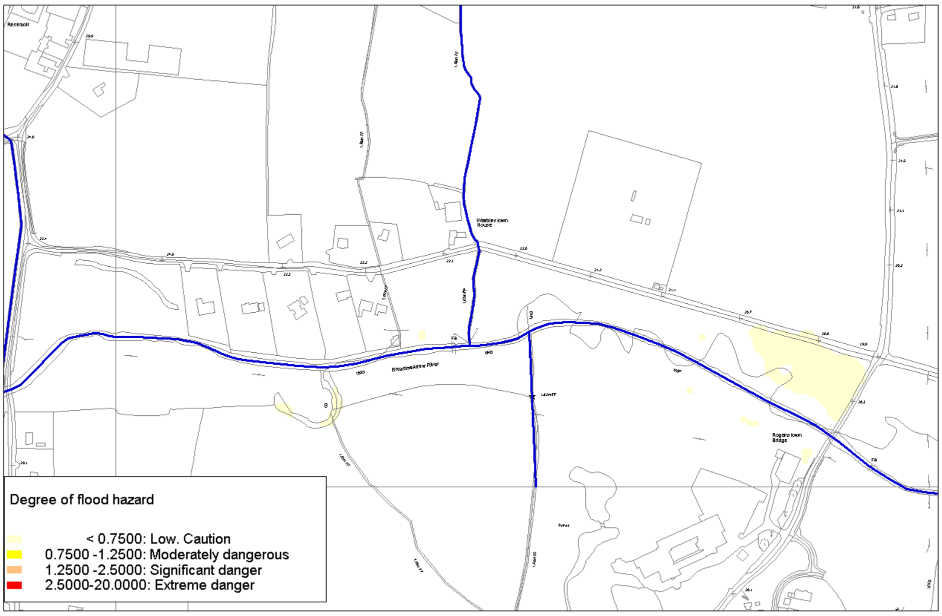

Figure 9-19 Broadmeadow River hazard map for the current scenario

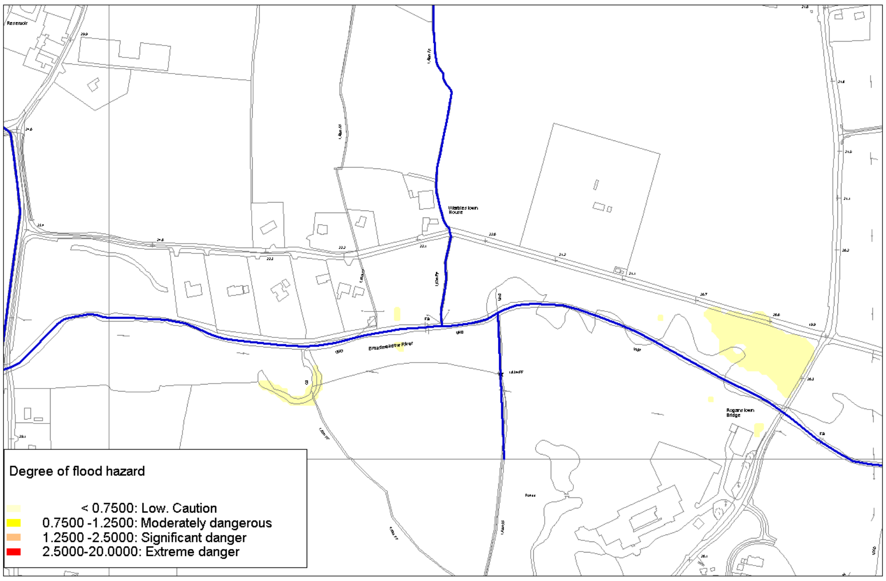

Figure 9-20 Broadmeadow River hazard map for the 30% blockage scenario

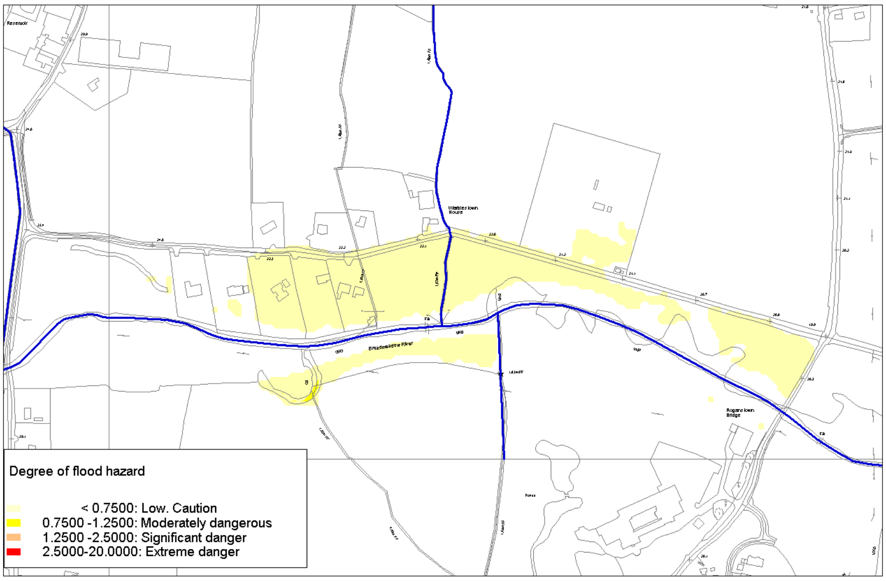

Figure 9-21 Broadmeadow River hazard map for the 70% blockage scenario

9.2.9. Broadmeadow tributary in Ashbourne

The Broadmeadow tributary in Ashbourne passes through a 65m culvert constructed of concrete. The inlet to the culvert is located at cross section 4Bau2326 and lies within the Ashbourne urban area.

Figure 9-22 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded orange) for 1% AEP fluvial event. The map indicates that there is a minimal increase in flood extents between the current scenario and the 30% blockage scenario. However, there is a sizeable increase in flood extents for the 70% blockage scenario with a number of additional properties at risk of flooding.

Figure 9-23, Figure 9-24 and Figure 9-25 show the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario for a 1% AEP fluvial event. The degree of flood hazard is in all cases less than 0.75 which has a hazard classification of 'caution'.

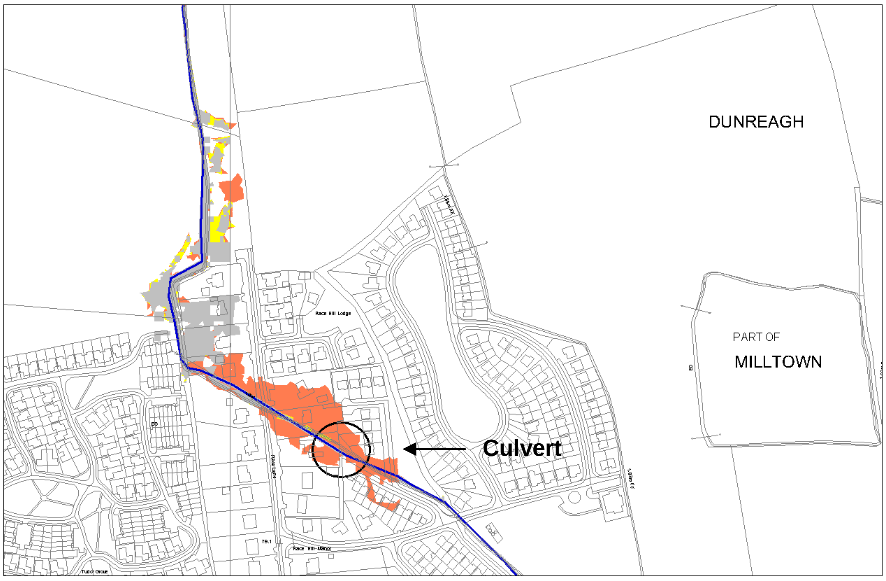

Figure 9-22 Broadmeadow tributary flood extents at Ashbourne for current scenario, 30% blockage scenario and 70% blockage scenario

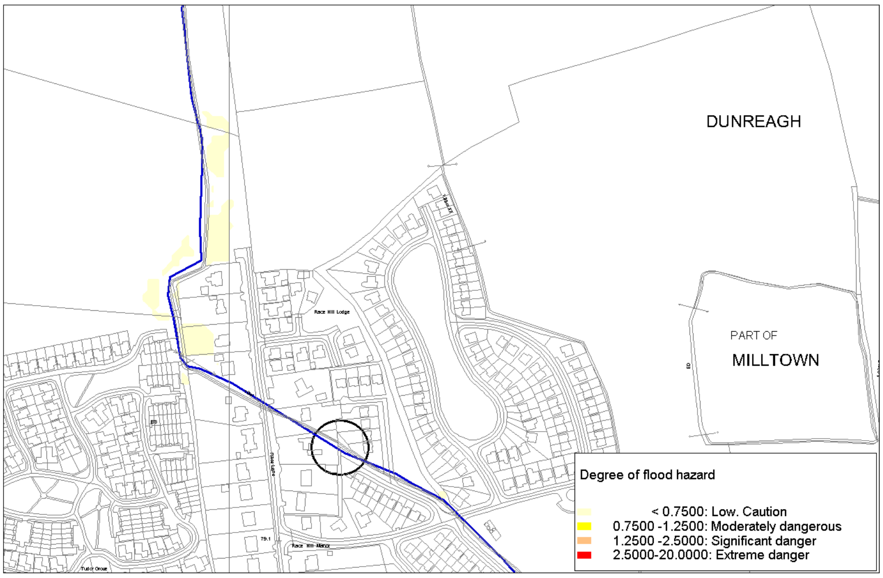

Figure 9-23 Broadmeadow tributary hazard map for the current scenario

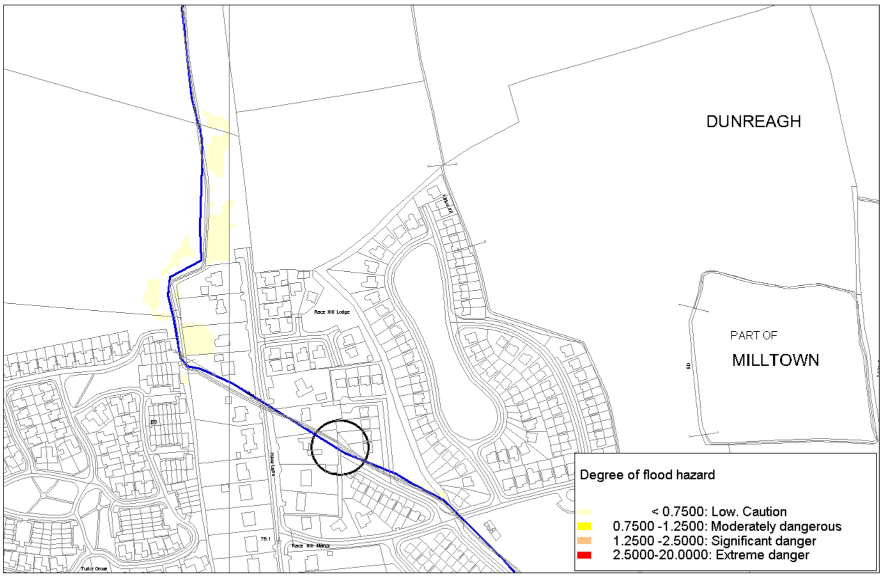

Figure 9-24 Broadmeadow tributary hazard map for 30% blockage scenario

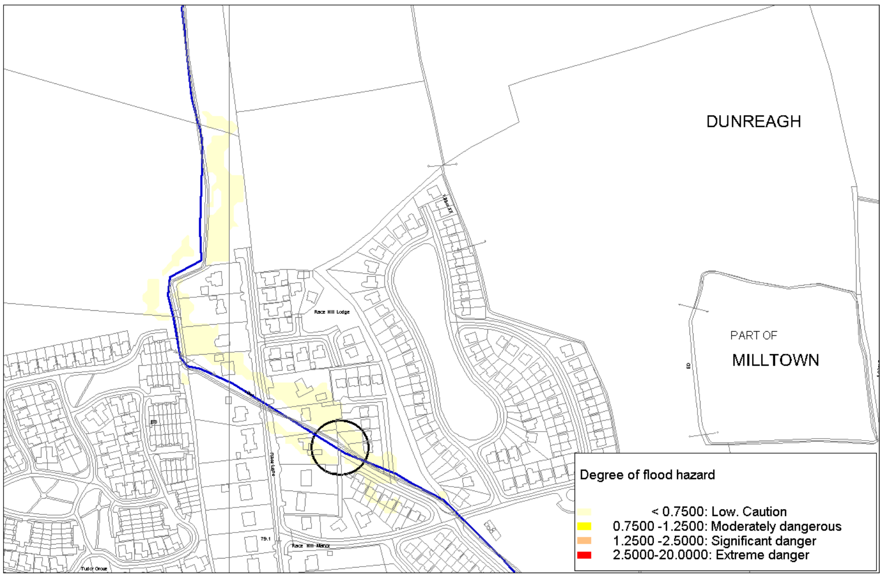

Figure 9-25 Broadmeadow tributary hazard map for 70% blockage scenario

9.2.10. Ballyboghil at Ballyboghil Bridge

The main channel passes through the 6.7m long Ballyboghil Bridge. It is constructed of concrete and has two openings separated by a vertical pier.

The photographs of the upstream face of the bridge (node 7Ba7547U) indicate that there is a fallen tree that is partially blocking the bridge. Debris will quickly build up against this obstruction. The water level differences between the current scenario and the two blockage scenarios are 0.15m and 0.50m respectively upstream of Ballyboghil Bridge. Downstream of the bridge, these differences do not exceed 0.15m.

Figure 9-26 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for 1% AEP fluvial event. The map indicates that there is an incremental increase in flood extents between the 30% blockage scenario and 70% blockage scenario. Oldtown Road on the right bank acts as a spillway. When a blockage occurs, more flow overtops the road and into the right bank floodplain. With this flow path, the bigger the % blockage, the higher the velocities. The increase in flooding to this floodplain places a number of properties at risk of flooding in Ballyboghil village.

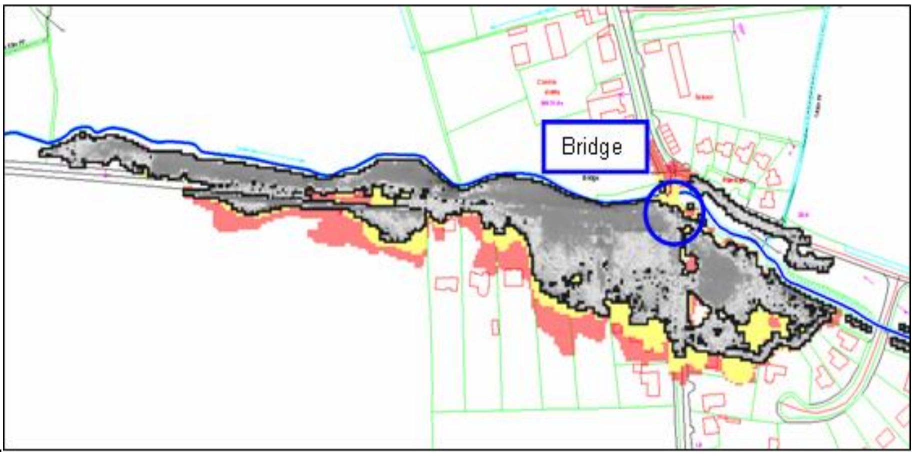

Figure 9-26 Ballyboghil River flood extents at Ballyboghil Bridge for current, 30% and 70% blockage scenarios

The maximum velocities when the water flows over the Oldtown Road are 1.2m/s approximately for current and 30% blockage scenarios. Over the Naul Road, the highest velocities are produced for the 70% blockage scenario and have values of 1.5m/s approximately. For the current scenario and both 30% and 70% blockage scenarios, the hazard values are generally lower than 0.75 in the majority of areas and therefore a hazard classification of “caution” applies. However, some isolated pockets of flooding have average hazard values that vary between 0.25 and 0.80 with maximum hazard values close to 1.30 for the 30% and 70% blockage scenarios. A hazard value of 1.30 is classified as “significant danger for most people”. Figure 9-27 shows the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenarios for a 1% AEP fluvial event.

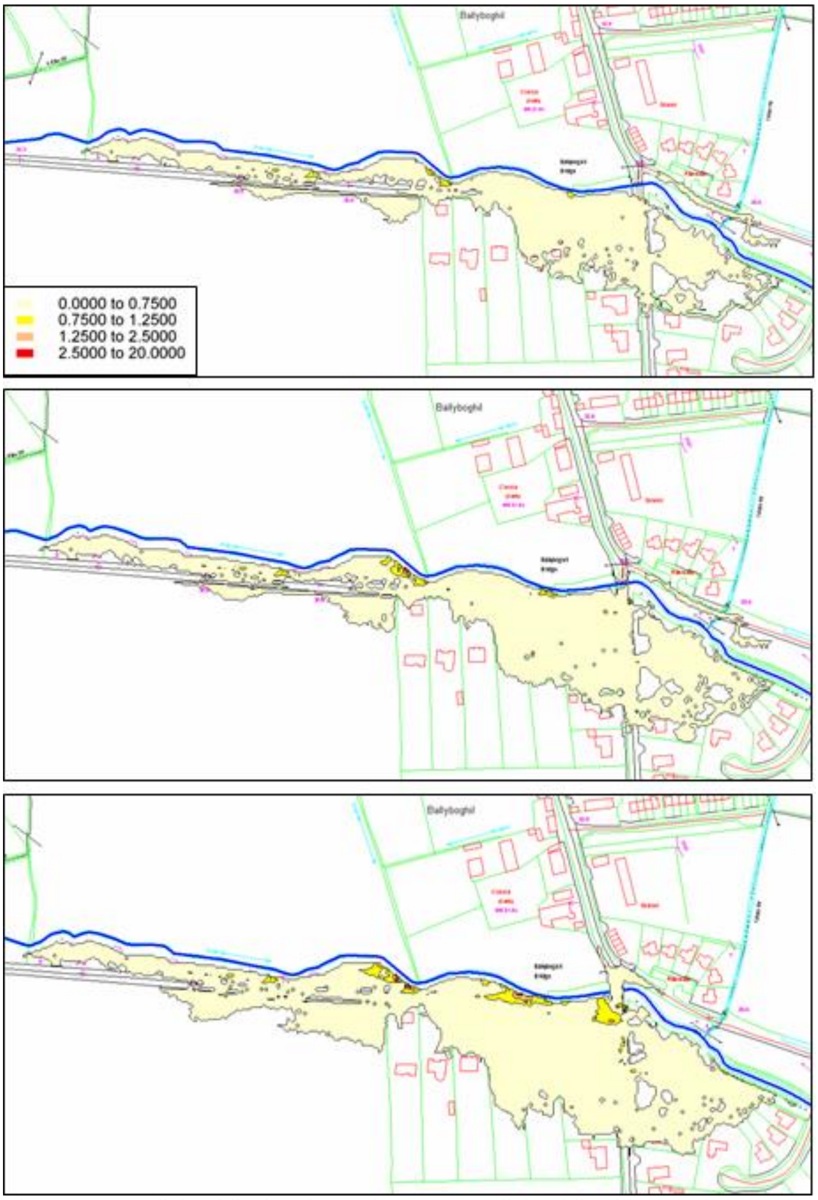

Figure 9-27 Ballyboghil River hazard maps at Ballyboghil Bridge for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.11. Corduff at R132

The main channel of the Corduff River passes through a 19.4m culvert where it crosses the R132 (Old N1) at node 8Ca1129. This culvert is constructed of concrete with an arched shaped inlet and outlet. There also is a bypass under the R132 crossroads that has been modelled with a blockage similar to the main opening.

Figure 9-28 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for 1% AEP fluvial event. The map indicates that there is an incremental increase in flood extents between the 30% blockage scenario and 70% blockage scenario. The 30% blockage scenario results in an increased flood risk to at least one additional property near the bridge while the 70% blockage scenario increases flood risk to at least two additional properties.

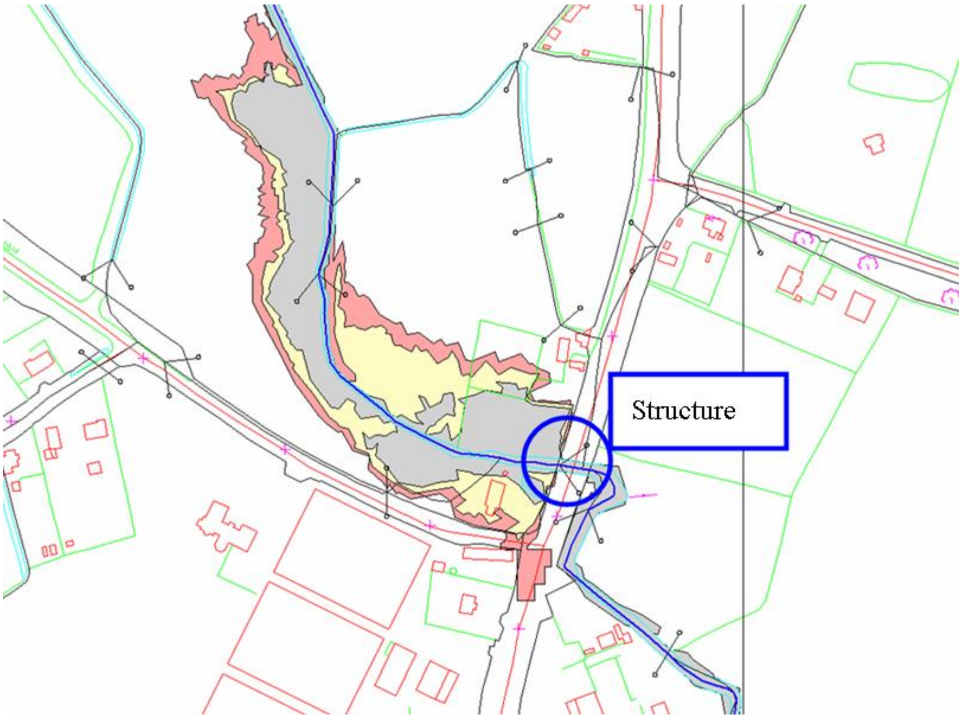

Figure 9-28 Corduff River flood extents at R132 for current (grey), 30% (yellow) and 70% (red) blockage scenarios

Figure 9-29 shows the hazard map for the current scenario and the 30% and 70% blockage scenarios for a 1% AEP fluvial event. For the current scenario hazard values in the flood plain are lower than 0.75 and therefore a hazard classification of “caution” applies (higher hazard values are shown in the channel). The increased flood risk due to the 30% and 70% blockage of the bridge opening results in some increased hazard in the floodplain with a hazard of ‘moderately dangerous’ and ‘significant danger’ occurring in some areas upstream of the bridge for the 70% blockage scenario. This increased hazard is also located close to properties.

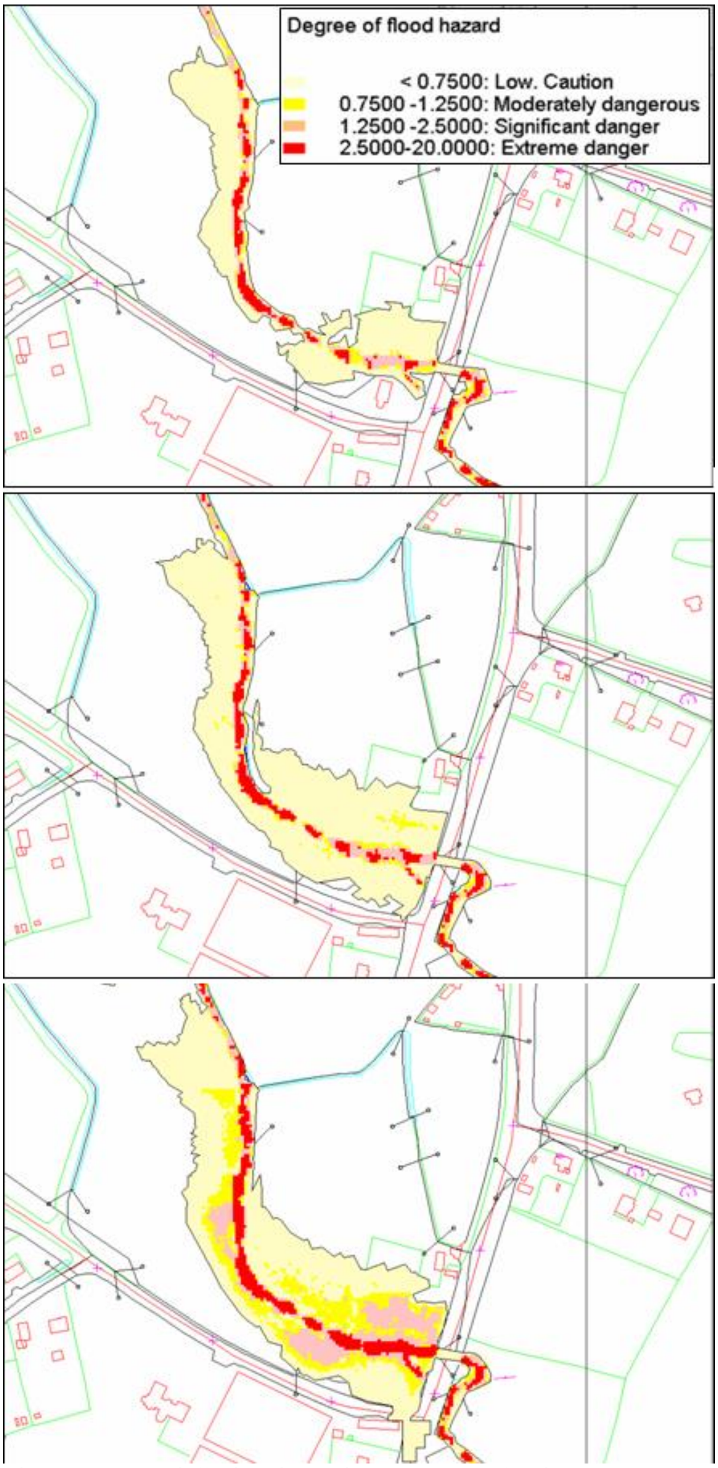

Figure 9-29 Corduff River hazard map at R132 for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.12. Baleally

The main channel passes through a 316m long circular corrugated metal culvert at node 9Ba3030 as is shown in Figure 9-30. This figure indicates that there is no flooding for the 1% AEP fluvial event, 30% and 70% blockage scenarios. This culvert has a high flow capacity (2.6m diameter) which prevents flooding in this area.

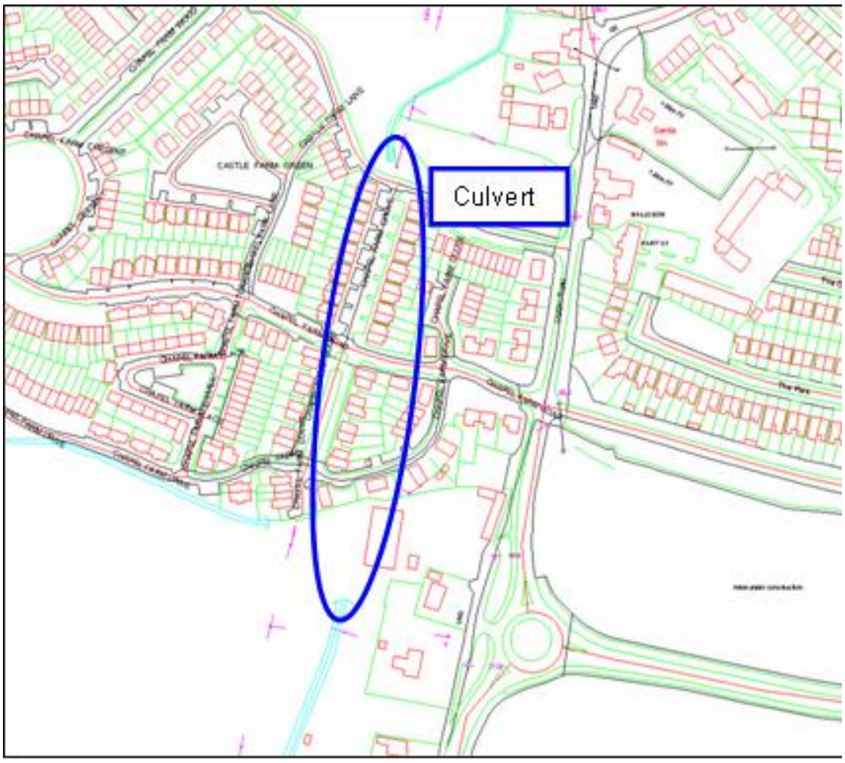

Figure 9-30 Baleally Stream culvert location. No flood for current, 30% and 70% blockage scenarios

9.2.13. Rush West at Channel Road

The Rush West Stream passes through a long culvert (node 11Wa267) where it crosses and follows Channel Road until it outfalls into the estuary. This culvert is 235m long and is of concrete construction. The inlet and outlet shapes are circular and 0.4m and 0.5m in diameter respectively, with the conduit that changes section 118m from the culvert inlet. At the inlet to this culvert is a trash screen that can easily be blocked by debris.

Figure 9-31 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for a 1% AEP fluvial event. The map indicates that there is almost no difference between the current and 30% blockage scenario for the 1% AEP event. The 70% blockage scenario results in slightly more extensive flooding along Channel Road. For a 1% AEP fluvial design event, the culvert is already surcharged. Therefore, a blockage occurring at this culvert for such an event will almost have no effect in terms of additional flood risk.

The highest velocities occur when water runs across Channel Road with similar maximum values of 0.45m/s for the current scenario and the two blockage scenarios. The maximum depths are located in the garden of properties along the left bank floodplain between South Shore Road and Channel Road with similar water levels of 0.65m for the current scenario and two blockage scenarios.

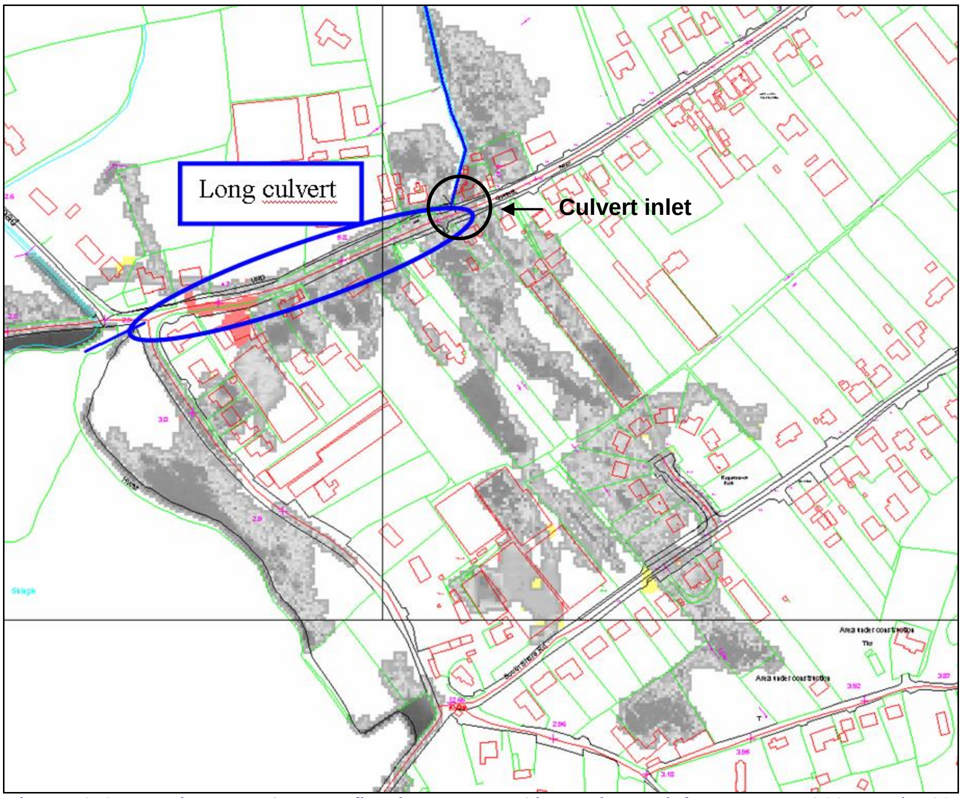

Figure 9-31 Rush West Stream flood extent at Channel Road for current, 30% and 70% blockage scenarios

Figure 9-32 shows the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario for a 1% AEP fluvial event. The hazard values for the current scenario, 30% blockage scenario and 70% blockage scenario are always lower than 0.75. Therefore, hazard classification is ‘caution’ for all areas. The maximum hazard is located in the gardens of properties between South Shore Road and Channel Road and has similar hazard values of 0.30 for the current scenario and two blockage scenarios.

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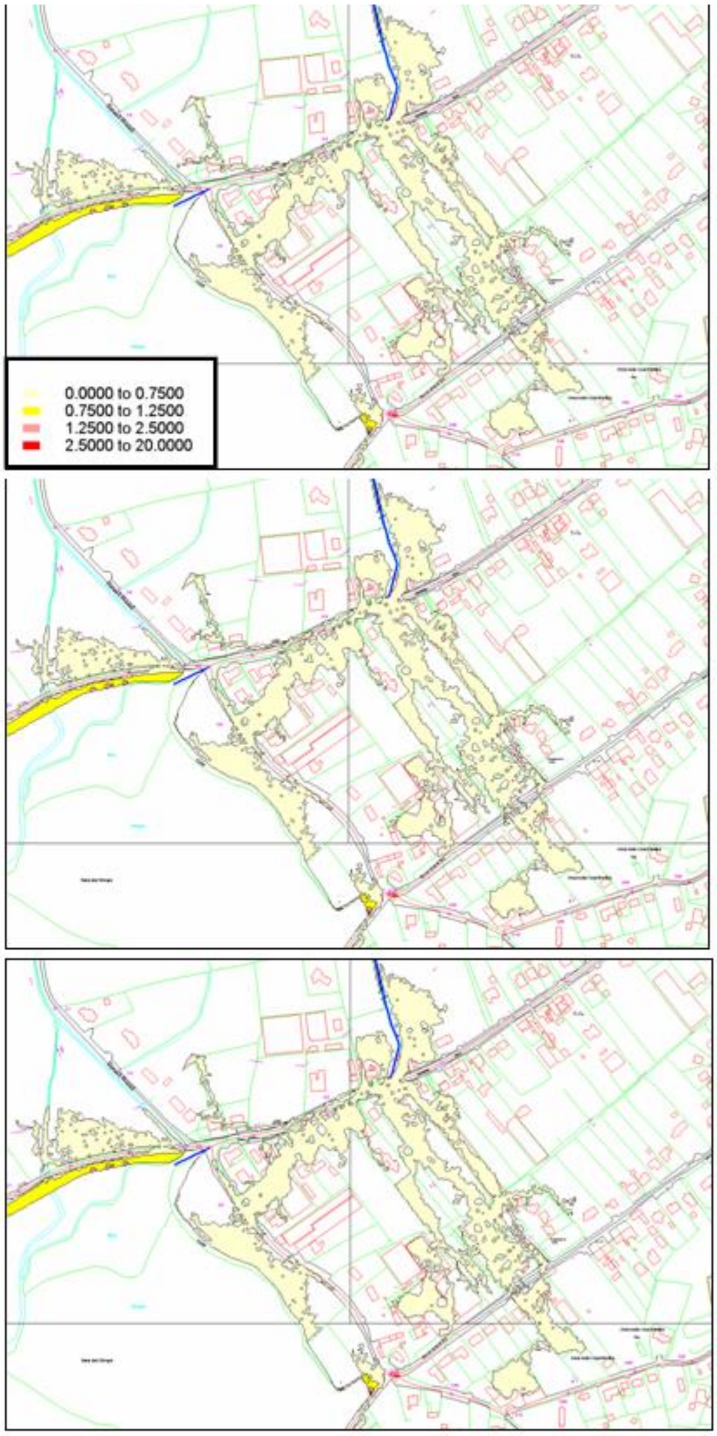

Figure 9-32 Rush West Stream hazard maps at Channel Road for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.14. Mill Stream at Holmpatrick Road, Skerries

The main channel passes through a 13.5m stone bridge where it crosses the Holmpatrick Road at node 15Ma222. The channel at this location has stone walls along both banks and there isn't too much vegetation in the river bed. The bridge soffit level is relatively low so the opening can be blocked by debris carried by the flow.

Figure 9-33 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for a 1% AEP fluvial event. The map indicates that there is almost no difference between the current and 30% blockage scenario for the 1% AEP event. The 70% blockage scenario results in a sizeable increase in flooding in Skerries. For the 70% blockage scenario the water on the upstream face of the bridge reaches 4m AOD and floods the urban area upstream of Holmpatrick Road and the sports ground on left bank until it reaches the Dublin Road. The highest velocities are close to 0.4m/s for the 70% blockage scenario in the narrow flow path before the sports grounds.

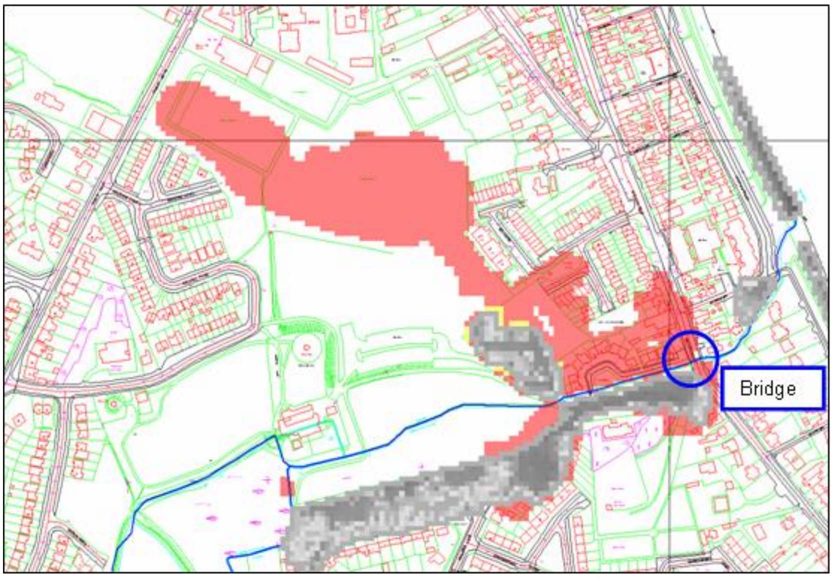

Figure 9-33 Mill Stream flood extents at Holmpatrick Bridge for the current scenario, 30% blockage scenario and 70% blockage scenario

Figure 9-34 shows the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario for a 1% AEP fluvial event. For both the 30% and 70% blockage scenarios the hazard values never exceed 0.75 (based on the 1% AEP event) and the hazard classification is always 'caution'.

Figure 9-34 Mill Stream hazard maps at Holmpatrick Bridge for the current scenario (top), 30% blockage scenario (bottom left) and 70% blockage scenario (bottom right)

9.2.15. Bracken River at Decoy Bridge

The main channel passes through a 36m long culvert at Decoy Bridge (node 16Ma5361). This is a circular culvert constructed of concrete. The culvert is located in quite a rural area along the M1 motorway.

Figure 9-35~~Figure 9-35Figure 9-35~~ shows the flood extents for the current scenario for the 1% AEP fluvial event (shaded grey) and the 30% culvert blockage (shaded yellow) and the 70% culvert blockage.

The map indicates that there is almost no difference between the current scenario and the two blockage scenarios for the 1% AEP event. For a 1% AEP fluvial design event, the culvert is already surcharged, therefore, a blockage occurring at this culvert for the same event has almost have no effect in terms of additional flood risk.

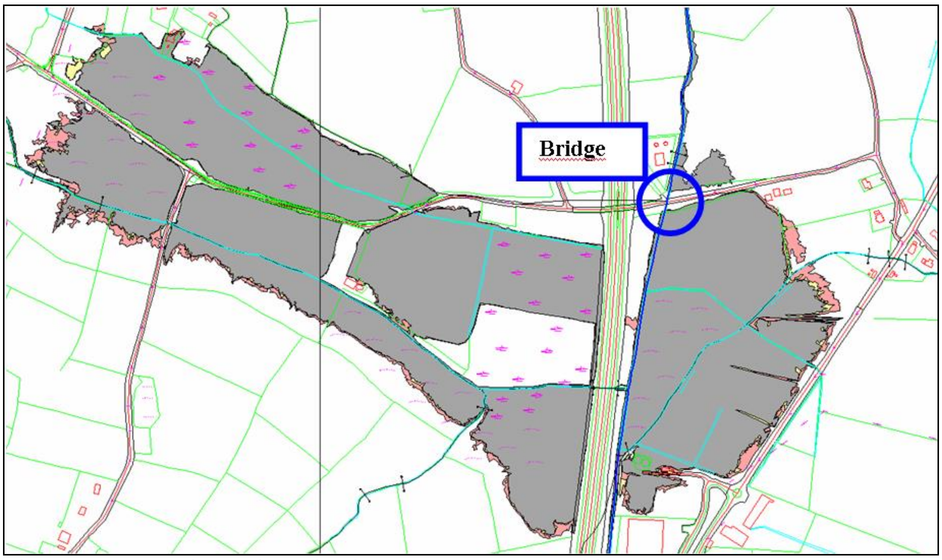

Figure 9-35 Bracken River flood extent at Decoy Bridge for the current scenario, 30% blockage scenario and 70% blockage scenario

Figure 9-36~~Figure 9-36~~~~Figure 9-36~~ shows hazard maps for the current scenario and the two blockage scenarios for a 1% AEP fluvial event. The hazard values for the current scenario and two blockage scenarios vary between less than 0.75 to greater than 2.5. This indicates a hazard classification that varies between 'caution' and 'extreme danger for all'. The maximum hazard is to the west of the M1 motorway and is as a result of deep flood water (up to 2m deep in places) which ponds in the floodplain at the Bog of the Ring.

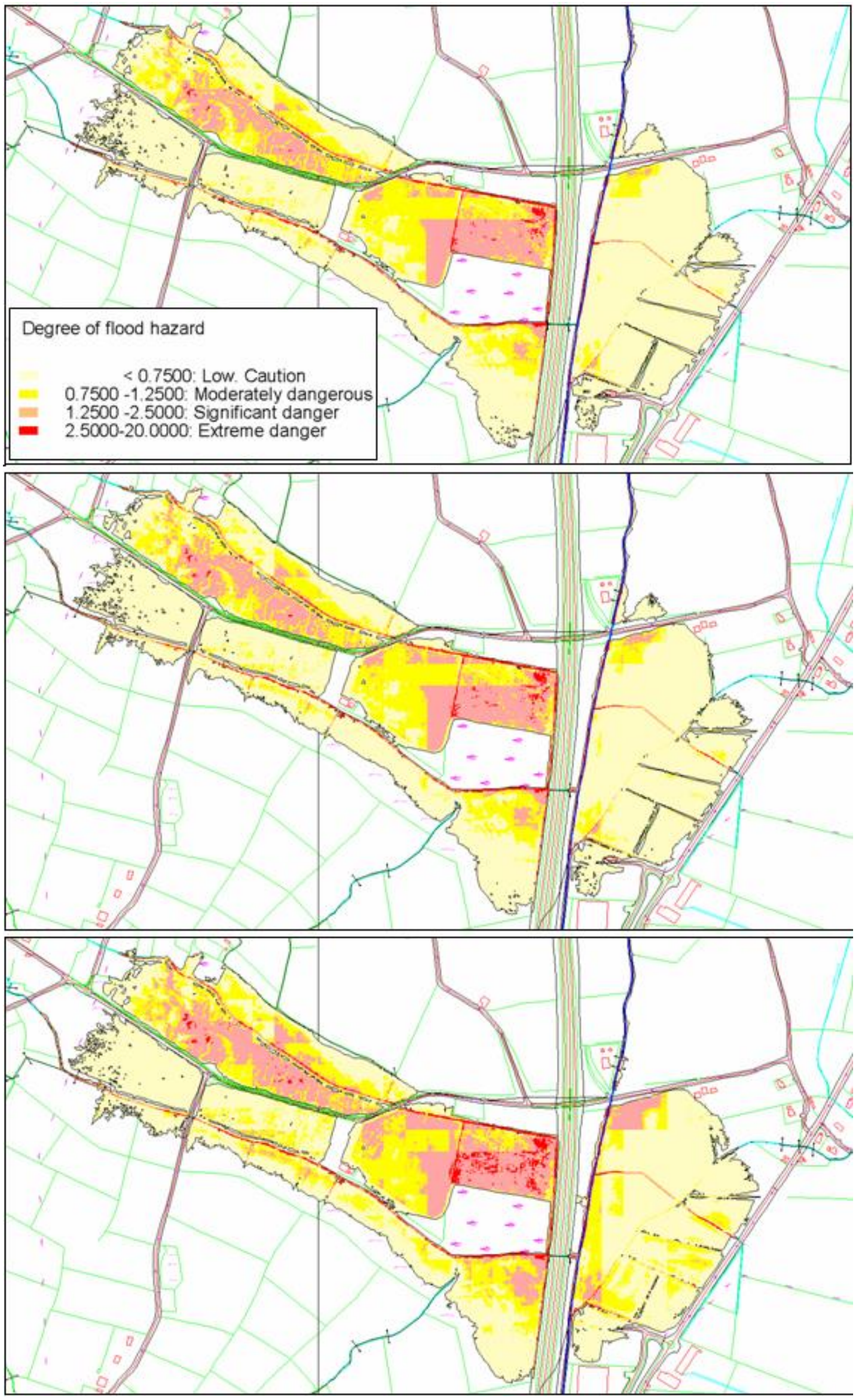

Figure 9-36 Bracken River hazard maps at Decoy Bridge for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.16. Bracken River at R132 Bridge

The main channel passes through a 13.5m stone bridge where it crosses Bridge Street, in Balbriggan at node 16Ma244.The channel at this location has stone walls along both banks and there isn't much vegetation in the river channel. The bridge is at risk of blockage due to the overhanging pipes at the bridge opening which can be blocked by debris carried by the flow.

Figure 9-37~~Figure 9-37Figure 9-37~~ shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for a 1% AEP fluvial event. The map indicates that there are minor differences between the current scenario and 30% blockage scenario, but that the 70% blockage scenario results in a sizeable increase in flood risk with flooding overtopping Bridge Street and flowing down along Quay Street.

The highest velocities occur when flow runs off in a north-easterly direction along Quay Street with maximum values of 0.30m/s, 0.45m/s and 1.25m/s respectively for the current scenario, 30% blockage scenario and 70% blockage scenario. The maximum depths values are located on the right floodplain just upstream from the bridge with values of 1.30m, 1.35m and 1.65m respectively for the current scenario, 30% blockage scenario and 70% blockage scenario.

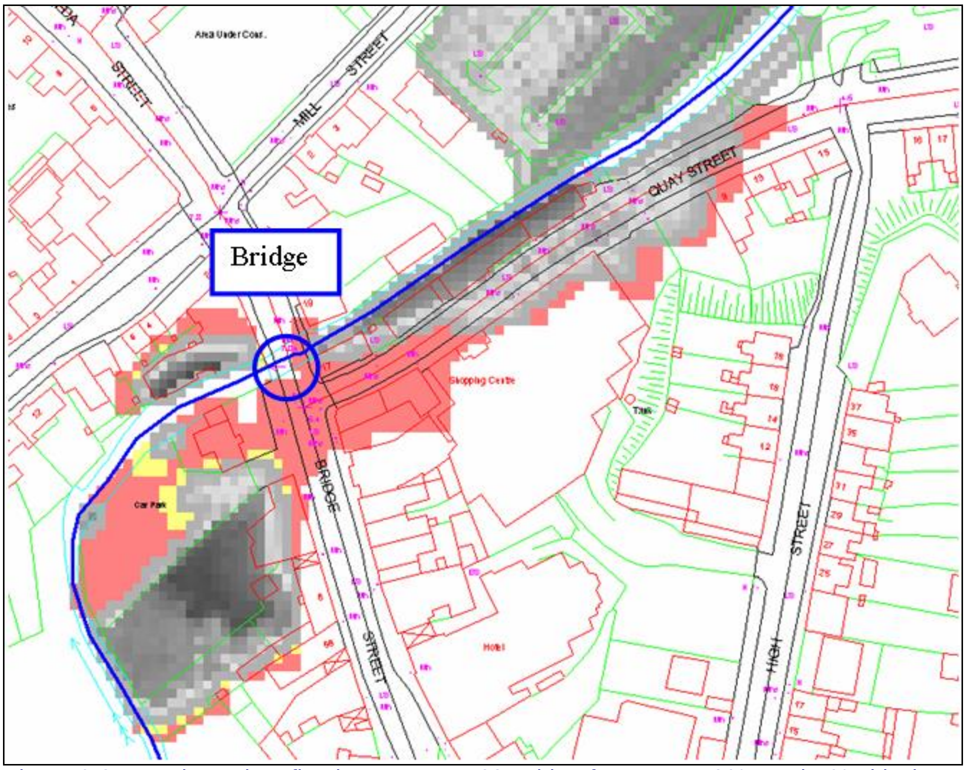

Figure 9-37 Bracken River flood extent at R132 Bridge for current, 30% and 70% blockage scenarios

Figure 9-38 shows the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario for a 1% AEP fluvial event. The hazard value is lower than 0.75 for the current scenario and 30% blockage scenario, which equates to a hazard classification of ‘caution’. For the 70% blockage scenario, the hazard classification is ‘moderately dangerous for some people’ upstream from the Bridge on the right floodplain. The maximum hazard is located on the right floodplain just upstream from the bridge and has values of 0.65, 0.70 and 0.90 for the current scenario, 30% blockage scenario and 70% blockage scenario respectively.

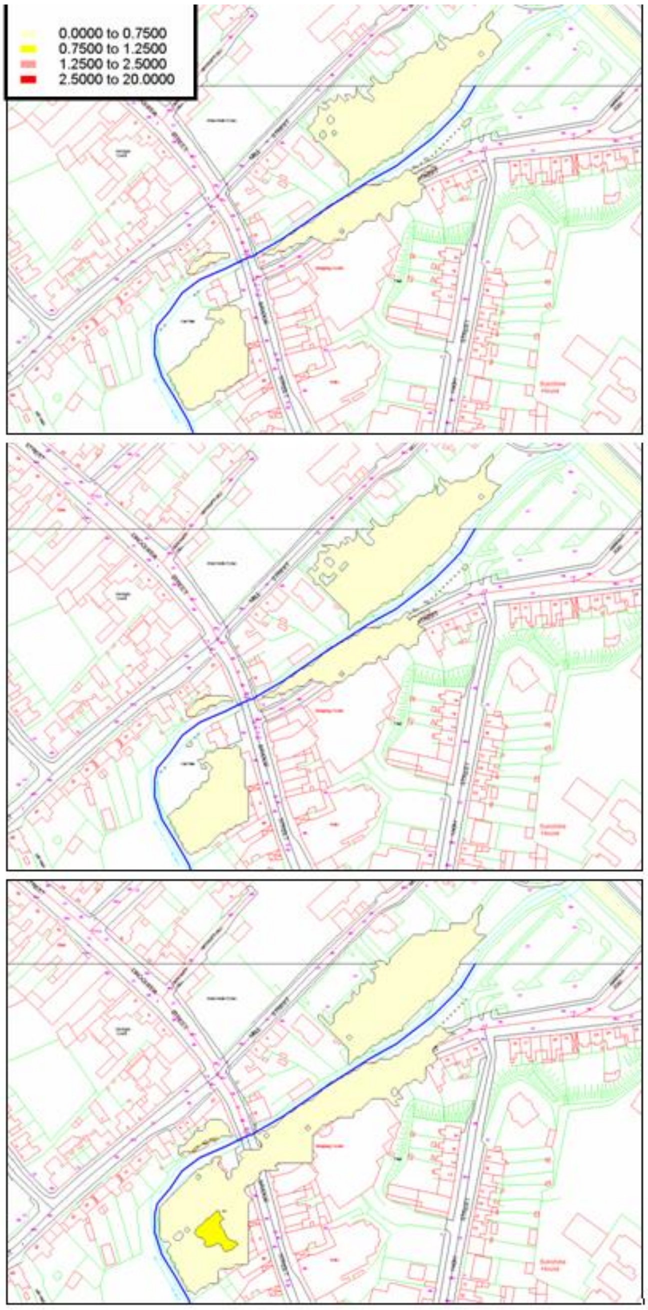

Figure 9-38 Bracken River hazard maps at R132 Bridge for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.17. Mosney at Mosney Road

The main channel passes through a 79m long circular culvert at Mosney Road which is constructed of concrete (node 19Maa548). The channel has a lot of scrub and bushes on the slopes of the channel banks. A partial blockage of the culvert is possible due to the debris carried by the flow.

Figure 9-39 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for a 1% AEP fluvial event. The map indicates that there are minor differences between the current scenario and 30% blockage scenario. There is a small increase in flooding along the Briarleas Road for the 70% blockage scenario.

The highest velocities occur where flow runs off in a north-easterly direction along the access road of the Refugee Centre with maximum values of 0.40m/s, 0.45m/s and 0.55m/s respectively for the for the current scenario, 30% blockage scenario and 70% blockage scenario respectively. The maximum depth values are located along Briarleas Road with values of 0.3m, 0.55m and 0.70m for the current scenario, 30% blockage scenario and 70% blockage scenario respectively.

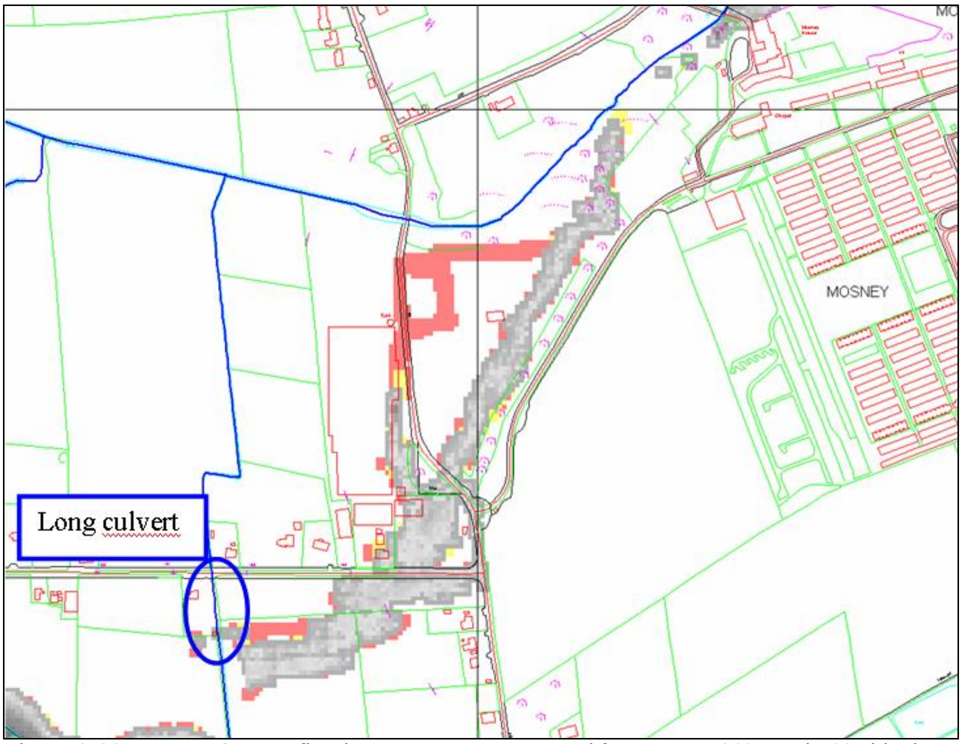

Figure 9-39 Mosney Stream flood extent at Mosney Road for current, 30% and 70% blockage scenarios

Figure 9-40 shows the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario respectively for a 1% AEP fluvial event.

The hazard values for the current scenario, 30% blockage scenario and 70% blockage scenario are always lower than 0.75 which equates to a hazard classification of ‘Low–caution’ for all areas. The maximum hazard values are located along Briarleas Road with values of 0.30, 0.45 and 0.55 for the current scenario, 30% blockage scenario and 70% blockage scenario respectively.

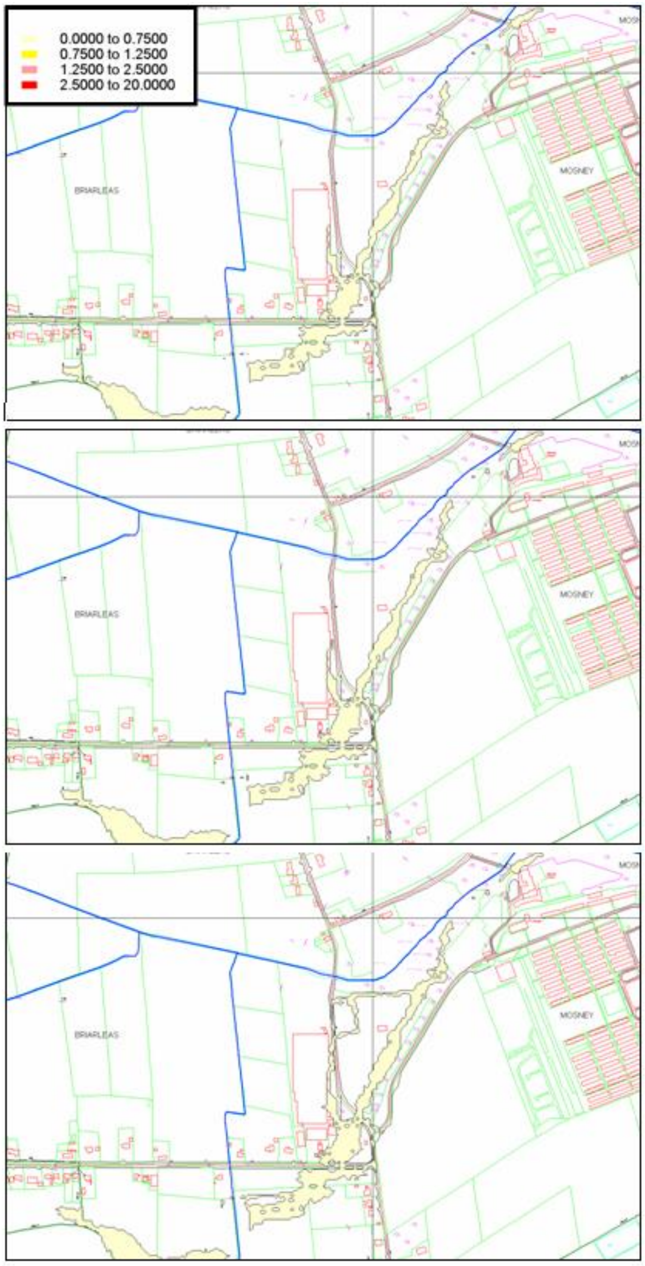

Figure 9-40 Mosney Stream hazard maps at Mosney Road for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.18. River Nanny, Paramadden tributary at Bridge Street

The Paramadden tributary (Nag channel) flows through a 6.5m long stone bridge with three arches at Bridge Street in Duleek (node 20Nag63). The channel at this location has a concrete wall on the left bank and an earth embankment on the right bank. There isn't much vegetation in the river channel or along the banks. A partial blockage of the culvert with debris carried by the flow is possible as the flow is split by the bridge piers.

Figure 9-41 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for a 1% AEP fluvial event. The map indicates that there is a sizeable increase in the flood extents between the current scenario, the 30% blockage scenario and the 70% blockage scenario. The map shows that the Millrace Estate on the left bank and Abbeylands on the right bank are inundated with both a 30% and 70% blockage of the bridge. For the 70% blockage scenario, the water overtops the R152 road before rejoining the River Nanny. The spilling of flood water as a result of this culvert blockage bypasses the existing flood defences which protect both of these housing estates.

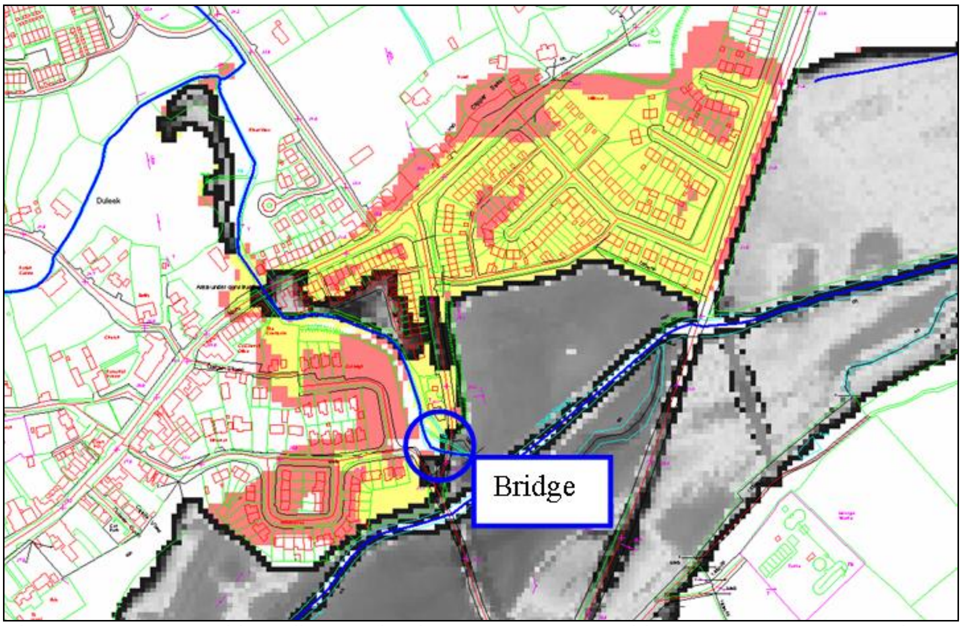

Figure 9-41 Paramadden tributary flood extent at Street Bridge for current, 30% and 70% blockage scenarios

High velocities are present along the left bank of the Paramadden tributary, when water flows into the floodplain, and also in the Millrace Estate along the main street where water flows in an easterly direction towards the R152 road. The highest velocities occur when flooding crosses the R152 road, for the 70% blockage scenario. On the right bank of the Paramadden tributary, velocities across Abbeylands are much lower. The maximum velocities are 0.15m/s, 0.50m/s and 2.20m/s for the current scenario, 30% blockage scenario and 70% blockage scenario respectively.

The maximum depth values are 0.55m, 0.80m and 1.85m for the current scenario, 30% blockage scenario and 70% blockage scenario respectively and occur in the Millrace Estate.

Figure 9-42 show the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario respectively for a 1% AEP fluvial event. The hazard values on the left bank floodplain do not exceed 1.25 for all the scenarios which equates to a hazard classification of 'moderately dangerous for some people'. Inside the Millrace Estate, values vary between 0.2 and 0.4 for the 30% blockage scenario and between 0.5 and 1.0 for the 70% blockage. The maximum hazard values are 0.25, 0.40 and 1.00 for the current scenario, 30% blockage scenario and 70% blockage scenario respectively.

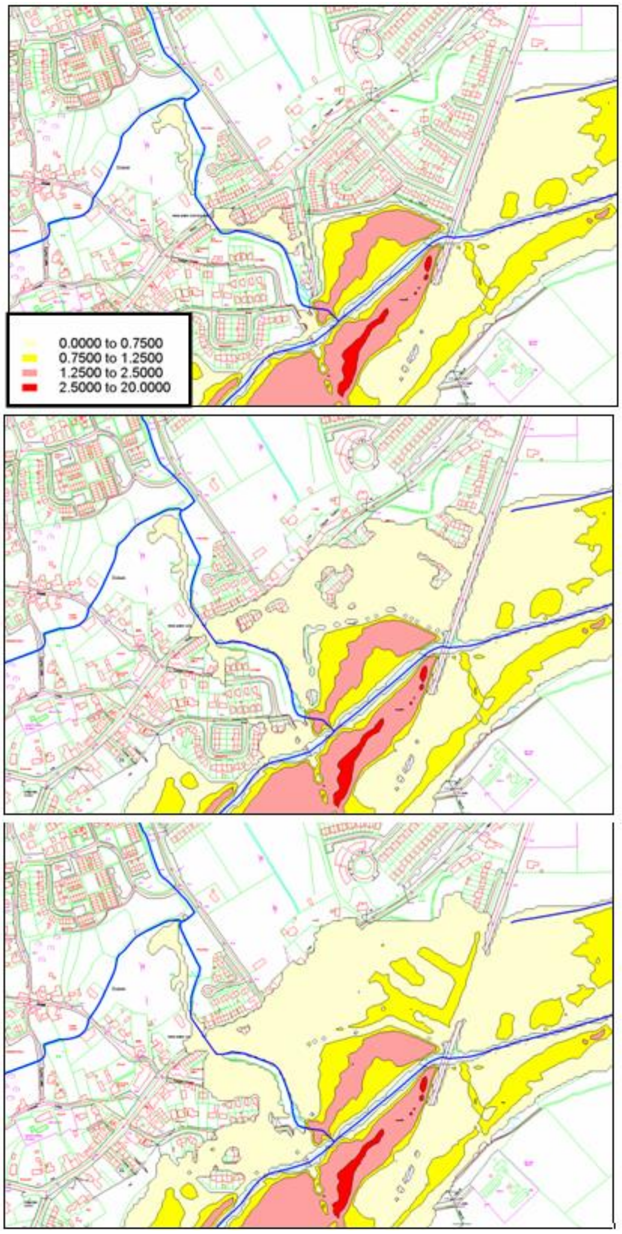

Figure 9-42 Paramadden tributary hazard maps at Bridge Street for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

9.2.19. Brookside Stream at Laytown Road Bridge

The main channel passes through a 13m long circular concrete culvert at Laytown Road in Bettystown. The channel has a lot of scrub and bushes along the channel slopes. A partial blockage of the culvert is possible due to debris carried by the flow.

Figure 9-43 shows the flood extents for the current scenario (shaded grey), the 30% culvert blockage (shaded yellow) and the 70% culvert blockage (shaded red) for a 1% AEP fluvial event. The map indicates that there is an incremental increase in the flood extents between the current scenario, the 30% blockage scenario and the 70% blockage scenario. For the 70% blockage scenario, a sizeable area of land is inundated with some additional properties at risk.

The highest velocities are in the middle of the floodplain with maximum values of 0.20m/s, 0.40m/s and 0.55m/s for the current scenario, 30% blockage scenario and 70% blockage scenario respectively. The maximum depth values are 0.5m, 0.60m and 0.75m for the current scenario, 30% blockage scenario and 70% blockage scenario respectively.

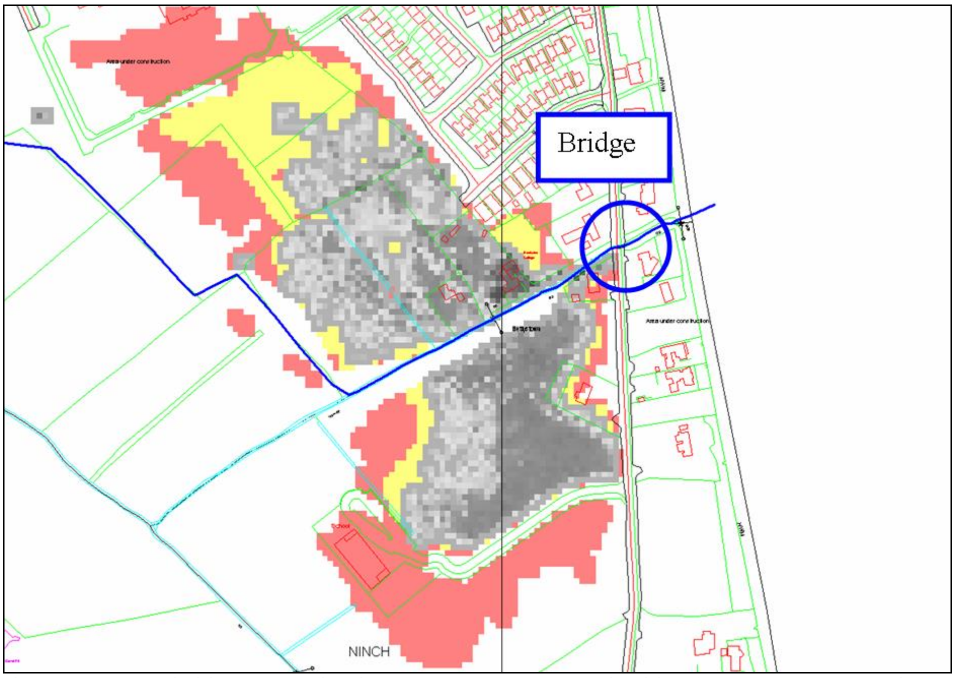

Figure 9-43 Brookside Stream flood extent at Laytown Road Bridge for current, 30% and 70% blockage scenarios

Figure 9-44 shows the hazard maps for the current scenario, 30% blockage scenario and 70% blockage scenario respectively for a 1% AEP fluvial event. The hazard values for all three scenarios are always lower than 0.75 which equates to a hazard classification of ‘Low - caution’ for all the areas. The maximum hazard values are located just upstream from the bridge, on the left bank around the Brookside Cottage with values of 0.40, 0.45 and 0.55 for the current scenario and the two blockage scenarios.

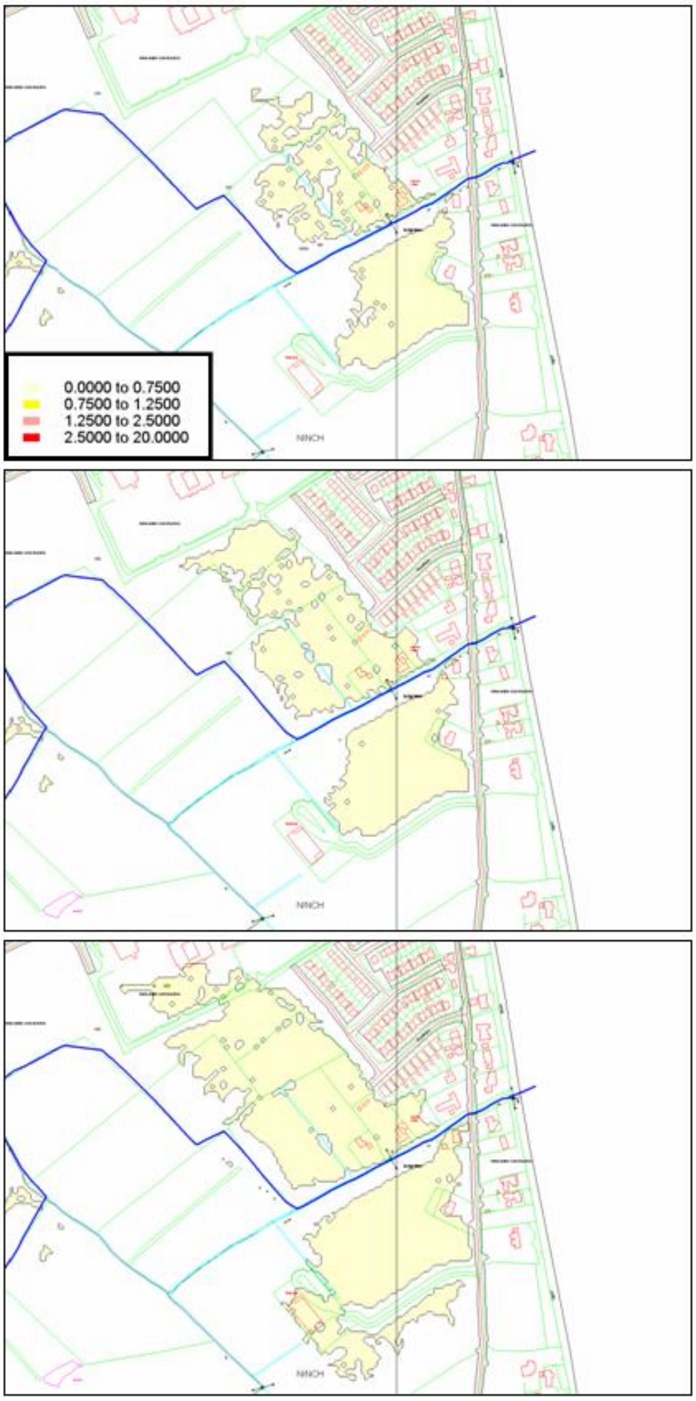

Figure 9-44 Brookside Stream hazard maps at Laytown Road Bridge for the current scenario (top), 30% blockage scenario (middle) and 70% blockage scenario (bottom)

10. Groundwater flood hazard

10.1. Introduction

This chapter summarises the analysis on Groundwater Flood Hazard undertaken as part of the FEM FRAM study. Details on the historic information, data used, methodology/approach to analysis, discussions on results and recommendation are presented in a separate Technical Note (refer to Appendix D).

The main objective of the Groundwater Flood Hazard analysis was to undertake a desk study review of the available data on groundwater to produce a meaningful assessment of the groundwater flood risk in the FEM FRAM study area; to investigate the necessity of GW monitoring in the study area, and if required, recommend GW monitoring locations. The study also investigates the mechanisms by which groundwater flooding can occur in the study area and their remedial measures.

10.2. Data and methodology used

Information on the groundwater bodies and hydrogeology were gathered from the Databases of the Geological Survey of Ireland (GSI) and the data produced as part of the Water Framework Directive and Eastern River Basin Development Plan (ERBD). Data sets have been reviewed and information collated on various facets of the groundwater environment within the study area. These data sets are summarised below:

- DTM of the study area;
- Groundwater Bodies: the GW bodies delineated in the WFD studies;
- Bedrock Geology: the bedrock geology of the study area;
- Aquifer Classification: the aquifers classified by the GSI within the study area;
- Vulnerability Classification: the vulnerability data and classification undertaken by the GSI;
- Groundwater Levels: there was no database of water levels available that would provide sufficient information to be incorporated into a risk assessment. The GW monitoring undertaken by the EPA at five different areas in the Fingal and East Meath are not located within the APSRs. Thus, there is a lack of GW level data in the APSRs and their vicinities; and
- Groundwater flood records: there were no records of GW flooding.

The various mechanisms that lead to GW flooding were described and evaluated in terms of the study area. While it was possible to develop an understanding of the mechanisms that can contribute to GW flooding, a quantified assessment of risk from GW flooding was not possible, especially on a strategic scale. The only mechanisms that are considered applicable to the area in question were areas underlain by permeable sands and gravels.

10.3. Summary of results and recommendations

Significant groundwater flooding in Ireland is associated with Karst landscapes and Turloughs. However, this setting does not occur in Fingal East Meath Study Area. The hydrogeological setting of the Fingal East Meath FRAMS area together with all the available information indicates that there is no significant groundwater flooding in the study area.

It is acknowledged that the excavation of basement and car parks has the potential to lower the effective groundwater level, and if the base of the excavation is below the water table there is a potential for groundwater to infiltrate and cause flooding.

As per the FEM FRAMS Brief, recommendations are to be made for monitoring of groundwater in the study area. Based on the results of a preliminary analysis of the available data, it is considered that there are no specific areas which are identified as being susceptible to groundwater flooding. Therefore, at this stage there is no justification in installing the groundwater monitoring as the groundwater flooding risk is considered to be insignificant.

The following points summarise the results of the groundwater hazard analysis undertaken for the FEM FRAM study area and the recommendations made based on the outcome of the analysis:

- Groundwater flooding occurs when the groundwater table exceeds the ground level e.g. Turloughs in a Karst environment;
- The water table, which is the level of water naturally occurring underground, varies from location to location and fluctuates with the weather conditions / seasons and in some cases this may actually be visible at ground level;
- A Preliminary Groundwater Risk Assessment was undertaken for the Fingal East Meath catchment which looked at the potential flood hazard arising from groundwater sources within the study area;
- The hydro-geological conditions in the Fingal East Meath catchment together with all the available information indicate that the conditions do not exist for groundwater flooding and hence that groundwater flooding is not a significant risk within the catchment;
- In the absence of any areas where groundwater flooding is known to have occurred, it is not considered necessary to implement a groundwater monitoring programme. However, if the groundwater monitoring programme is to proceed, guidelines for the selection of possible monitoring sites are provided in the Groundwater Technical Note. The benefit of a groundwater level monitoring programme would be to provide a clearer picture of the hydro-geological regime. It would not assist assessing the risk of something that there is no evidence to support;
- In the unlikely event that groundwater flooding were to occur (contrary to the present indications), the flooding would be within areas already delineated as being at risk of pluvial/fluvial flooding. Thus, under the planning guidelines, development within these areas would be generally prohibited or subject to the justification test and the risk of flooding from all sources pluvial, fluvial and groundwater would be considered at that stage;
- There is a risk of groundwater flooding of poorly constructed basements. Developments that incorporate basements or deep excavations should be required to

drill a borehole & install a piezometer to establish the depth of the groundwater table in relation to the base of the excavation. If the water table is within 1 meter of the base then the developer needs to be conditioned to ensure that the basement is adequately sealed / tanked. All basements must be designed in accordance with British Standard BS8102:1990. This British Standard defines four grades of basements ranging from Grade 1 Car parking where some seepage is allowed to Grade 4 Archives and stores – totally dry environment;

- Basement flooding also can occur from other sources such as surface water from the street, backing up of storm or sewer pipes and so forth. These types of flooding are not considered as part of the GW assessment remit and further details can be obtained from the GDSDS Regional Drainage Policies, Volume 6 – Basements; and
- A recommendation for future work includes the development of a basement register which notes the location of the basement, size, floor level, purpose, record of flooding and the type of flooding.

11. Pluvial flood hazard

11.1. Introduction

This chapter summarises the Pluvial Flood Risk Assessment undertaken as part of the FEM FRAM study. Details on the historic information, data used, methodology/approach to analysis, results of analysis and their comparison with the historic information, consultation with local authority and recommendation for further analysis / studies / monitoring are presented in a separate Technical Note in Appendix E.

The main objective of the pluvial analysis was to assess the potential locations where pluvial floodwaters and surface runoff might accumulate within APSRs during extreme rainfall events and/or blockage or saturation of the stormwater drainage systems and assess the potential degree (extent and depth) of flooding that could occur. Thus, the assessment has not required consideration of the capacity or arrangement of the urban stormwater drainage systems.

11.2. Data and methodology used

The data used for the pluvial analysis was

- The Digital Terrain Model (DTM) for the main rivers and coastal areas covering all 33 APSR's in the study area which was made available by the OPW;
- Rainfall data in the study catchment which was acquired from Met Eireann; and
- The information on historic flood events in the study area which was available from the National Flood Hazard website www.floodmaps.ie, and the GIS layers for this information from the OPW.

For the purpose of this analysis, a single DTM was created for the full study area. This was then broken down into smaller DTMs (total seven), bounding each APSR, so as to facilitate data pre and post processing and to work within the limits of the available computing environment.

The ISIS-2D (FAST) pre-processor was used to identify topographic depressions where water will pond and the pathways where water can pass between them. An overall rainfall input of 50 mm was applied in the model instantaneously over the entire study area. This rainfall is considered to have an approximate return period in excess of 1 in 100 years for the rainfall event of 1 to 5 hour duration. It is noted here that the UK's Environment Agency's National Pluvial Flood Risk Assessment has adopted a 200 year event over 6 hour duration. However, no such national standards or requirements for pluvial flood risk assessments exist for Ireland.

The ISIS-2D (FAST) computational engine has been used to route pluvial flood water over the floodplain. It uses a simple set of rules based on how water levels respond to the topography described in the DTM. The ISIS-2D (FAST) adopts a 3 stage approach to modelling pluvial flooding (steps 1 -3 in Figure 11-1~~Figure 11-1~~Figure 11-1). The final output from the model was obtained in the form of an ASCII grid containing the depths of water for the region. The result of pluvial model analysis was then presented in 1:50,000 maps (extent and depths) for review purpose.

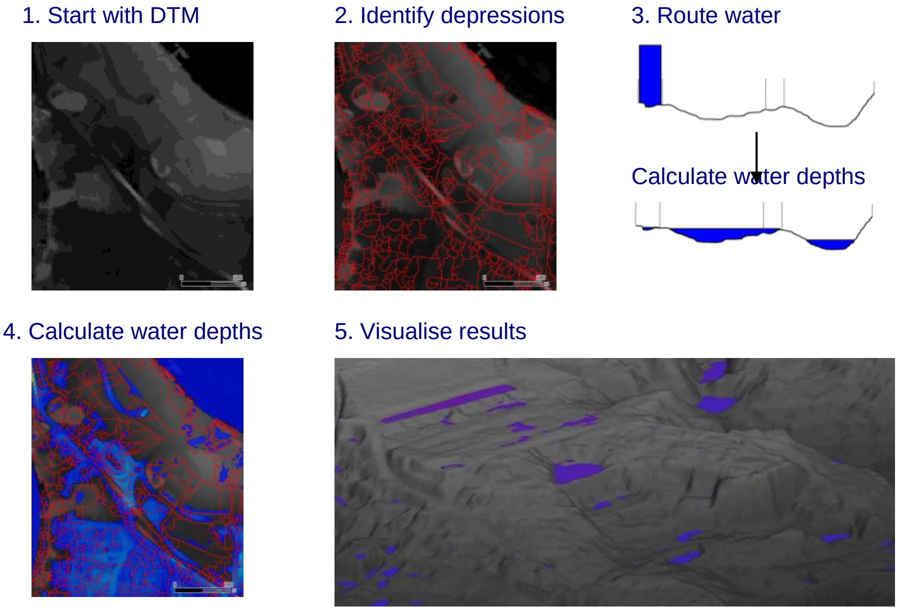

Figure 11-1 Approach used to modelling pluvial flooding

11.3. Summary of results

The results of the pluvial model analysis were compared visually with the historic flood locations at all 33 APSRs in the study area. The results replicated the historical flooding (either from fluvial, pluvial or coastal sources or a combination of sources) at almost all locations in the APSRs. The model results also showed additional areas of flooding adjacent to APSRs). The results showed that a few of the APSRs are at risk of flooding from only pluvial sources (e.g., Donabate area), whereas other areas are at risk of flooding from either fluvial, coastal, pluvial or a combination of all three types of flood sources.

A consultation workshop with FCC, MCC and the OPW was held on 9th March 2010 which reviewed the draft pluvial flood maps. The workshop provided valuable feedback confirming that the pluvial flood maps were representative of expectations and knowledge of the area.

11.4. Recommendations

A number of recommendations have been made to so as to enhance the analysis or improve reporting beyond current requirements of the study. These recommendations are related to:

- Identification of critical storm durations / frequencies in the study area;
- Consideration of sewer capacity/rural infiltration in the modelling;
- Estimation of approximate velocities and depths along flow paths;

- Routing of flow along the river network and drainage channels; and
- Use of single DTM and use of a high powered computing environment.

12. Geomorphological assessment

12.1. Introduction

A preliminary, broad-scale desk-based investigation into the geomorphology of the watercourses in the study area and their catchments was undertaken as part of the FEM FRAM study. The principal focus of this fluvial geomorphological analysis was to undertake a preliminary investigation of the sediment erosion, transport and deposition processes which transport sediments from upland areas within river catchments, into and through the valley lowlands to the coastal zone. It is important to recognise that high-levels of uncertainty are associated with such broad-scale, desk-based studies. It is also not possible to identify specific local geomorphological responses to drivers such as climate change and urbanisation at the catchment-scale level.

The fluvial geomorphological assessment undertaken at the FEM FRAM study area comprises:

- Collation of available data on topography, drift geology, soils, land use and any historical records of erosion/deposition;
- A review of any previous geomorphology studies in the catchment;
- GIS-based assessment of the likelihood, degree and spatial extent of erosion or deposition; and
- Comparison of the GIS based assessment and previous studies, to the flood risk maps and areas of property and infrastructure.

Details on the geomorphological assessment are presented in a separate Technical Note in Appendix F. This chapter summarises the data and methodology used for the geomorphological assessment and briefly discusses the results and recommendations made during the assessment.

12.2. Data and methodology used

The following data was used for the geomorphological assessment:

- The GIS data for the study area supplied by the OPW ;
- Historic six inch maps of the study area, which were accessed from the Ordnance Survey of Ireland’s website (www.osi.ie); and
- Historical erosion and deposition data.

The methodology used in the study is based on the Broad Scale Ecosystem Assessment (BSEA) Toolbox 1 (Defra, 2006). The objective of the BSEA is to provide consolidated ecological assessment guidance for practitioners in flood management policy analysis. This was developed to support the production of Catchment Flood Management Plans (CFMPs) and Shoreline Management Plans (SMPs) in the UK, and hence it is considered to be appropriate for a preliminary, broad-scale desk-based geomorphological investigation for the FEM FRAMS.

12.3. Discussions on results

To identify locations with the potential for high rates of erosion or deposition, a desk-based assessment was undertaken using the key variables such as channel gradient, channel sinuosity, dominant drift geology and dominant land use of each of the watercourses in the study area. With this, a high level overview of each of the watercourses in the study area was carried out, which is summarised below:

- In general, the study area is characterised by low lying, undulating topography, with a covering of glacial till. Although the areas of woodland are limited, agricultural land directly bordering watercourses, which is more vulnerable to erosion than grassland or woodland, is extensive;
- The gradients of all watercourses are predominantly low to medium; therefore the ability of the watercourses to transport sediment is relatively low. Sediment transfer is more likely to be "pulsed" during high flow events with temporary storage of sediment in-channel features such as bars. Therefore, sediment deposition under normal flow conditions (i.e. not during time of flood) is likely to occur within the channel;
- The majority of the watercourses have been straightened and have been in their present location since approximately 1837, and their planform has not changed since this time. There are no major reservoirs or lakes in the study and therefore major sediment sinks will be limited; and
- A review of the 10 year and 100 year return period flood maps indicates wide fluvial flood extents in agricultural land. During flood events there may be increased deposition of suspended sediment on the floodplain in the flood extent locations. Flood water draining back into the channel may also erode agricultural land.

12.4. Recommendations

The study recommended to undertake more detailed field survey work (at detailed design stage) such as walk over surveys (noting for example current geomorphology), river channel shape (width, depth, cross-section), slope, planform and any historical meanders, floodplain geomorphology, land use on the floodplain, bed sediment, bed features (e.g. riffle-pools etc), management of banks (bank profile, bank material, bank protection), channel management regime and organisation undertaking this, in the following locations on the watercourses so to ascertain any threat from flooding and erosion to the road / railway line / housing estate etc:

- The Mayne River – upstream of the M1;
- The Sluice River – upstream of the railway line;
- Lissenhall Stream – upstream of the N1;
- The Turvey River – between the N1 and the railway line;
- The Mill Stream – upstream and downstream of the railway embankment;
- The Bracken River – in the headwaters alongside the M1 motorway embankment;
- Mosney stream – to ascertain risk to Mosney;
- Nanny / Hurley confluence – to ascertain erosion potential and risk to Athcarne; and

- Brookside Stream - to ascertain impact of potential erosion on housing estate.

Recommendations for further detailed monitoring could be made following the above works.

13. Summary and recommendations

13.1. Summary of key outputs

The main objective of the hydraulic assessment undertaken as part of FEM FRAMS was to determine the flood risk for the watercourses, estuaries and coastline in the study area for specific design events and future scenarios. The study involves modelling 23 rivers and streams in the study area and three estuaries as detailed in Table 13-1~~Table 13-1Table 13-1~~.

Table 13-1 Rivers, streams and estuaries included in the FEM FRAMS

River name (abbreviation)		
Mayne River (MAY)	Baleally Stream (BAY)	Balbriggan North Stream (BNS)
Sluice River (SLU)	Bride's Stream (BRI)	Delvin River (DEL)
Gaybrook Stream (GAY)	Jone's Stream (JON)	Mosney Stream* (MOS)
Ward River (WAR)	Rush West Stream (RWS)	River Nanny (NAN)
Broadmeadow River (BRO)	Rush Town Stream (RUT)	Brookside's Stream (BSS)
Lissenhall Stream (LIS)	St Catherine's Stream (CAT)	
Turvey River (TUR)	Rush Road Stream (RUR)	Baldoyle Estuary
Ballyboghil River (BAL)	Mill Stream (MIL)	Broadmeadow Estuary
Corduff River (COR)	Bracken River (BRA)	Rogerstown Estuary

** The Mosney Stream is also known as the Bradden Stream*

Twenty river/estuary hydraulic models were developed to model the twenty three rivers and the three estuaries. The Ward and Broadmeadow, Ballyboghil and Corduff and the Bride's and Jone's streams were modelled together to ensure that any interaction in flood flows between the rivers was accurately captured. A coastal model and a pluvial (surface water) model were also developed.

Of the twenty river/estuary models developed under the study, seventeen are ISIS 1D-2D linked hydrodynamic models whereas the remaining three are ISIS 1D hydrodynamic models. The river/estuary models used surveyed channel cross sections and structure details of approximately 305km length of river channel (165km in the high priority watercourses (HPWs) and 140km in the medium priority watercourses (MPW)). In addition, LiDAR DTM, aerial maps and photographs were also used in the development of the hydraulic models. The level of complexity of these hydraulic models is much higher at APSRs and their vicinity, with closer cross sections and greater detail for out-of-bank flow routes.

The coastal model, developed using the study area LiDAR DTM, covered the Fingal and East Meath coastline. The flood extents from the coastal model have been merged with those of the river models (tidally dominated runs) to produce flood extents for the coasts, estuaries and tidally dominated reaches of the rivers. There is limited coastal flooding in the Fingal-East Meath study area, due to generally high ground levels along the coast.

The pluvial model, which was developed using the LiDAR DTM, covered the entire study area. The main objective of the pluvial analysis was to assess the potential locations where pluvial floodwaters and surface water runoff might accumulate within APSRs during extreme rainfall events and/or blockage or saturation of the stormwater drainage systems and to assess the potential degree (extent and depth) of flooding that could occur. Thus, the assessment did not require consideration of the capacity or arrangement of the urban stormwater drainage systems.

The hydraulic analysis involved design event simulations for eight annual exceedence probabilities (AEP) (i.e., 50%, 20%, 10%, 4%, 2%, 1%/0.5% and 0.1% AEP), for both fluvial and tidal events, with and without defences and for the current and future flood risk scenarios. (1%/0.5% refers to the 1% AEP fluvial and 0.5% AEP tidal which is the standard design event for fluvial and tidal events).

One of the major outputs of the FEM FRAM study is a suite of flood maps of the rivers, estuaries and coastline in the study area which provide a visual interpretation of the results of the hydrological and hydraulic analyses. The suite of mapping includes flood extent maps, flood depth maps, flood velocity maps and flood hazard maps. The study has also estimated the uncertainties associated with the hydrological and hydraulic assessment, and the level of confidence associated with the flood outlines.

Where suitable data was available, the hydraulic models were calibrated to the peak flows and levels at the gauging stations (Table 4-3~~Table 4-3Table 4-3~~). The models were also calibrated to historic flood data such as recorded flood marks, photographs and reports. Draft flood maps were prepared and reviewed at two workshops (14 December 2009 and 9 March 2010) by the Local Authority engineers and the area engineers from the OPW. The draft flood maps showed the historic flood locations and the flood extent for the 10% AEP and 1% AEP (fluvial) and 0.5% AEP (tidal) events.

The sensitivity scenarios undertaken indicate, in terms of water level differences, that the river models are more sensitive to changes to the inflow values than to changes to the roughness values. Furthermore, it was observed that, in general, the increase in water levels between the current scenario and MRFS are similar to the 20% inflow increase for the sensitivity test.

13.2. Unique features of some river catchments

In the course of the development of the hydraulic models, some unique features were encountered for some of the river catchments in the study area. These features are summarised below.

13.2.1. Ballyboghil and Turvey River upstream of the M1

While developing the hydraulic model of the Ballyboghil River, it was observed that the flood flow from the Ballyboghil River overspills into the Turvey River catchment upstream of the M1 even at some AEPs (e.g. 10% AEP). For all design events, this additional inflow was estimated using flow data from the 2D model domain of the Ballyboghil and Corduff model and accordingly distributed along the Turvey River main channel upstream of the M1 motorway. For more extreme fluvial events, this additional flow can peak at twice the flow in the Turvey River upper catchment.

13.2.2. Gaybrook Stream

At low AEP events (e.g. 0.1% AEP), some of the flow from the upstream part of the Gaybrook Stream overspills into the Sluice River catchment. This required some minor modification to the original Sluice River hydraulic model to represent this flow.

13.2.3. Sluice River at Feltrim

There is a large Roadstone quarry in the upper catchment of the Sluice River on the Feltrim Road. During extreme rainfall events, the quarry acts as a large attenuation system, thus reducing local flooding. Roadstone confirmed that when the surface water and groundwater accumulating in the quarry exceeds certain levels, the water is pumped into the surface water/river network. However, this pumping is unlikely to coincide with the extreme rainfall period. Based on this, and following discussions with Local Authority engineers, only 50% of the sub-catchment area upstream of the quarry was taken as the active contribution area for the most upstream sub-catchment of the Sluice River.

13.2.4. Balbriggan North Stream

The Balbriggan North Stream main channel length is 1.7km in length and there are two tributaries that have a combined length of 0.62km. The entire main channel and one of the tributaries are culverted. The river has been modelled using a combined 1D-2D hydrodynamic model to simulate the routing of fluvial flows. The model results show that flooding, via the surcharging of manholes in Drogheda Street, only occurs for the 0.1% AEP fluvial event.

13.3. Summary of results from other models

13.3.1. Summary of coastal flood risk

The results of the coastal modelling indicated that there is limited coastal flooding in the Fingal-East Meath study area, mainly due to the high ground levels along the coastline. Localised coastal flooding for lower AEP events does occur in Bettystown, Laytown, Skerries, Rush, the Burrows, Malahide and Portmarnock. The results also showed that defences added to the coastal model actually have a limited impact on the flooding for the current scenario (identified as ‘defended areas’ on the flood extent maps). However, there is an increase in the flood extent, and hence the risk of coastal flooding, for the MRFS particularly in Balbriggan, Skerries, Malahide, Portmarnock and Baldoyle.

The FEM FRAMS coastal results were compared to two previous coastal studies (Dublin Coastal Flood Protection Project (DCFPP) and Irish Coastal Flood Protection Strategy Study (ICPSS)) and the results were found to be compatible.

13.3.2. Summary of pluvial flood hazard

The results of pluvial modelling show that the historical flooding (either from fluvial, pluvial or coastal sources or a combination of sources) was replicated at almost all locations in the APSRs. The model results also show some additional areas of flooding adjacent to APSRs. The results show that a few of the APSRs are at risk of flooding from only pluvial sources (e.g. Donabate area), whereas other areas are at risk of flooding either from fluvial, coastal, pluvial or a combination of all three types of flood mechanisms.

13.3.3. Summary of groundwater flood hazard

According to the groundwater (GW) flood hazard analysis there is no indication of significant GW flood risk in the study area. This was based on the review of the hydro-geological conditions in the catchment and a review of all other available information. In the absence of any areas where groundwater flooding is known to have occurred, it is not considered necessary to implement a groundwater monitoring programme. However, if GW flooding events are recorded in the future within APSRs, then boreholes can be installed at these locations to allow monitoring of GW level fluctuation patterns. The study recommends the development of a basement register which notes the locations of basements, size, floor level, purpose, record of flooding and the type of flooding.

13.3.4. Summary of geomorphological assessment

The geomorphological analysis found that all watercourses in the study area have predominantly low to medium gradients and therefore the ability of the watercourses to transport sediment is relatively low. However, some dredging of the lower reaches of the Broadmeadow river/estuary is undertaken by the OPW. The study recommends undertaking more detailed field survey work, such as walk over surveys, so as to ascertain any threat from flooding and erosion upstream of the M1, N1 and the railway line crossing by the main rivers including the Mayne, Sluice, Lissenhall, Turvey, Mill Stream and the Bracken Rivers, as well as at the Nanny/Hurley confluence, Brookside Stream and Mosney Stream which pose a threat to the nearby housing estates.

13.4. Recommendations

The following recommendations have been identified during the modelling and flood mapping stage of the study.

13.4.1. Broadmeadow River

At section Baqa791, a culvert has not been included in the model due to instability problems that could not be resolved. The impact of omitting this structure on the model results is negligible because of the low flows (even for high return periods), the significant storage and cross-section conveyance capacity of the channel and the rural location. The omission of this culvert will have no impact on the flood extents and hence flood risk management options at this location. However, should any planning application be considered at this location then this should be reviewed.

13.4.2. Mayne River - Cuckoo Stream tributary

The Dublin Airport Drainage and Pollution System Control were included in the Mayne model according to the details provided by DAA. Simplifications were assumed to represent the installations of the system into the model. The attenuation tank (50 separate parallel culverts) located in the Cuckoo Stream was modelled as a reservoir unit with an equivalent area.

13.4.3. Sluice River at Feltrim

It is recommended that consideration is given to formalising the current informal procedure of pumping flood water stored in the quarry during periods of dry weather to ensure flood water is not pumped into the river during a period of high flow that may cause an increased flood risk.

13.4.4. Mill Stream in Skerries

There is a control structure on the Mill Stream downstream of the railway embankment. The control structure comprises two sluices (one to regulate the flow into the river and the other to regulate the bypass flow into the mill) and one control weir. Access to this chamber is very difficult so the exact dimensions and levels of the three structures were defined based on a sketch and photographs provided by the surveyors. As flooding in this area is significant, it is recommended that further investigation of this structure is undertaken (e.g. CCTV or through the identification and review of as built drawings, if available).

13.4.5. Balbriggan North model

As discussed earlier, the Balbriggan North Stream main channel and one of the tributaries are culverted. The data used to build the model was provided by the client and was based on design drawings. The culverts do appear to be sized appropriately and flooding, via the surcharging of manholes only occurs for the 0.1% AEP fluvial event. If flooding does occur, a manhole and CCTV survey of this culvert is recommended to confirm its size.

Given the extent of the culverted reaches through an urban area, the most appropriate method for modelling this watercourse would be an urban drainage modelling tool such as InfoWorks CS. It is recommended that this watercourse is modelled with an urban drainage modelling tool to check on the accuracy of the results from the 1D-2D hydrodynamic model.

13.4.6. Gaybrook Stream

A significant amount of time was spent trying to determine the precise details of the 1.4km culverted section of the Gaybrook Stream through the Hollywell Estate, under the M1 motorway and where it connects into the double box culvert further north. The information gathered was based on the planning file and discussions with the Developer and the Local Authority. It was not possible to obtain an 'as built' drawing. It is therefore recommended that a survey of this culvert and the double box culvert is undertaken.

Also, following a site visit it was established that there was an interface between the Gaybrook stream and the ponds upstream of the Hollywell Estate. These were included in the survey data based on the LiDAR data and estimated control structures dimensions. Surveying of the two ponds and their respective inlet/outlet and control structures is recommended to improve the accuracy of the hydraulic model in this area.

13.4.7. Nanny River

The Nanny River's main tributary, the River Hurley was not included as a HPW or MPW in the brief and hence was not surveyed or modelled. The Hurley catchment represents 42% of the whole Nanny River catchment and this was represented in the Nanny River model as a point inflow (on the 2D domain). It is therefore recommended to improve the Nanny model by including the Hurley River in the 1D model.

The existing defences on the Paramadden tributary should be extended further upstream as during the 1% AEP design fluvial event the flow goes out of bank upstream of the existing defences and causes flooding of the western part of the Millrace Estate.

13.4.8. Bracken River

One of Bracken's main tributaries, located to the west of the M1 motorway (Bog of the Ring), was not included as a HPW or MPW in the brief and hence was not surveyed or modelled. It was included in the Bracken River model as a reservoir unit. It is therefore recommended to improve the Bracken model by including the tributary in the 1D model.

13.4.9. Calibration of hydraulic models

Calibration has been carried out for the rivers Broadmeadow, Ballyboghil, Nanny and Sluice. As a general conclusion, the calibrated events are accurately reproduced despite a little overestimation of flow peaks in the Nanny River and a little delay in the peak for the August 1986 event in the Ballyboghil River.

To improve data availability for calibration it is recommended that rainfall stations are installed and monitored. There is a significant lack of operational water level/flow gauging stations in the catchment. As noted in the FEM FRAMS Hydrology Report, it is also recommended that these are reinstalled and made operational.

Similarly, it is also recommended that tidal gauging stations are installed and monitored to provide useable tidal gauge data.

13.4.10. Structured data collection

It is recommended that the OPW's structured flood data collection process is followed in recording flood event information and that the standardised datasheet is filled out on site during or immediately after a flood event. Photographs and actual water level measurements are also important evidence of the flood event. This information and reports from other sources such as the Local Authority, other studies and local people can be found on the OPW flood maps website (www.floodmaps.ie). The field evidence of flooding and flood events is very useful in calibrating hydraulic models.

13.4.11. Joint probability analysis

The joint probability analysis for the study was based on the Defra/EA Technical Report (FD 2308), with further investigation/verification through the hydraulic model sensitivity analysis. The results of the sensitivity analysis showed that the JP combination has a large impact on the fluvial flood maps in the tidally-dominated zone (i.e. downstream of the fluvial/tidal transition point).

As there are no national standards or policy in Ireland to cover flood mapping in the tidal/fluvial transition area, a national policy is required to be implemented by the OPW on joint probability for catchments in Ireland.

13.4.12. Defence asset survey

The location, extent and height of defences in the hydraulic model were based on information in the FDAD and other sources of information available to the project team (i.e. channel and structure cross sectional survey data, LiDAR data and aerial photographs). Table 13-2 provides a list of defences which are included in the hydraulic model and which should be surveyed as part of the next update to the DAS.

Table 13-2 List of defences which are included in the hydraulic model and which should be surveyed as part of the next update of the DAS

N°	Waterbody	Defences	Defence classification
1	Nanny and Paramadden at Duleek	Earth embankment and concrete walls at Duleek along the left bank of the Nanny River and both banks of its tributary, the Paramadden.	Formal

Information on the coastal defences along the Fingal coastline within the FDAD was sourced from the DCFPP. It is recommended that these defences are surveyed as part of future DAS to provide a consistent standard of reporting within the FDAD.

For future projects, it is recommended that a more thorough investigation of defences to be surveyed as part of the DAS should be undertaken at the start of the project. This investigation would identify flood defences which provide flood protection (i.e. the flood defence scheme in Duleek) and reduce the extent of natural river channels and banks which form a large proportion of the data in the FEM FRAMS FDAD.

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List of abbreviations

AAD	Annual Average Damages
AEP	Annual Exceedence Probability
AMS	Annual Maximum Series
AOD	Above Ordnance Datum
APSR	Area of Potential Significant Risk
APMR	Area of Potential Moderate Risk
AU	Analysis Unit
BCR	Benefit Cost Ratio
CFRAMS	Catchment Flood Risk Assessment and Management Study
CFRMP	Catchment Flood Risk Assessment and Management Plan
DAS	Defence Asset Survey
DCC	Dublin County Council
DTM	Digital Terrain Model
EPA	Environmental Protection Agency
ESB	Electricity Supply Board
EU	European Union
FCC	Fingal County Council
FDAD	Flood Defence Asset Database
FRM	Flood Risk Management
HEFS	High End Future Scenario
HPW	High Priority Watercourse
MPW	Medium Priority Watercourse
IRR	Individual Risk Receptor
km	Kilometres
km^2	Square kilometres
LiDAR	Light Detection And Ranging
m	metres
m^3	Cubic metres
MCA	Multi Criteria Analysis

MCC	Meath County Council
MDSF	Modelling Decision Support Framework
mm	millimetres
MRFS	Mid Range Future Scenario
OPW	Office of Public Works
SAC	Special Area of Conservation
SEA	Strategic Environmental Assessment
SPA	Special Protection Area
ERBD	Eastern River Basin District
ERFB	Eastern Regional Fisheries Board
WFD	Water Framework Directive
WTP	Water Treatment Plant
WWTW	Waste Water Treatment Works

Glossary of terms

Term	Description
Annual Exceedence Probability (AEP)	The probability that an event of a specified magnitude will be exceeded in any given year
Bathymetry	The measurement of the depth of water.
Catchment	The total area of land that drains into a watercourse
Critical storm duration	The duration of a storm that produces the greatest extent of flooding in a catchment.
Digital Elevation Model (DEM)	A digital representation of the ground surface topography including buildings and vegetation
Digital Terrain Model (DTM)	A bare earth model of the ground which has all the buildings and vegetation removed
Flood Estimation Handbook (FEH)	Publication giving guidance on rainfall and river flood frequency estimation in the UK
Flood Studies Report (FSR)	Current industry standard for flood studies in Ireland
Flood Studies Update (FSU)	The ongoing updating of the Flood Studies Report in Ireland by the OPW
Floodplain	The land adjacent to a stream or river that experiences occasional or periodic flooding
Fluvial	Related to a river or a stream
Gauged catchment	Catchments in which river flows are measured through the use of a gauge.
Geographical Information Systems (GIS)	Software tools used for, storing, analyzing and managing data and associated attributes which are spatially referenced to the earth.
Hydrograph	A plot of the discharge of water as a function of time.
HX lines	Hydraulic modelling approach used to link 1D and 2D model domains
ISIS	1-D computational hydraulic model developed by Halcrow and HR Wallingford

Term	Description
ISIS .dat files	ISIS compatible text file format
ISIS .ied files	ISIS compatible text file format
ISIS Reservoir unit	ISIS computer model unit used to model floodplain storage. In an unsteady model, it will ensure conservation of mass so that, for example, the overbank spills from a channel are accounted for and may drain back into the main channel as the flood subsides.
ISIS 2D	2-D computational hydraulic model developed by Halcrow
ISIS 1D2D	Linked 1-D and 2-D computational hydraulic model
ISIS-2D (FAST)	ISIS FAST is a new rapid inundation model designed and developed by Halcrow to allow quick assessment of flooding using simplified hydraulics. Its was used in the pluvial analysis
HEFS (High End Future Scenario)	Extreme climate change event, characterised by 30% increase in rainfall, 1000 mm rise in sea level and 400% increase in urbanisation.
Light Detection and Ranging (LiDAR)	An airborne mapping technique which uses a laser to measure the distance between the aircraft and the ground to produce a digital terrain map of the catchment
MRFS (Mid Range Future Scenario)	Most likely climate change scenario, characterised by 20% increase in rainfall, 350 mm rise in sea level and 100% increase in urbanisation.
Muskingum-Cunge method	A particular method for calculating channel-routing in a hydraulic model where a steep section of channel may cause a model to become unstable.
Normal depth downstream boundary	ISIS computer model unit which enables the user to specify a downstream boundary which automatically generates a flow-head relationship based on cross section data.
Pluvial flooding	Flooding form rainfall-generated overland flow, before the runoff enters any watercourses or sewers.
Return period	Measurement indicating the likelihood of a flood event of a certain intensity occurring or being exceeded in any given year
Schematisation	An outline of the hydraulic model.
Spill	ISIS computer model unit which represents out-of-bank flow. Can

Term	Description
	be used to model flow onto floodplains or weirs.
Ungauged catchment	Catchment in which there is no gauge to measure river flows
Unsteady flow simulation	A simulation in which the flow changes with respect to time. Opposite of steady flow, in which flow stays constant with respect to time.
Z Lines	Hydraulic modelling approach used to represent features (i.e. flood defences) in the 2D model domain.