

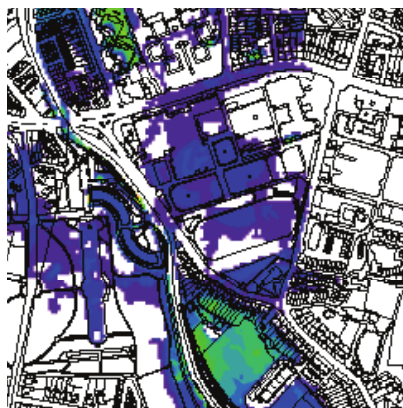
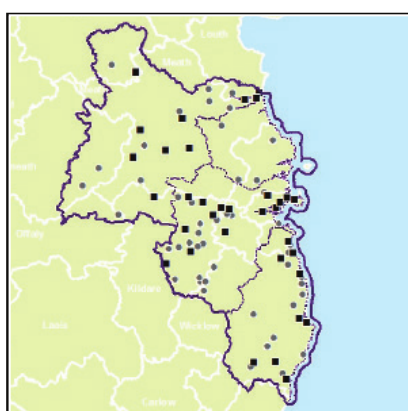
RPS



Eastern CFRAM Study

UoM07 Hydraulics Report

IBE0600Rp0025





Eastern CFRAM Study

UoM07 - Hydraulics Report

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ABBREVIATIONS

AEP	Annual Exceedance Probability
AFA	Area for Further Assessment
AMAX	Annual Maximum
CFRAM	Catchment Flood Risk Assessment and Management
CORINE	Coordination of Information on the Environment
DDF	Depth Duration Frequency
DHI	Danish Hydraulics Institute
hDTM	hydrologically-corrected Digital Terrain Model
ERBD	Eastern River Basin District
ESB	Electricity Supply Board
FARL	Flood Attenuation from Rivers and Lakes
FEM-FRAMS	Fingal East Meath Catchment Flood Risk Assessment and Management Study
FRA	Flood Risk Assessment
FRMP	Flood Risk Management Plan
FRR	Flood Risk Review
FSU	Flood Studies Update
GDSDS	Greater Dublin Strategic Drainage Study
GPU	Graphical Processing Units
HA	Hydrometric Area
HEFS	High End Future Scenario (Climate Change)
HEP	Hydrological Estimation Point
HPW	High Priority Watercourse
ICM	Integrated Catchment Modelling
ICPSS	Irish Coastal Protection Strategy Study
IfSAR	Interferometric Synthetic Aperture Radar
INFOMAR	Integrated Mapping for the Sustainable Development of Ireland's Marine Resource
ISTM	Irish Surge and Tidal Model
LA	Local Authority
LiDAR	Light Detection and Ranging

MHWS	Mean High Water Springs
MPW	Medium Priority Watercourse
MRFS	Mid Range Future Scenario (Climate Change)
NDHM	National Digital Height Model
OD	Ordnance Datum
OPW	Office of Public Works
OSi	Ordnance Survey Ireland
PFRA	Preliminary Flood Risk Assessment
RBD	River Basin District
RMSE	Root Mean Square Error
SI	Statutory Instrument
UoM	Unit of Management

EXECUTIVE SUMMARY

The Office of Public Works (OPW) is currently undertaking a national programme of six river Catchment-based Flood Risk Assessment and Management (CFRAM) Studies, in line with the European Directive on the Assessment and Management of Flood Risks (2007/60/EC) and Irish Law (Statutory Instrument No. 122 of 2010) and to deliver on core components of the 2004 National Flood Policy.

RPS were commissioned to undertake the Eastern Catchment Flood Risk Assessment and Management Study (Eastern CFRAM Study). The objective of this report is to describe the hydraulic analysis undertaken within the Boyne Catchment – Unit of Management 07 (UoM07). It details the development of the hydraulic models used to gain an understanding of the Study area's flood response and mechanisms to assess both flood risk and determine flood risk management solutions.

UoM07 includes ten Areas for Further Assessment (AFAs) which has resulted in the development of eight separate models for flood risk analysis. A single model was developed for the Drogheda, Baltray & Mornington AFAs, due to their proximity and interaction.

The hydraulic analysis utilised computational modelling software informed by detailed topographical survey information (channel sections, in-channel/flood defence structures, bathymetric and floodplain), combined with hydrological inputs (riverine inflows and sea levels) and water-level control parameters (such as channel-roughness), to determine flood hazard.

The principal modelling software package used is the MIKE FLOOD software shell which was developed by the Danish Hydraulics Institute (DHI). This provides the integrated and detailed modelling required at a river basin scale and provides a 1-/2- dimensional interface for all detailed hydraulic model development thus enabling seamless integration of fluvial and coastal models in the AFAs for which this is required.

For the Drogheda, Baltray & Mornington model, an integrated catchment modelling approach was chosen using InfoWorks ICM. This software provides a single platform to incorporate both urban and river catchments using fully integrated 1D and 2D hydrodynamic simulation techniques. Tidal boundaries are applied to both 1D and 2D model domains to incorporate tidal inundation where appropriate.

Key flood events, where available, were used in the calibration of each model whereby the model was reviewed in order to make sure historic flooding is accurately represented. The principal model parameters that were reviewed and amended during the model calibration process are:

- Bed and floodplain roughness coefficients;
- Structure roughness and head loss coefficients;
- Timing of hydrographs;

- Magnitude of hydrographs;
- Incorporation of additional survey information (e.g. additional cross-sections or missed structures).

The calibrated models were used to simulate present day and future flood hazard conditions for events with a range of annual exceedance probabilities (AEP). There are inherent assumptions, limitations and uncertainty associated with hydraulic modelling, which are detailed for each hydraulic model within this Report. There were no defence failure scenarios required. Sensitivity tests have been conducted for each model. The parameters selected for the sensitivity analysis were dependent on the specific model but generally included:

- roughness coefficients
- 2D domain grid cell size
- critical structure coefficients
- flow inputs
- operation of dynamic structures
- downstream boundary conditions
- representation of buildings in 2D model domain
- timing of tributaries
- flow volume

A series of flood extent, zone, depth, velocity and risk-to-life maps known collectively as flood hazard maps were generated based on the model results. The outputs from the hydraulic assessment will inform the subsequent stages of this study - the models will be used to simulate potential options, facilitating the appraisal of possible flood risk management actions and measures.

1 INTRODUCTION

1.1 BACKGROUND TO STUDY AREA

The Office of Public Works (OPW) commissioned RPS to undertake the Eastern Catchment Flood Risk Assessment and Management Study (Eastern CFRAM Study) in June 2011. The Eastern CFRAM Study was the second catchment flood risk management Study to be commissioned in Ireland under the EC Directive on the Assessment and Management of Flood Risks 2007, as implemented in Ireland by SI 122 of 2010 European Communities (Assessment and Management of Flood Risks) Regulations 2010.

The Eastern CFRAM Study Area covers approximately 6,250 km² and includes four Units of Management (UoM); Hydrometric Area (HA) 07 (Boyne), HA08 (Nanny – Delvin), HA09 (Liffey-Dublin Bay) and HA10 (Avoca-Vartry). There is historical evidence of a high level of flood risk within certain areas of the Eastern CFRAM Study area, with significant coastal and fluvial flooding events having occurred in the past.

HA07 is a predominantly rural catchment with the major urbanised areas being Drogheda and Navan. Within HA07 the OPW has implemented and maintains the Boyne arterial drainage scheme, which has resulted in significant alteration of the natural river channels in some areas to improve conveyance capacity and reduce flooding of agricultural land.

Within HA07 there are ten Areas for Further Assessment (AFA). The principal source of flood risk in HA07 is fluvial flooding, with nine of the ten AFAs being subject to some degree of fluvial flood risk. Tidal flood risk within HA07 is limited to the Boyne Estuary where three AFAs, Baltray, Mornington and Drogheda are considered to have some element of coastal flood risk. The full list of AFAs and HPWs within HA07 is shown in Table 1.1 and Figure 1.1, which also describe the associated flood source, fluvial and/or coastal, requiring assessment under the CFRAM Study.

Table 1.1: Fluvial and Coastal Flood Risk at each AFA

AFA/HPW	Fluvial Risk	Coastal Risk
Athboy	✓	-
Ballivor	✓	-
Baltray	-	✓
Drogheda	✓	✓
Edenderry	✓	
Johnstown Bridge	✓	
Longwood	✓	
Mornington	✓	✓
Navan	✓	
Trim	✓	

Note Mornington AFA is incorporated into the Eastern CFRAM Study reporting under HA07 as it was excluded under the FEM FRAM Study which otherwise addressed HA08.

As indicated by Table 1.1, the principal source of flood risk within HA07 is fluvial flooding. Two AFAs, Drogheda and Mornington, have been identified as requiring integrated analysis to include coastal flooding. Baltray AFA has been identified as only requiring analysis of the coastal flood risk.

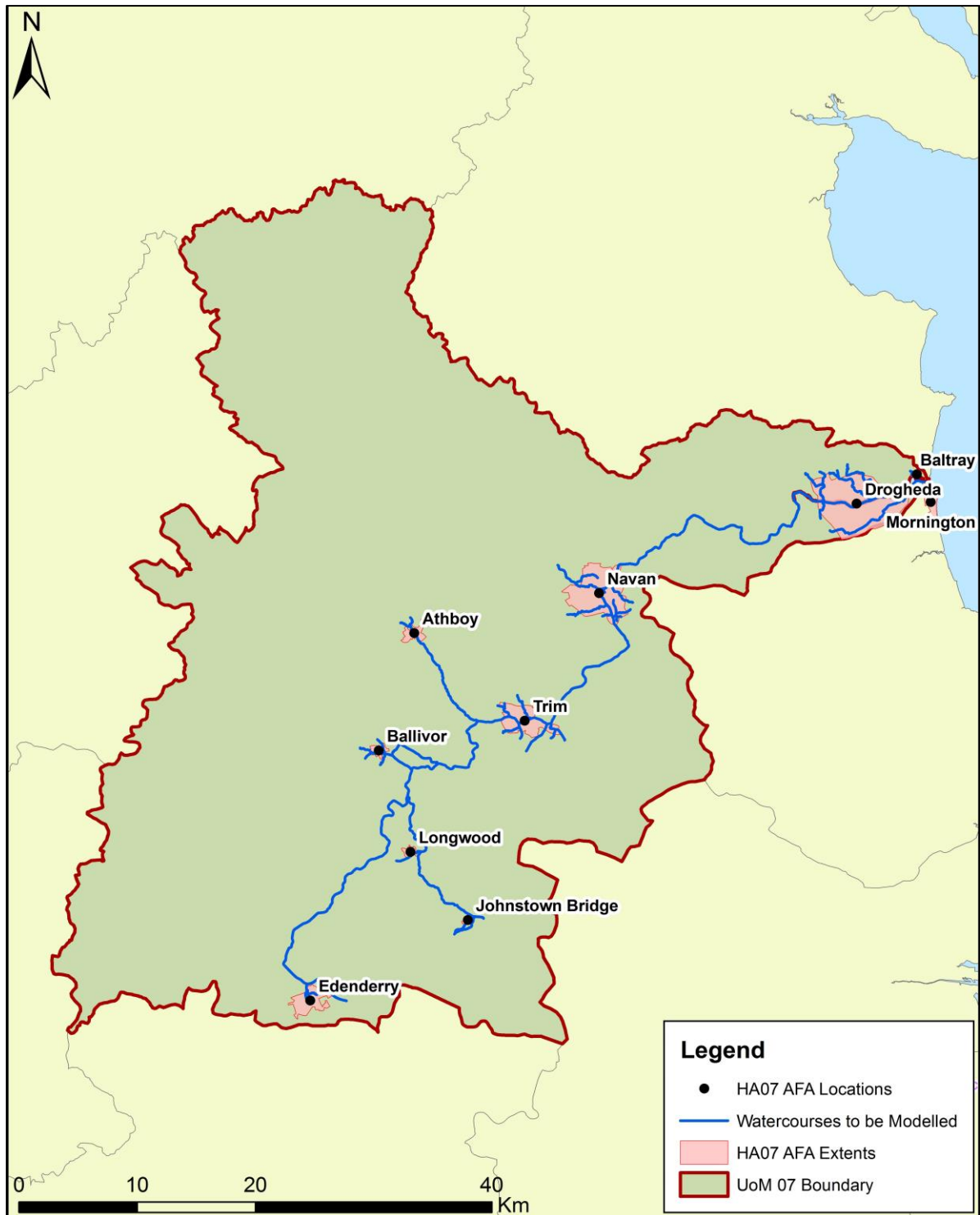


Figure 1.1: HA07 AFA Locations and Extents

1.2 OBJECTIVE OF THIS HYDRAULICS REPORT

The objective of this hydraulics report is to set out the work and analysis undertaken in relation to, and the findings and conclusions of, the surveys and hydraulics analysis as defined within Section 7.8 of the Generic (Stage 1) Project Brief (Ref 2149/RP/002/F, May 2010), hereafter termed “the Stage 1

Project Brief". The report will detail any assumptions made, including the need for such assumptions and their justification, supporting discussion and appended information as necessary.

HA07 includes ten AFAs, (refer to Table 1.1) which has required the development of eight separate models for flood risk analysis. Drogheda, Mornington and Baltray AFAs have been combined into a single model, due to their proximity and interaction. This report has been structured so that each model is reported on in a detailed and concise tabulated manner within Chapter 4. This approach enables the systematic and transparent reporting of every aspect of the hydraulic modelling process, detailing the work that has been undertaken with justification and assumptions clearly stated for each individual model. This avoids unnecessary repetition of generic information relating to all models or HA07 as a whole. Such information is provided within Chapters 1 to 3 to set the scene for the hydraulic analysis and provide ample background information.

The modelling referenced for each of the AFAs under Chapter 4 includes the following topics:

- General Hydraulic Model Information
- Hydraulic Model Schematisation
- Hydraulic Model Construction
- Sensitivity Analysis
- Hydraulic Model Calibration and Verification
- Hydraulic Model Assumptions, Limitations and Handover Notes
- Future Scenarios

This provides an easily accessible single source of reference for each AFA in terms of specific model inputs, approaches and outputs which can be readily utilised in future.

The report does not aim to provide a first principles explanation of hydraulic modelling theory, nor is it intended as a guidance document on how modelling software works.

2 DATA COLLECTION

2.1 BACKGROUND

The process of data collection for the Eastern CFRAM Study as a whole has been ongoing since Project Inception and is detailed in the Eastern CFRAM Study, HA07 Inception Report (IBE0600Rp0003, 2012), hereafter termed “The Inception Report”. Data specific to hydraulic analysis is described as follows.

2.2 HYDROLOGICAL DATA

2.2.1 Fluvial Hydrological Data

The availability of hydrometric data within HA07 is detailed in the Eastern CFRAM Study, HA07 Hydrology Report (IBE0600Rp0012, 2013), hereafter termed “The Hydrology Report”. In general HA07 can be considered to be a relatively well gauged catchment with all but the small Longwood model having at least one hydrometric gauge station with flow data available – see Figure 2.1. Furthermore all nine of these models contain stations which have either:

1. An FSU rating classification indicating confidence in the rating at Q_{med} or;
2. Are subject to rating review such that confidence in the rating at Q_{med} is achieved.

The existing hydrometric data has been utilised as much as possible to inform hydrological analysis and the subsequent derivation of:

1. Historical flood event peak flows and hydrographs – those used for hydraulic model input / boundaries and calibration of each model are detailed in the Inception Report and Chapter 4 of this report, Section 4.1.5 to Section 4.9.5 respectively.
2. Design flows and hydrographs for the required present day Annual Exceedance Probabilities (AEPs) ranging from 50% to 0.1% and future scenarios –refer to Hydrology Report for full details of hydrological analysis and design flow estimation for both gauged and ungauged catchments.

For each gauging station which has data on water levels, this information has been used to inform the model calibration process with further details provided in Chapter 4. The integration of hydrological and hydraulic analysis is at the core of the methodology undertaken in this Study in arriving at final hydraulic modelling outputs. This is discussed further in Section 3.3.7 and detailed per AFA/model under Chapter 4.

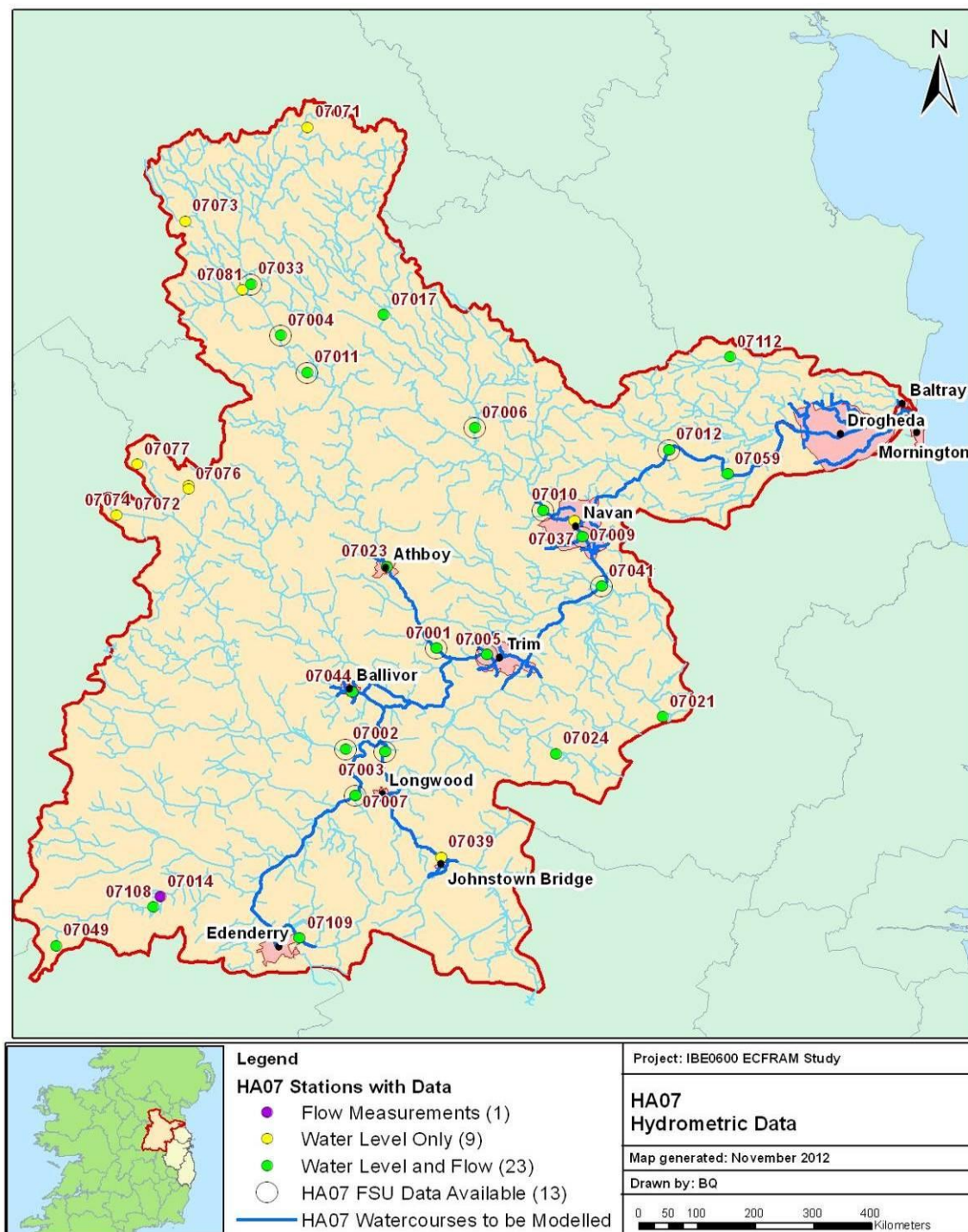


Figure 2.1: Hydrometric Data Availability in HA07

2.2.2 Tidal Data

The Hydrology Report (Section 6.2) discusses the use of tidal data within the Study. This data has been taken from the Irish Coastal Protection Strategy Study (ICPSS) and is discussed further in Section 3.4 of this report.

2.2.3 Rainfall Data

The hydrological analysis undertaken in the Eastern CFRAM Study has utilised rainfall data to supplement the principal CFRAM Study statistical based analysis as follows:

1. Dublin Airport catchment aggregated rainfall radar data has been used as high temporal resolution input data to hydrological models from which design flows and hydrographs were derived in cases where it was deemed useful to augment / supplement existing hydrometric data;
2. Existing daily and hourly rain gauge data (Figure 2.2) was used to ground truth rainfall radar data prior to hydrological model input, and the entire time series were used in the production of simulated flow trace and derived Annual Maximum (AMAX) flow series for the same time period.
3. Where radar data was not available temporally or spatially, existing rain gauge data was used within the hydrological models to produce simulated flow trace and AMAX series for the time period. The Thiessen polygon area weighting method was utilised to derive a rainfall time series which is representative of all of the nearest rain gauges depending on their closeness to the catchment.

Full details of rainfall data analysis and associated hydrological modelling are provided in the Inception Report and Hydrology Report. Further to the analysis contained within the Hydrology Report, rain gauge data is used in this report to provide estimates of the frequency of historical flood events where no observed or simulated hydrometric data was available upon which to base flood flow frequency estimates.

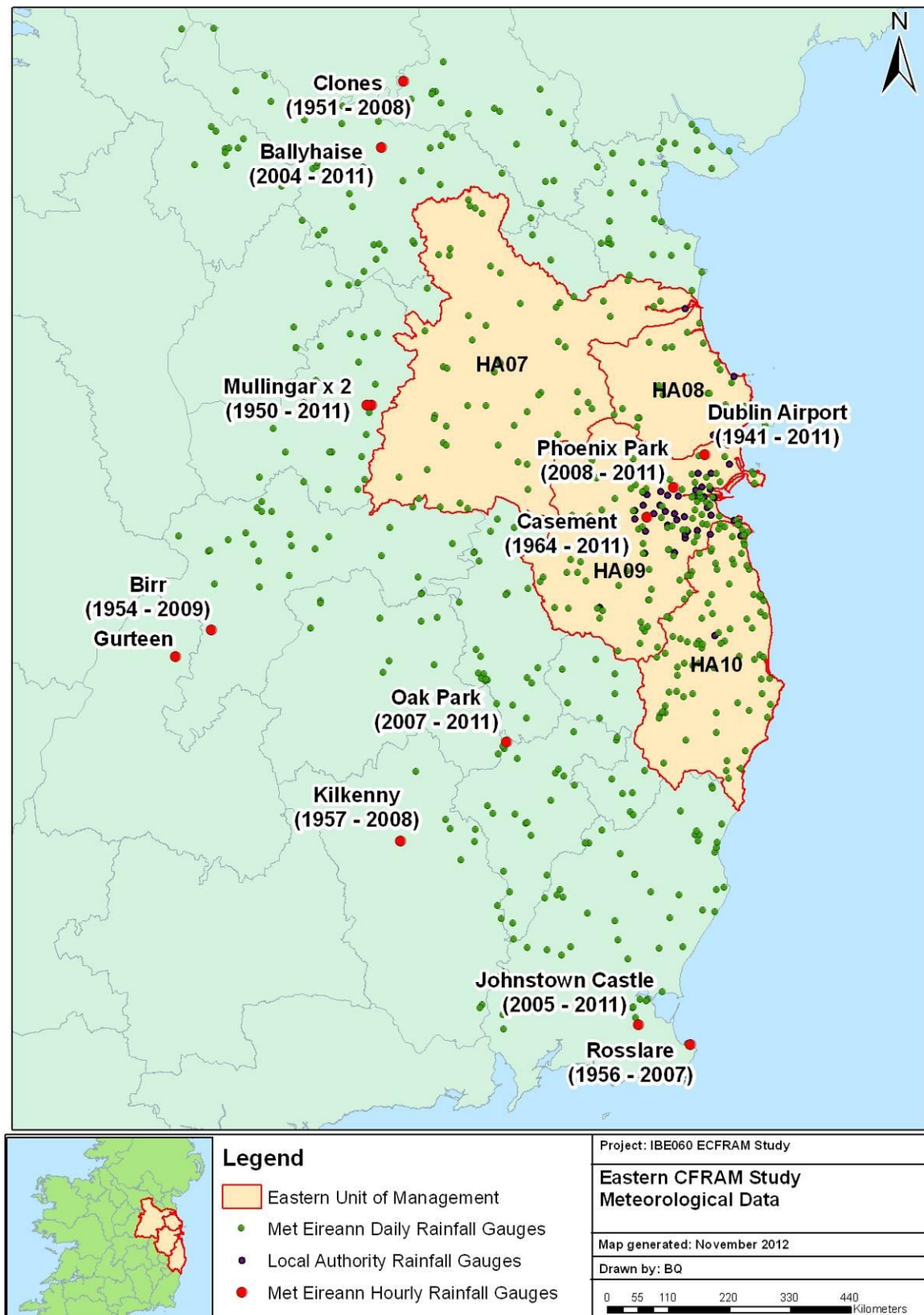


Figure 2.2: Meteorological Data Availability

2.3 TOPOGRAPHICAL SURVEY DATA

2.3.1 Channel and Structure Survey Data

The most significant aspect of data collection since the inception stage of the Eastern CFRAM Study has been the capture of channel and structure survey data to provide cross-section and long-section information (x, y, z spatial coordinates) of river channels and banks, on-line channel structures (bridges, weirs, sluices, etc.) and flood defences (walls, embankments, etc.). This information is necessary for the development of hydraulic models of the High Priority and Medium Priority Watercourses (HPW and MPWs) within HA07.

The specification, procurement and management of the survey contract for HA07 was undertaken by RPS. In order to produce the specification, RPS conducted a walkover survey in each AFA and a desk-based study using GIS datasets was completed. The specific tasks undertaken, all of which will relate to the building of hydraulic models were:

- Establishment of suitable survey control along the survey areas;
- Survey of river channel cross sections, at prescribed locations within the survey areas;
- Survey of relevant structures identified within the survey areas;
- Survey of identified flood defences within the survey areas;
- Delivery of outputs as appropriate to the nature of the survey;

The raw survey data was provided electronically in the following formats:

- ISIS input format (.DAT - also compatible with ICM);
- MIKE input format;
- Cross-section XYZ format;
- Left & Right Bank Only XYZ format: This includes integrated cross-section crest levels, flood defence crest levels and any intermediate bank levels surveyed between cross sections, provided in a separate XYZ file for each bank;
- GIS shapefiles of surveyed watercourse centrelines and channel cross sections with populated attribute tables showing Reach IDs, chainages, and coordinate data;
- AutoCAD drawings;
- Georeferenced site photographs and videos;
- Digital metadata.

Specific details of the survey data received can be found under Chapter 4. The survey contract for HA07 was carried out by Murphy Surveys Ltd between 12/04/2012 and 10/08/2012 within five pre-defined work packages under the contract. Incoming survey data was received and quality checked using the following process:

1. Quality assurance check by RPS including inspection of selected sample data on GIS using a checklist and comparison with specification in terms of surveyed reach alignment and length; cross section quantity; surveyed structures; flood defences; survey completeness and quality of deliverables;
2. Upon detection of a suspected error or omission, a survey query sheet was completed and submitted to the Surveyor;
3. If the survey query response confirmed that survey data was missing that was required for hydraulic modelling, then this was procured as additional works under existing survey contracts.

All survey data used within each AFA/Model are listed under Chapter 4, including digital data folder structure, file names, folder references; any survey issues identified (survey queries) and details of survey query resolution. The details are provided under the relevant AFA/Model under Chapter 4 (Sections refer 4.1.2 to 4.9.2, Item (8) respectively for each of the nine AFAs).

The survey issues identified within HA07 are summarised as follows:

- Drogheda: Cross-sections and culvert details at the Old Mill site on the north side of the river Boyne in Drogheda were requested under Infill Survey, as they had not been included in the original survey data received. The infill survey data received was included in the model.
- Edenderry: The surveyed river centreline on the Weavers Drain did not agree with the OSi vector mapping. A review of the centreline found it to be inaccurate. Weavers Drain centreline was therefore corrected and updated in the network file. The cross section chainage was also adjusted to place the cross sections in the right location.
- Mornington: It was noted that the development called Northlands was constructed since the Mornington scheme was completed and as such the watercourse and hence 1D section through this area is considered to have changed (partially culverted). Although this area is located outside the AFA extents, details of the culverts were requested under Infill Survey and included in the model.
- 'Glass Walls' within the Navan, Trim, Drogheda, Edenderry, Johnstown, Longwood and Athboy models: Glass walls can occur along the 1D only reaches of a model when the simulated water level exceeds the surveyed ground level at the extent of the cross-section. This may result in an artificially high simulated water level as the full extent of the cross-section is not represented within the model. Where 'glass walls' have been identified during model construction, the relevant cross-sections have been extended based on LiDAR (where

available) or the NDHM (Section 2.3.4). This process was conducted using ArcGIS to generate an ASCII file (based on LiDAR and NDHM data). This file was used as a Digital Elevation Model and was imported, along with a copy of the model, to MIKE 2014. This version of the software has a tool which allows cross-sections to be extended to a defined point (where the 'glass wall' effect no longer occurs) without intersecting other cross-sections. Further detail on the reaches where cross-sections were extended is provided for the relevant models in Chapter 4.

Digital Survey Data is also provided as an accompaniment to this report.

Raw survey data has not been converted for the purposes of the CFRAM Study since its provision was already in the format compatible with direct import to hydraulic modelling software.

2.3.2 Floodplain Survey - LiDAR

The Stage 1 Project Brief indicated that the OPW would supply the results of a floodplain survey by November 2011; however delivery of some of the processed floodplain survey information was delayed until October 2012 due to weather issues during the fieldwork period. All floodplain survey data undertaken as part of this Study was received by September 2012. This survey utilised airborne laser scanning technology (LiDAR - Light Detection and Ranging). The Inception Report has already discussed how RPS provided input into the required coverage of this survey. On receipt of the LiDAR information, RPS checked its readability, resolution and validated the extent of its spatial coverage. This was efficiently performed via the superimposition of multiple ESRI ArcGIS shape-files of the data. This methodology allowed for rapid visualisation and subsequent identification of any geographical inadequacies. If supplied LiDAR information was found to contain insufficient coverage of AFAs and HPWs (areas designated for two-dimensional modelling); then these areas were targeted for rectification and additional LiDAR requested. Figure 2.1 illustrates the extent of LiDAR coverage in HA07 in relation to modelled watercourses.

The DTM derived from the received LiDAR data was assumed to meet the vertical accuracy as specified in the Stage 1 Project Brief - 0.2m RMSE. Given the quality of the received surveys, additional manipulation or post-processing work was not required for the LiDAR data at HA level.

The LiDAR for UoM07 was considered fit for purpose by RPS. However, post-processing was required in one area where visual inspection identified an error in the Drogheda LiDAR data between the Drogheda and Mornington AFAs. This was corrected using LiDAR data from the extended LiDAR data survey covering the southern portion of the Mornington AFA - details have been provided under Chapter 4.

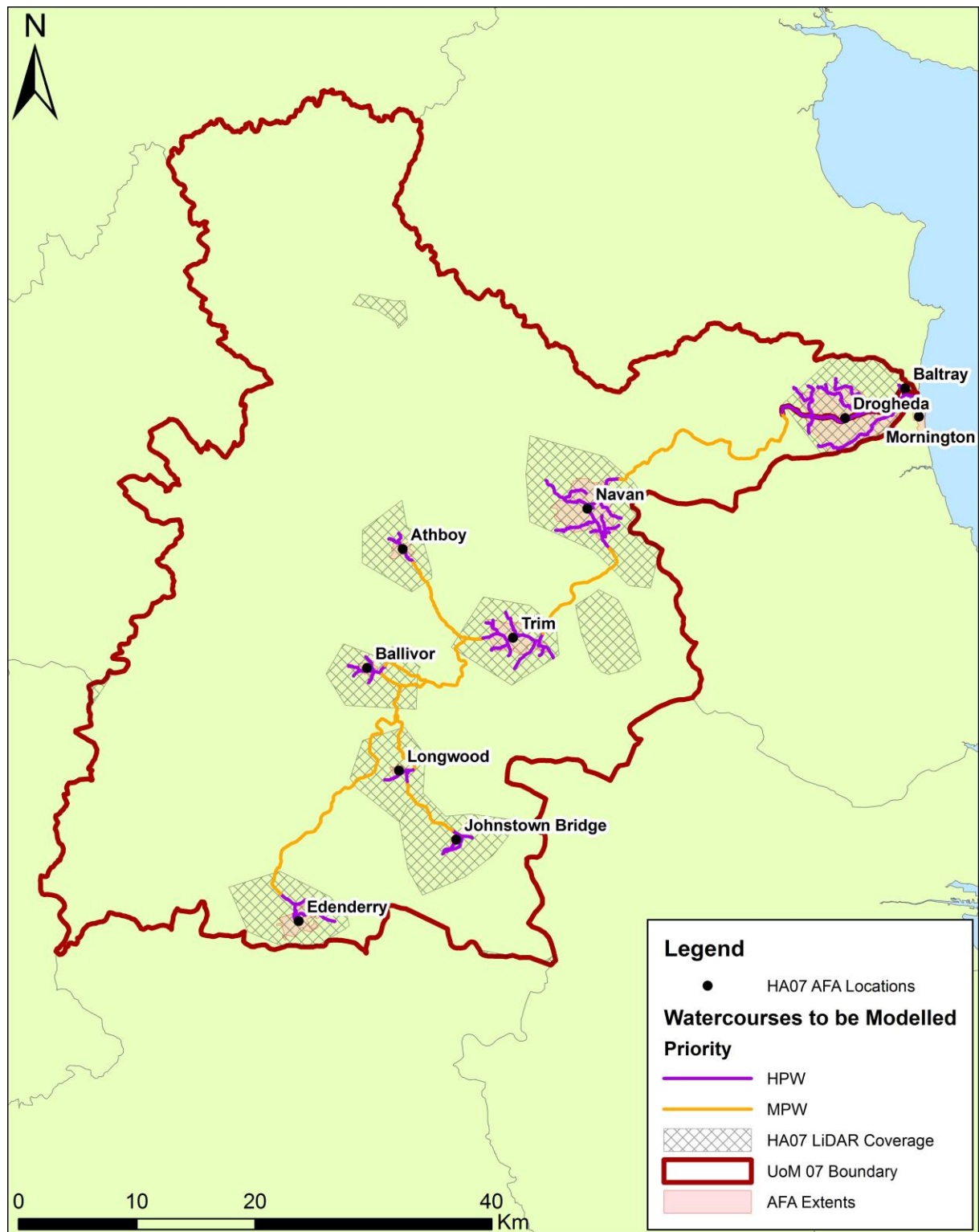


Figure 2.3: Extent of LiDAR Coverage in HA07

2.3.3 Coastal Bathymetry

Bathymetric data was required for all models located within areas of tidal influence. Those areas with no direct coastal inundation required basic bathymetric data in the vicinity of the mouth of the relevant rivers. Those areas subject to complex coastal inundation required more detailed and extensive bathymetric data. In those cases, sufficient offshore data was required to represent the various

channels, drying zones and offshore banks within the model domain. Details of coastal bathymetry data used per model are included under Chapter 4.

Some parts of the bathymetry information used in the models was obtained from INFOMAR survey data, a joint venture between the Geological Survey of Ireland (GSI) and the Marine Institute, supplemented with Admiralty Chart data, as digitally supplied by C-Map of Norway – see Table 2.1.

Table 2.1: Bathymetric datasets used in each model

Model	Bathymetric datasets used
Baltray	<ul style="list-style-type: none"> • LiDAR
Drogheda	<ul style="list-style-type: none"> • LiDAR
Mornington	<ul style="list-style-type: none"> • LiDAR

The OPW LiDAR data provided as part of this Study, in conjunction with the OPW LiDAR commissioned as part of the Irish Coastal Protection Strategy Study (ICPSS), along with significant numbers of more localised hydrographic surveys already in existence, were used to provide specific information for inshore and overland areas. Where necessary, the OPW LiDAR data was trimmed to the Ordnance Survey Ireland (OSI) High Water Mark, in order to remove areas containing water level elevations, rather than bare earth data.

In areas where no other data was available, the National Digital Height Model (NDHM) was included in the models, although it was noted that it is of lesser accuracy to the OPW LiDAR data.

RPS processed and quality checked all bathymetric data to ensure its suitability for use within the modelling systems, consistently ensuring that any model interpolation processes produced valid meshes which were representative of the input data. This was a manual process where the modeller inspected the model bathymetry files to ensure that the relevant features were adequately represented within the model. Where relevant, buffers were used between adjoining datasets in order to ensure a smooth transition, and additional interpolated data was included in locations where data was unavailable. Bathymetry data at boundary locations and transition areas between 1D and 2D model components was also edited where necessary in order to prevent boundary drying and achieve model stability. The datum of bathymetric datasets was checked and it was verified that levels between adjacent datasets were consistent. The data, having been checked, was deemed appropriate for use in the models.

2.3.4 Other Digital Elevation/Terrain Models

As detailed in the Inception Report, the OPW provided National Digital Height Model (NDHM) (5m resolution IfSAR) data covering HA07 in its entirety at the project outset.

In addition to this data, the OPW also provided hydrologically-corrected Digital Terrain Model data (hDTM). These datasets, which are hydrologically corrected and presented in a 20m resolution, cover the entire spatial extent of HA07.

On receipt of this information, RPS reviewed the datasets in order to check for adequate project areal coverage. As the xyz data had already been converted into ESRI Grid files, no further post-processing was required for geographical data visualisation. Where localised post-processing work has been undertaken at an AFA/Model level, the details have been provided in Chapter 4.

2.4 DEFENCE ASSET DATABASE

No known flood defence assets within HA07 were identified within the tender brief; however subsequent scoping visits undertaken by RPS identified a number of assets which were reported on in the Inception Report. The geometric survey of these assets, along with the identification and geometric survey of additional flood defence assets, was a requirement of the HA07 channel and cross section survey contract.

On receipt of the survey contract deliverables in mid 2012, RPS extracted the identified assets and circulated mapping and shapefiles to the Eastern CFRAM Study Progress Group Local Authorities/Regional OPW representatives within HA07. Further confirmation of the assets was received including the as-constructed details of the recently completed Mornington flood defences, which informed the scope of the condition survey and subsequent defended/undefended model analysis.

Table 2.2 summarises the agreed list of flood defence assets for HA07. The hydraulic performance and effectiveness of these assets were tested within the models under Chapter 4.

Following discussions via the Floods Directive National Technical Coordination Group, the OPW confirmed a CFRAM Defence Asset Database spreadsheet storage format on 9 December 2013. RPS populated this format with the condition survey, which commenced in the first quarter of 2014 following project-level trialling with the OPW engineering staff.

Table 2.2: Flood Defence Assets identified for HA07

Location	AFA	Structural Form	Identification Stage
Abbey Lane	Trim	Wall	OPW Desktop Study
Ballivor	Ballivor	Walls	Inception
Baltray	Baltray	Coastal Flood Wall	Inception
Boyne Hill	Navan	Embankment	Survey
Bridge Street	Trim	Wall	OPW Desktop Study
Church Road	Drogheda	Wall	Survey

Location	AFA	Structural Form	Identification Stage
Drogheda	Drogheda	Walls and Embankments	Inception
Drogheda Port	Drogheda	Wall	Survey
Drogheda Road	Drogheda	Wall	Survey
Dunderry Road	Trim	Wall	OPW Desktop Study
Dunfieth Park	Johnstown Bridge	Wall	OPW Desktop Study
Frenchs Lane	Trim	Wall	OPW Desktop Study
Glebe Wood	Ballivor	Wall	OPW Desktop Study
Hamilton Place	Trim	Wall	OPW Desktop Study
Johnstown Bridge	Johnstown Bridge	Walls	Inception
Lackanash Estate	Trim	Wall	OPW Desktop Study
March Road	Drogheda	Wall	Survey
Maryville	Navan	Wall	Survey
Mornington	Mornington	Walls and Embankments	Inception
Mornington Road	Drogheda	Wall	Survey
N1 / Rathmullan Road	Drogheda	Wall	Survey
Navan	Navan	Walls	Inception
North Strand	Drogheda	Wall	Survey
Parkstown View	Ballivor	Wall	OPW Desktop Study
Patrickswell Lane	Drogheda	Wall	Survey
Queensborough	Drogheda	Wall	Survey
R154 Scurlockstown	Trim	Wall	OPW Desktop Study
Rathmullan Road	Drogheda	Wall	Survey
River Boyne, Dublin Road	Navan	Wall	Survey
Rivermill View	Navan	Wall	Survey
Strand	Drogheda	Embankment	Survey
Summer Ville	Navan	Wall	Survey
Talbot Castle	Trim	Wall	OPW Desktop Study
The Glebe	Johnstown Bridge	Wall	OPW Desktop Study
The Mall	Drogheda	Wall	Survey
The Sycamores	Drogheda	Wall	Survey
Trim	Trim	Walls	Inception

Location	AFA	Structural Form	Identification Stage
Wellington Quay	Drogheda	Wall	Survey
Swan River	Navan	Embankments	Model Construction

2.5 LAND USE DATA

The Environmental Protection Agency (EPA) GIS Dataset “Coordination of Information on the Environment” known as CORINE was provided at the project outset (7th June 2011 from the OPW) for the most recent version in 2006 (500m grid resolution). The CORINE Land Cover (CLC) is a map of environmental landscape based on interpretation of satellite images. There are five broad levels of land use classification:

1. Artificial Surfaces
2. Agricultural Areas
3. Forest and semi-natural areas
4. Wetlands
5. Water Bodies

These categories are further broken down into 44 classes of specific land use and were provided as a GIS polygon shapefile covering the Eastern CFRAM Study area. This data was used in the hydraulic modelling phase to define catchment roughness parameters as detailed in Section 3.3.5.

3 HYDRAULIC MODEL METHODOLOGY

3.1 INTRODUCTION

Hydraulic analysis is a critical part of a CFRAM Study. The objective of hydraulic analysis is to gain a detailed understanding of the Study area's flood response and mechanisms to assess both flood risk and determine flood risk management solutions. The accuracy of the models representing existing conditions in terms of flood level, depth, extent and flow velocity allows the possible benefits of flood options to be meaningfully assessed, allowing the appropriate actions/decisions to be taken. To achieve such accuracy; detailed hydraulic modelling analysis (including comparison with historical flood events) and estimation of design and future flood level, depth, velocity and extent conditions, has been undertaken for each AFA. This analysis takes account of factors influencing in-stream flow and water level, such as the effect of lake and floodplain retention and control structures.

The principal modelling software package that has been used is the MIKE FLOOD software shell (refer to Section 3.3.1), which was developed by the Danish Hydraulics Institute (DHI). This provides the detailed modelling required at a river basin scale and provides a 1-dimensional/2-dimensional interface for all detailed hydraulic model development. By adopting MIKE FLOOD, a series of fully dynamically linked 1-dimensional/2-dimensional models have been developed, thereby incorporating a degree of flexibility into the extent of coverage of the 1-dimensional and 2-dimensional elements within each area. The MIKE FLOOD software shell comprises MIKE 11 for 1-dimensional modelling (fluvial applications) and MIKE 21 for 2-dimensional modelling (fluvial and coastal applications), thus enabling seamless integration of fluvial and coastal models in the AFAs for which this is required.

There is one model in which the use of MIKE FLOOD was replaced with ICM, following agreement between RPS and the OPW (refer to Table 3.1). ICM (Integrated Catchment Modelling) is a 1D/2D dynamically linked modelling package developed by Innovyze (formerly MWH Soft and Wallingford Software) and is an integrated platform which enables both above and below ground drainage systems to be modelled in one package. A full and detailed representation of the natural open channel systems can therefore be augmented with the man made open and culverted systems of the urban environment to take account of anthropogenic changes to the river catchment. ICM utilises a flexible triangular 2D mesh to model overland flow. The use of flexible mesh allows the mesh size to be varied throughout the model, providing detail where required. RPS completed a technical note in April 2012 detailing the reasoning behind choosing Infoworks ICM over MIKE FLOOD for selected culverted urban watercourse modelling, "CFRAMS Technical Note 4, Culverted Watercourse Modelling". The details and justification of methodology are included in Technical Note 4 and summarised as follows:

- Walkover surveys and subsequent channel and structure survey data in the Eastern CFRAM Study Area (particularly Drogheda) identified a number of HPWs within AFAs that are partially culverted over significant lengths;

- MIKE 11 or ISIS (both of which are included on the OPW's pre-approved list of modelling software) is not suited to modelling these extensive sections of culverted watercourse or sections of watercourse which vary continually between open channel and culvert;
- Infoworks ICM has been recently developed by Innovyze to specifically model the integration of piped networks and open channel flow. RPS therefore consider it best placed to model these partially culverted watercourses;
- Infoworks has been developed from the ISIS engine and is therefore not a significant departure from the pre-specified software for CFRAM Studies. In addition it can provide mapping output as specified in Section 7.5 of the Stage 1 Project Brief and can directly utilise the survey data that has been captured for HA07.

Infoworks ICM has also been utilised for modelling coastal flood risk within the same AFAs for which it was used to model fluvial flood risk (refer to Table 3.1). This enables seamless integration of both models for flood mapping. For consistency Infoworks ICM has been used for all coastal AFAs within HA07.

The subsequent sections of this Chapter describe the overall conceptualised models and detail the key aspects of each modelling software package used, including model inputs, how channel structures are represented and model parameters selected. The integration of the hydraulic analysis with the hydrological analysis undertaken previously is also outlined, with AFA/HPW specifics provided where relevant under Chapter 4.

3.2 MODEL CONCEPTUALISATION

The Inception Report (Chapter 5) and the Hydrology Report (Chapters 4 and 6.3.1) outline the hydraulic model conceptualisation process which resulted in nine hydrodynamic models within the HA07 UoM. AFA/HPW specific model conceptualisation, including modelling software used is detailed in Chapter 4 of this report and summarised in Table 3.1 below.

All HPW's have been modelled as 1D-2D, with MPW's normally modelled as 1D only. Links are provided, normally at the top of each river bank, to allow water to pass between the 1D and 2D model domains. The number and boundaries of the models have been largely chosen due to modelling practicalities such as having one 2D mesh per model and therefore one AFA per model and where possible such that gauge stations separate models and therefore can be used to directly calibrate flow estimations on both models.

The downstream boundaries of the Athboy and Ballivor models are located at the confluence of the River Boyne with the Athboy River and the Ballivor and Stoneyford Rivers respectively. The downstream boundaries of the Johnstown Bridge and Longwood models are also located at the confluence of the River Boyne with the Blackwater River, for both models. The Johnstown Bridge model is located upstream of the Longwood model and incorporates an overlapping reach of the

Blackwater River from Longwood to the confluence with the River Boyne. Whereas the Longwood model has modelled this reach in a combination of 1D-2D modelling and 1D modelling, the Johnstown Bridge model has modelled it using 1D modelling only and is therefore considered to be less accurate. The presentation of the Johnstown Bridge flood extents therefore ends just upstream of the Longwood model and the remainder of the Blackwater River is presented in the Longwood model. The Johnstown Bridge model was also used to provide the upstream flow hydrograph in the Blackwater River for the Longwood model. Further details are provided in Chapter 4.5 and Chapter 4.6.

The River Boyne, from Edenderry to the Boyne estuary, is split across four models, as shown in Figure 3.1. These models are Edenderry, Trim, Navan and Drogheda. The OPW gauging station called Boyne Aqueduct (07007) is located at the downstream boundary of Edenderry Model (and the upstream extent of the Trim model) just west of Longwood and on the Boyne main channel where the Royal Canal traverses the river. The EPA gauging station called Ballinter Bridge (07041) is located at the downstream boundary of Trim Model (and the upstream extent of the Navan model) just south of Navan. The downstream boundary of the Navan model (and upstream boundary of the Drogheda model) is located at the Broadboyne Bridge (located north-east of Navan). Drogheda, Baltray and Mornington have been assessed within a single model due to the geographical proximity and hydraulic connectivity between the AFA.

MIKE FLOOD software has been selected for all of the models within HA07, with the exception of Drogheda, Baltray and Mornington (Infoworks ICM). AFA/HPW specific model conceptualisation is detailed under Chapter 4 of this report and summarised in Table 3.1 below. Figure 3.1 illustrates the extent of fluvial models and also the AFA locations. Figure 2.1 shows the location of Hydrometric Gauging stations throughout the catchment.

Table 3.1: HA07 Model Conceptualisation

Chapter 4 Reference	AFA/HPW	Fluvial Risk	Coastal Risk	Fluvial Model Software	Coastal Model Software	Comments
4.1	Athboy	✓	-	MIKE FLOOD (2011)	-	Rectangular grid
4.2	Ballivor	✓	-	MIKE FLOOD (2011)	-	Rectangular grid
4.3	Drogheda & Baltray	✓	✓	INFOWORKS ICM (v6.5)		3 AFAs in a Single Model including Mornington, which is reported separately. Flexible Mesh
4.4	Edenderry	✓	-	MIKE FLOOD (2011)	-	Rectangular grid
4.5	Johnstown Bridge	✓	-	MIKE FLOOD (2011)	-	Rectangular grid
4.6	Longwood	✓	-	MIKE FLOOD (2011)	-	Rectangular grid
4.7	Mornington	✓	✓	INFOWORKS ICM (v6.5)		Flexible Mesh
4.8	Navan	✓	-	MIKE FLOOD (2011)	-	Rectangular grid
4.9	Trim	✓	-	MIKE FLOOD (2011)	-	Rectangular grid

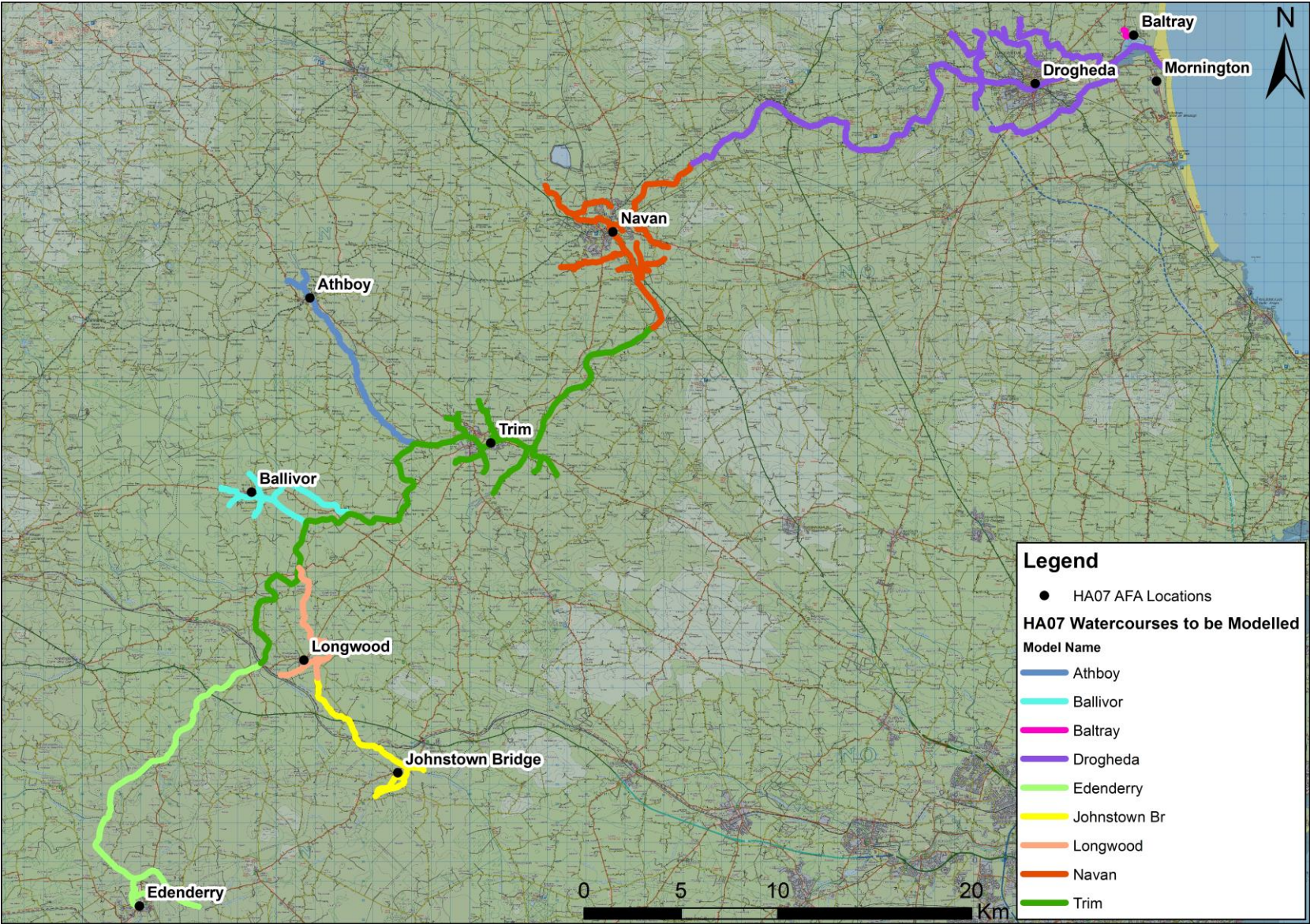


Figure 3.1: HA07 Modelled Watercourses and AFAs

3.3 FLUVIAL MODELLING

3.3.1 Fluvial Model Software – MIKE FLOOD

The MIKE FLOOD modelling system was utilised for seven models, the details of which are included under Chapter 4.

MIKE FLOOD is a software shell comprising the following two components:

- A 1-dimensional river model (MIKE 11 HD) to describe the flow in linear rivers and channels
- A 2-dimensional model (MIKE 21 HD) to describe the free surface flow in the river floodplain.

MIKE FLOOD integrates the one-dimensional model and the two-dimensional model into a single, dynamically coupled modelling system. This enables the best features of both model types to be utilised, whilst at the same time avoiding many of the limitations of resolution and accuracy encountered when using either model separately.

The integration of MIKE11 and MIKE21 is provided by a series of lateral links, on both the left and right banks. Each lateral link allows a string of MIKE21 cells to be laterally linked to a defined reach in the MIKE11 model. These links are used to simulate overflow from the river channel onto a floodplain. MIKE FLOOD provides options to adjust the parameters associated with each link including friction, weir coefficient, calculation type and source of flooding i.e. water transfer occurs when the water level exceeds the highest of the MIKE21 cell level or the marker level in MIKE11. These parameters are set as the default values unless otherwise specified in Chapter 4.

Standard links may also be used, where one or more MIKE21 cells are linked to the end of a MIKE11 river branch. These links are used to connect the MIKE21 grid / mesh into a broader MIKE11 network. The third main type of link is the structure link which is used to simulate structures within the MIKE21 model (instead of the MIKE11 model). These links can be used to simulate structures which are remote from the river but convey water when flooding occurs.

The 1-D hydrodynamic models constructed within comprise a Simulation Editor file which contains details of the simulation and provides a link to other MIKE11 editor files. For each hydraulic model created, the simulation editor has the following input files:

- A Network Editor file (see example given by Figure 3.2) containing the location of the river channel and any branches and details of hydraulic structures on the river (weirs, culverts, bridges etc.) in the tabular view;
- A Cross-Section Editor file containing all river channel cross-sectional information;
- A Boundary Editor file (see example given by Figure 3.3) containing all boundary conditions applied to the model including an upstream input discharge hydrograph for each watercourse,

a specified downstream boundary and a number of point / distributed discharge hydrographs along the length of the river;

- A Hydrodynamic Editor file containing details of the hydrodynamic parameters adopted in the simulations.

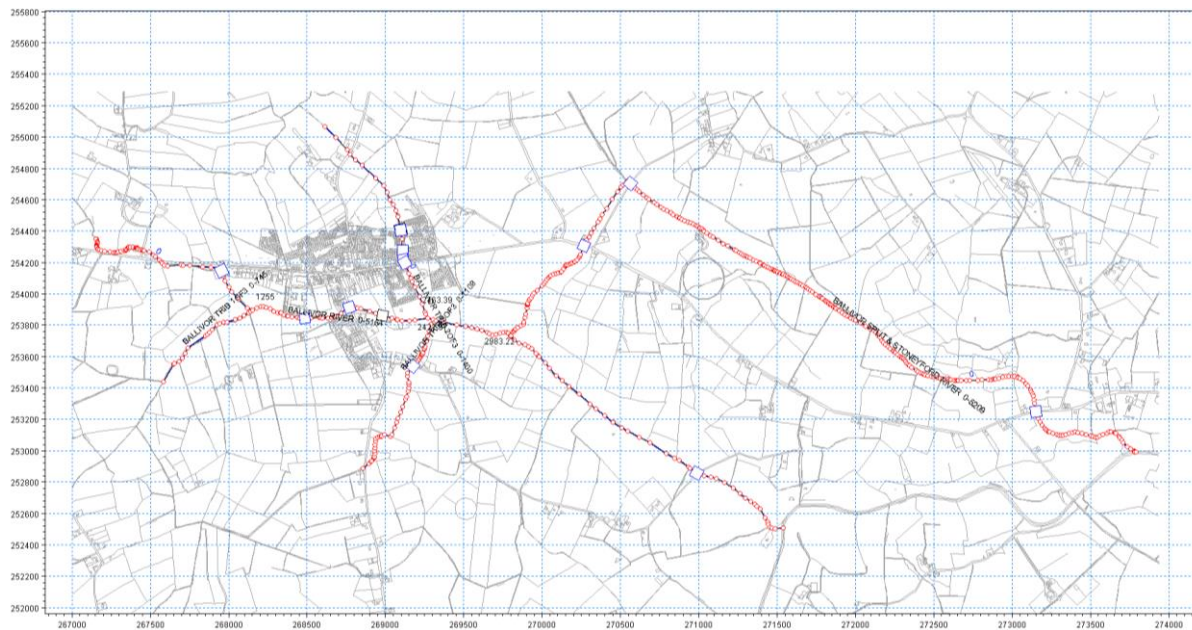


Figure 3.2: Example MIKE 11 Network Editor File

	Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
1	Open	Inflow	BALLIVOR RIVER	0	0		
2	Open	Inflow	BALLIVOR TRIB 10F3	0	0		
3	Open	Inflow	BALLIVOR TRIB 20F3	0	0		
4	Open	Inflow	BALLIVOR TRIB 30F3	0	0		
5	Point Source	Inflow	BALLIVOR RIVER	3538.64	0		
6	Point Source	Inflow	BALLIVOR SPLIT & STONEYFORD RIVER	1271	0		
7	Distributed Source	Inflow	BALLIVOR RIVER	0	2120		
8	Distributed Source	Inflow	BALLIVOR SPLIT & STONEYFORD RIVER	0	5154		
9	Distributed Source	Inflow	BALLIVOR RIVER	2983.22	3538.64		
10	Distributed Source	Inflow	BALLIVOR TRIB 20F3	0	1400		
11	Distributed Source	Inflow	BALLIVOR TRIB 30F3	0	1109		
12	Open	Q-h	BALLIVOR RIVER	5164	0		
13	Open	Q-h	BALLIVOR SPLIT & STONEYFORD RIVER	5209	0		

Figure 3.3: Example Boundary Editor File

The input files for the 2D- MIKE21 models are the topography file and the resistance file – further details provided in Section 3.3.4 and Section 3.3.5 respectively.

3.3.2 Fluvial Model Software – Integrated Catchment Modelling (Infoworks ICM)

InfoWorks ICM was selected for one model. This software provides a single platform to incorporate both urban and river catchments using fully integrated 1D and 2D hydrodynamic simulation techniques. It models the hydrological cycle from rainfall input to the catchment discharge point

allowing all flow paths to be represented accurately. Tidal boundaries are applied to both 1D and 2D model domains to incorporate tidal inundation where appropriate.

ICM incorporates the following elements:

- Integration of 1D watercourse and 2D floodplain – used for one model (refer to Chapter 4) or
- Integration of 1D watercourse, 2D floodplain and representative existing storm drainage network – used for the Drogheda portion of the ICM model in HA07.

River channels are defined using surveyed watercourse cross sections for the 1D element as illustrated by Figure 3.6. Banks are defined as lateral spills using either survey or LiDAR data (defines the linkage from the open channel 1D domain to the 2D domain). For Infoworks ICM, there is a single network file which incorporates all of the components within the 1D and 2D model domains.

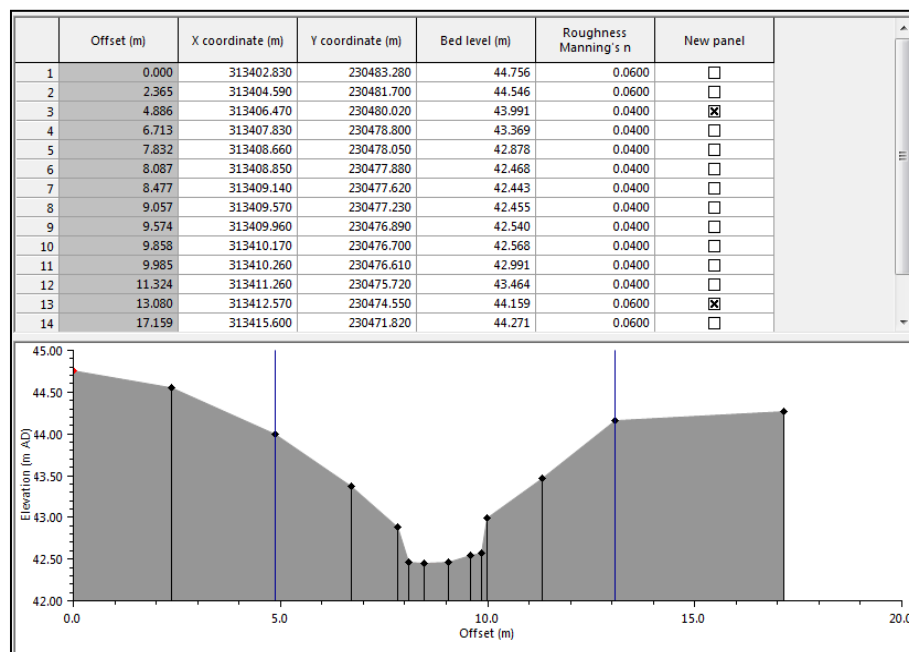


Figure 3.4: ICM 1D Model Inputs – Channel Cross Sections

3.3.3 In Channel Structures

For MIKE models, in-channel structures have been incorporated through the network file (tabular view). The geometry of irregular shaped culverts and bridges are normally defined by 'Cross-Section DB', with regular shaped culverts defined as being circular or rectangular. The 'Cross-Section DB' and Level-Width options have both been employed when installing weirs.

In terms of model stability, the MIKE software developers (DHI) advise that culverts are more stable than bridge structures in MIKE and that culverts (and weirs to allow overtopping of the structure) should be inserted as a proxy for bridges when possible. There is no difference between defining the geometry of the culvert in the model or using a cross-section in the cross-section file (Cross-section

DB). DHI also recommend using a series of closed cross-sections to represent long culverts instead of a structure in the network file, as this approach more accurately represents frictional effects.

For Infoworks ICM models, in-channel control structures i.e. weirs, bridges, sluices etc. can be defined using their specific geometry as illustrated by Figure 3.5. Infoworks ICM can accurately simulate spills from piped networks, overland routing of flow and then re-entry of overland flow back into the watercourse. This is a particularly important feature when assessing urban flow paths. Infoworks ICM is also able to incorporate changes in culvert gradients which are essential for accurately simulating hydraulic performance. Spill units are incorporated into the model where culverts are subject to overtopping – this feature is contained within bridge structures and so spill units are not required.

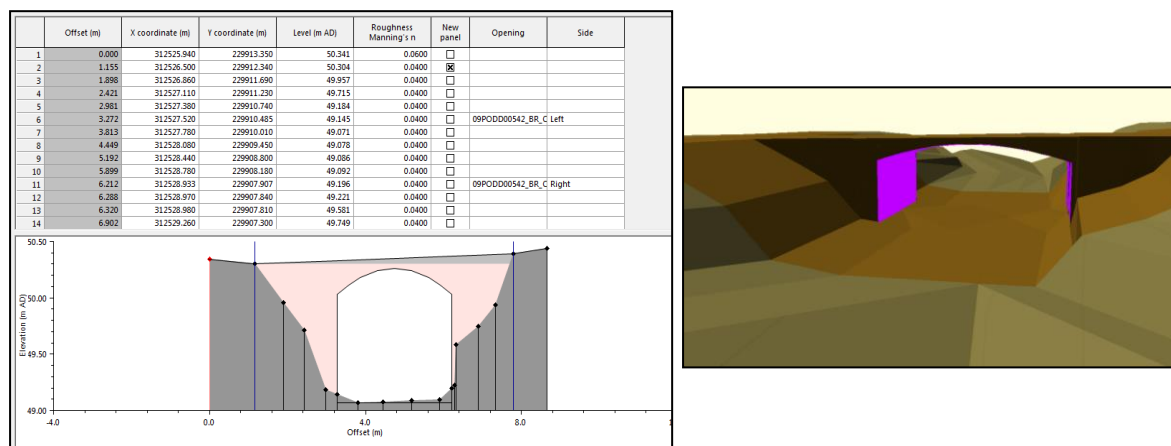


Figure 3.5: ICM Model Inputs – In Channel Structures

3.3.4 2D Domain Topography

The files used in the models to define the floodplain are based on the LiDAR and DTM data supplied for the Eastern CFRAM Study (refer to Section 2.3.2 and Chapter 4). A mesh was created from the provided LiDAR data to ensure the accurate assessment of 2D out of bank flow. For MIKE flexible mesh models, the resolution varies from typically 5m^2 in areas where greater detail is required e.g. roads to greater than 100m^2 in areas requiring less detail e.g. rural areas. For MIKE classic (or rectangular) grid models, the resolution has been set to 5m^2 (unless specified otherwise in Chapter 4) as this resolution provides sufficient detail to meet the requirements of this study, for the majority of floodplain features. Where there are features that play an important part of the flooding regime which cannot be represented with this resolution, then they have been explicitly modelled within the 2D domain. Further details are provided within Chapter 4, where this approach has been adopted.

For Infoworks ICM models, a flexible mesh is generated based on LiDAR data and allows varying detail to be used on the floodplain (compromise between detail and run times). There is an option available to apply terrain sensitive mesh generation. Infoworks ICM utilises GPU (Graphical Processing Units) parallel processing capabilities to reduce run times. The typical mesh size used is

from 1m² to 25m² in urban areas with the maximum size increasing to up to 1000m² in some large open areas.

Buildings can be represented in a number of ways:

- Voids (or blocked cells) – no flow
- Porous Polygons – percentage of free flow
- Mesh Zones with increased elevation – free flow over a specified elevation or height above the DTM
- Combination of above

Building footprints were defined by a GIS file extracted from national vector mapping and the relevant cells blocked out (MIKE models) or assigned zero porosity (Infoworks ICM models) to force water to flow around them. A paper on this topic prepared by Engineers Australia, Water Engineering in February 2012 informed the decision on adopting this approach. It is acknowledged that in reality buildings would provide an element of flood storage thus marginally reducing the overall flood extents but there is uncertainty as to the actual volume they would store. Therefore it was considered that preventing flood flows through buildings was a more conservative approach and would ensure flood extents are not underestimated. Details of the bathymetry files used and how they are applied in each relevant model are provided under Chapter 4.

In accordance with our tender methodology and the Dodder Pilot CFRAMS, it is considered that the above approach adequately represents the flow paths across the floodplain for each model. Predominantly, the LiDAR will be most accurate along the road network (away from vegetation cover) with the entire dataset assumed to meet the vertical accuracy as specified in the Stage 1 Project Brief - 0.2m RMSE. As buildings have been blocked out of the mesh, this will define the preferential flow paths through urban areas.

3.3.5 Roughness Coefficients

Roughness coefficients for cross-sections and structures within 1D river models are taken from the CIRIA (1997) Culvert design guide (see Table 3.2 and Table 3.3). Through site visits, photographs and videos included within the topographical survey information, an appropriate Manning's n value is selected for each cross-section and structure by the modeller. These initial Manning's n values may be amended (within normal bounds) to facilitate achieving model calibration.

Table 3.2: Manning's n Values for Normal Channels and Floodplains (CIRIA 1997)

Type of Channel and Description	Manning's n value		
	Minimum	Normal	Maximum
Natural Streams (top width at flood stage <30m)			
Clean, straight stream			
-full stage, no rifts or deep pools,	0.025	0.030	0.033
-as above, but more stones and weeds.	0.030	0.035	0.040
Clean, winding stream			
-some pools and shoals,	0.033	0.040	0.045
-as above, but some weeds and stones,	0.035	0.045	0.050
-as above, lower stages, more ineffective slopes sections,	0.040	0.048	0.055
-as above but more stones.	0.045	0.050	0.060
Sluggish reaches, weedy deep pools.	0.050	0.070	0.080
Very weedy reaches, deep pools, or floodways with heavy stands of timber and underbrush.	0.070	0.100	0.150
Mountainous streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high water levels			
-gravel bed with cobbles and few boulders,	0.030	0.040	0.050
-cobble bed with large boulders.	0.040	0.050	0.070
Floodplains (examples only)			
Pasture, no brush			
-short grass,	0.025	0.030	0.035
-high grass.	0.030	0.035	0.050
Cultivated areas			
-no crop,	0.020	0.030	0.040
-mature row crops,	0.025	0.035	0.045
-mature field crops.	0.030	0.040	0.050
Brush			
-scattered brush, heavy weeds,	0.035	0.050	0.070
-light brush and trees, in winter,	0.035	0.050	0.060
-light brush and trees, in summer,	0.040	0.060	0.080
-medium to dense brush, in winter,	0.045	0.070	0.110
-medium to dense brush, in summer,	0.070	0.100	0.160

Table 3.3: Manning's n Values for Culverts (CIRIA 1997)

Barrel, wall and joint description	Manning's n value		
	Minimum	Normal	Maximum
Concrete pipe			
-good joints, smooth walls	0.011	0.012	0.013
-good joints, rough walls	0.014	0.015	0.016
-poor joints, rough walls	0.016	0.0165	0.017
Concrete box			
-good joints, smooth walls	0.012	0.0135	0.015
-good joints, rough walls	0.014	0.015	0.016
-poor joints, rough walls	0.016	0.017	0.018
Metal pipe			
-68mm x 13mm corrugations	0.022	0.0245	0.027
-100mm x 20mm corrugations	0.022	0.0235	0.025
-127mm x 25mm corrugations	0.025	0.0255	0.026
-153mm x 50mm corrugations	0.033	0.034	0.035
-200mm x 55mm corrugations	0.033	0.035	0.037
-spiral rib metal pipe, good joints	0.012	0.0125	0.013
Concrete			
-trowel finish	0.011	0.0125	0.014
-float finish	0.013	0.0145	0.016
-unfinished	0.014	0.017	0.020
Brick			
-glazed, good condition	0.011	0.014	0.017
-cement, mortar, good condition	0.012	0.015	0.018
-poor condition	0.022	0.026	0.030

The selection of roughness values used for the 2D domains has been based on the 500m grid resolution CORINE land use dataset (Section 3.3.5). This is the best land use dataset currently available, covering Ireland at a consistent resolution meaning it is available for all 2D model extents within the CFRAM Study Area. This automates the approach successfully applied in the Dodder Pilot CFRAMS and Skibbereen FRAMS. The modeller may edit the roughness coefficients during model calibration where it is deemed necessary and can be justified. The CORINE dataset comprises of 44 different land use types - each of these were reviewed by Senior RPS Modellers and assigned an appropriate Manning's n and M value (Manning's 'M' is the inverse of the commonly used Manning's 'n' number and is the only roughness coefficient recognised by the MIKE21 software). The CORINE shapefile incorporating Manning's values was converted allowing it to be imported into the hydraulic modelling software. The values selected are shown in Table 3.4.

Table 3.4: CORINE Description and corresponding Manning's Values

CORINE - Description	Manning's Value	
	n	M
Continuous urban fabric	0.011	91
Discontinuous urban fabric	0.045	22
Industrial and commercial units	0.014	71
Road and rail network	0.013	77
Sea ports	0.014	71
Airports	0.013	77
Mineral extraction sites	0.03	33
Dump	0.05	20
Construction sites	0.04	25
Green urban areas	0.03	33
Sport and leisure facilities	0.03	33
Non-irrigated arable land	0.035	29
Permanently irrigated land	0.03	33
Fruit trees and berries plantations	0.07	14
Pastures	0.035	29
Annual crops associated with permanent crops	0.035	29
Complex cultivation patterns	0.04	25
Land principally occupied by agriculture with significant areas of natural vegetation	0.06	17
Agro-forestries	0.06	17
Broad-leaved forests	0.07	14
Coniferous forests	0.06	17
Mixed forests	0.065	15
Natural grassland	0.035	29
Moors and heathlands	0.045	22
Transitional woodland scrub	0.06	17
Beaches, dunes, sand	0.025	40
Bare rocks	0.02	50
Sparsely vegetated areas	0.025	40
Burnt areas	0.025	40
Inland marshes	0.025	40
Peat bogs	0.06	17
Salt marshes	0.03	33
Salines	0.03	33
Intertidal flats	0.02	50
Stream courses	0	0
Water bodies	0	0
Coastal lagoons	0	0
Estuaries	0	0
Sea and ocean	0	0

3.3.6 Other Parameters

The MIKE 21 models provide a facility for specifying the depth at which the model cells are identified as wet or dry. The drying depth is the minimum water depth allowed in a cell or element before it is taken out of the calculation. The flooding depth is the depth at which the cell or element will be entered into the calculation. This removes very shallow depths of water from the flood maps, leading to better representation of the flood extents. Unless otherwise stated, the drying depth is normally 0.02m and the flooding depth is normally 0.03m.

In MIKE 21 and Infoworks ICM, the value for eddy viscosity is normally defined as $0.02(x^2/T)$ where x represents the mesh resolution and T is the timestep interval. The eddy viscosity value can be amended beyond this calculated value (within normal bounds) in order to improve model stability.

RPS has made an assessment of dynamic structures to determine whether the operating controls will have an impact on the flood extents upstream and downstream of the control location. Where the structure will impact on flood extents, RPS has endeavoured to ascertain the operating controls of the dynamic structure. Details of these controls and the modelling assumptions made are specified in Chapter 4. RPS has assumed that all other dynamic structures are fully open.

The selection of the timestep varies for each model. For 1D models, the normal range is between 1 second and 5 seconds. Generally, the timestep selected for the 2D model is the same as the 1D model, unless otherwise specified under Chapter 4.

The first MIKE models constructed in HA07 used the current software version at that time - MIKE 2011, consequently RPS have constructed all MIKE FLOOD Rectangular mesh models throughout HA07 using this software version to maintain consistency. Version 6.5 of the ICM software has been used for ICM modelling in HA07. A summary of this information is provided in Table 3.1.

3.3.7 Integration of Fluvial Hydrological Analysis with Hydraulic Modelling

The hydrological analysis for HA07 was completed prior to the hydraulic analysis phase of the report and had the overall objective of providing hydrological input files (boundary conditions) in terms of design flows and hydrographs for each hydraulic model, and also flood event calibration data (as derived from hydrometric data recorded for past flood events). The hydrology report documented the methodology, process and outputs and also identified areas where further detail and analysis would be undertaken at the hydraulic analysis stage of the CFRAM Study. The core aspect of this is integration of hydrology analysis and hydraulic modelling to achieve final design flows. There are also specific aspects of the hydrology which require further review as part of the hydraulic modelling and these are addressed later in this section.

The hydraulic analysis for each AEP/Model is very much integrated with the fluvial hydrological analysis as outlined in the Hydrology Report and in Figure 3.6. The hydrological analysis produced

boundary input and intermediate check files for each hydraulic model. In most cases, these files consisted of design hydrographs for each AEP as defined at every Hydrological Estimation Point (HEP) in the model. Lateral inflow hydrographs were also provided between HEPs to ensure any interim contributing catchment areas were not missed, and to provide a form of flow balancing moving downstream. These hydrographs were simulated in the hydraulic model as the first step in the integration of hydrological and hydraulic analysis.

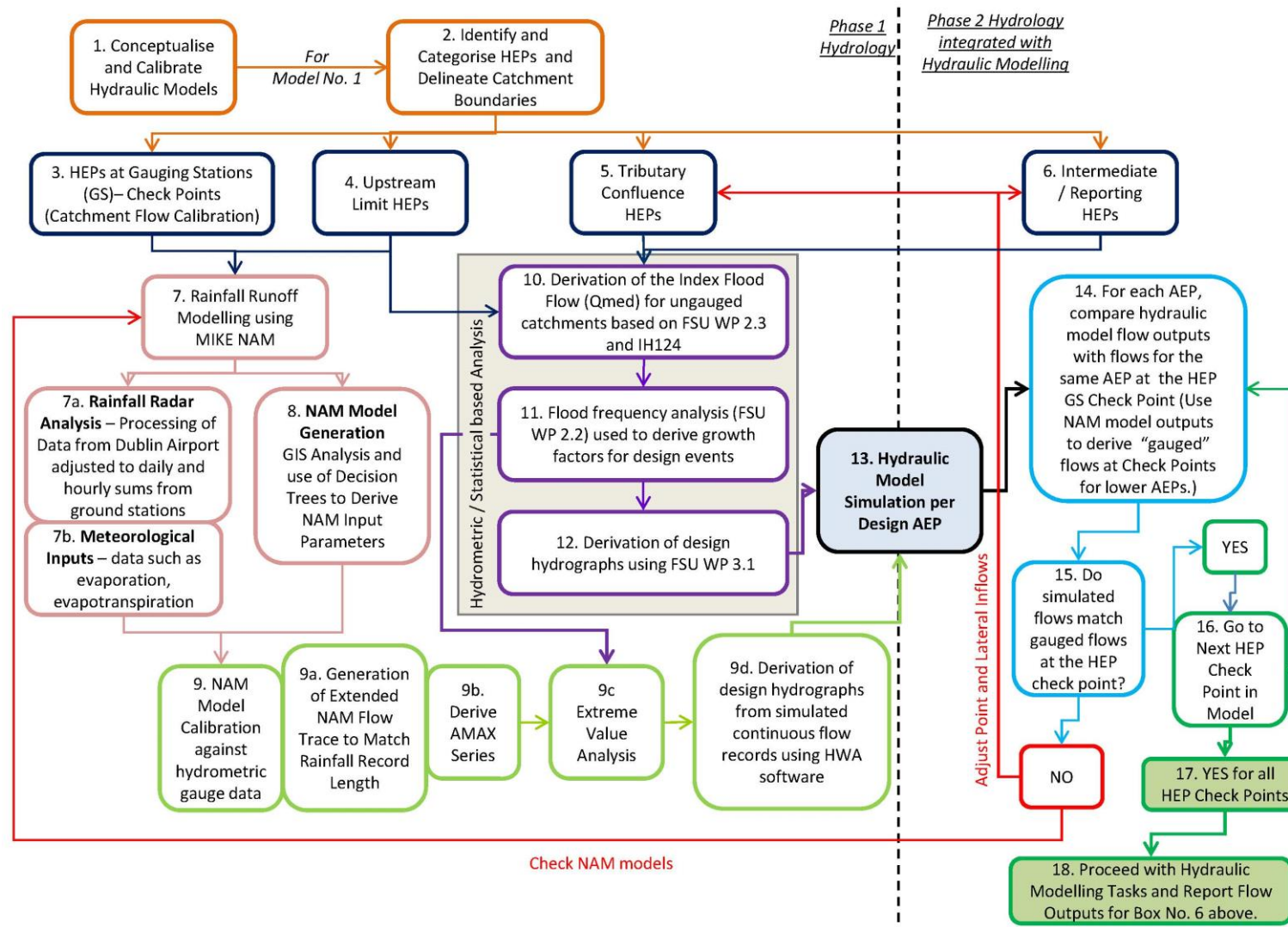


Figure 3.6: Fluvial Hydrology Process Flow Chart (refer to HA07 Hydrology Report)

Building on Phase 2 as shown in Figure 3.7, hydrological analysis was revisited using the following hierarchical approach:

1. Fluvial Joint Probability (refer to Hydrology Report Section 6.3.1) - the initial assumption of the same frequency conditions in both watercourses at confluence points is tested against the guidance in FSU WP 3.4 "Guidance for River Basin Modelling" whereby the AEP in the tributary watercourses is reduced based on:
 - gauged data where available on both watercourses or;
 - based on the AREA, FARL and the distance between the centroids of both catchments (see Table 13-1, FSU WP 3.4).
2. Lateral inflows may also be subject to minor adjustment. These flows have been scaled based on the total catchment flow to that point and as such some adjustment may be appropriate.
3. Where the sum of the flows does not achieve the peak flow for the required AEP at the check point then the modeller may refer the model back for hydrology design flow estimation review and / or hydrological re-analysis. Where this is the case the catchment descriptors will initially be checked and further checks on the appropriateness of the adjustment factor and growth factor / pooling group may also be considered.
4. Alternative hydraulic modelling techniques may be considered for urban catchments requiring rainfall based hydrological data input rather than flow based inputs derived from statistical analysis.

The details and justification for this approach are supplied in the Hydrology Report and is referred to here as an example of the integrated approach that has been taken between hydrology and hydraulics. The approach ensures that modelled flows are 'anchored' to the design flows at each HEP throughout the entire catchment. HEPs have been located at intermediate points along the channel and at the interface between models such that the total flow in one model is tied to the inflow in the next model downstream such that both are tied to the hydrological estimate. Where there is a large discrepancy between the total flow at the downstream boundary of a model and the hydrological estimate, this is investigated further to ascertain if the modelled flow or the hydrological estimate is truly reflective of the catchment flow conditions. Where it is deemed that the model is capturing something that the hydrological estimates are incapable of, such as hydraulic attenuation due to a structure, then the modelled flows are used as the upstream boundary for the model downstream. Alternatively, it may be the case that the modelled flows are not truly reflecting a catchment feature, such as the attenuating effect of a lake represented within a 1D only model. In this instance the hydrological estimate is retained as the upstream boundary to the model downstream. Consequently, this approach ensures that the flood maps are representative of the stated annual exceedance probability.

All cases in which application of the aforementioned hierarchal approach were undertaken as part of the hydraulic analysis phase are detailed under Chapter 4 as appropriate.

3.4 COASTAL MODELLING

3.4.1 Overview

In order to facilitate the computational modelling for those AFAs located within close proximity to the coast, a similar approach was taken as for the inland, fluvially-dominated areas. However, some major differences included the addition of coastal boundaries and coastal bathymetry, the use of flexible mesh where necessary and the consideration of joint probability between fluvial and tidal components.

Each coastal area was reviewed in order to ascertain if the tidal component was influential to the cause of flooding in the area. Where this was the case, a decision was made whether to utilise flexible or rectangular mesh, depending on the topography of the area and the extents and position of those areas likely to flood. In order to make this judgement, a thorough review of available LiDAR information was undertaken. Taking into account the worst possible coastal water level to be considered within this Study, the 0.1% AEP HEFS, those coastal areas with elevations below the corresponding water level, with a direct flood path from the sea, would most likely be coastally inundated. Areas where coastal inundation is an issue were modelled using an ICM flexible mesh approach, which allowed more extensive areas to be covered by varying the mesh size across the domain as appropriate.

A fully functioning tidal model was developed for each relevant AFA. It was important to ensure a representative tidal model was achieved, with water moving freely and realistically throughout the model domain. The floodplain and buildings were also included in the model.

A bed roughness map was produced for all models, using the CORINE dataset. Coastal bed resistance values were taken as a Manning's M value of $30m^{1/3}/s$. Flood defence assets, where they have been identified (see Table 2.1), were included in the ICM mesh. These have been incorporated into the mesh as base linear structures.

3.4.2 Coastal Modelling Software

The computational modelling was undertaken using ICM. To adequately represent the variable bathymetry and topography, the model mesh for each flexible mesh AFA was generated and refined in regions of most importance to achieve satisfactory model performance. The flexible mesh technology allowed the size of the computational cells to vary across the domain of each model, allowing smaller cells of circa 5 metres to be positioned in areas of rapidly changing bathymetry, such as offshore banks and channels, along with detailed areas of topography. Smaller cells were vital in depicting flood paths between buildings. Larger cells in the order of 100 to 200 metres were used in areas of more consistent bathymetry, such as agricultural land, mud flats and the open sea.

3.4.3 Coastal Model Boundaries

The influence of coastal water levels has been modelled by applying an appropriate water level boundary profile to the downstream extent of the relevant fluvial model. The effects of the sea levels are propagated upstream by the modelling software allowing the interaction of river flows and coastal water levels to be modelled accurately. This ensures that both coastal and fluvial flooding mechanisms are investigated.

Coastal model boundaries were established on an individual basis for each model and are detailed under Chapter 4. In general, the boundaries were located in areas of similar topography and suitable water depth, at an appropriate distance offshore. The boundaries were representative of extreme total water levels derived under the ICPSS, with a range of suitable AEPs available. The ICPSS water levels are total water levels, comprising tidal and surge components which together yield a joint probability event of a particular AEP. These vary around the coastline and specific values for each AFA are detailed in Chapter 4.

Temporally varying water levels have been used to represent the coastal boundaries in all relevant AFAs throughout this Study. The inclusion of a temporal element within any detailed assessment of tidal flood risk is a very important consideration due to the relatively rapid variation in even extreme tidal events associated with the normal astronomical tidal cycle. In general, this limits the duration of defence exposure and overtopping and consequently is an important consideration in establishing the volume of water that can enter vulnerable areas. RPS' experience with detailed modelling of coastal flooding has indicated that it is seldom sufficient to simply model a single tidal cycle, as extreme tidal surges often persist over multiple tidal cycles. Consequently the most onerous tidal flooding is normally a result of the accumulation of flood waters entering the area over multiple tidal cycles.

Using information from the Admiralty Tide Tables, RPS established a tidal water level half-way between the Mean High Tide and Mean High Water Springs (MHWS). This was considered appropriate as a significant tidal event, as MHWS was considered too extreme when assessed in conjunction with extreme surge events. As the total water level for each AEP event was specified from the ICPSS, the magnitude of the required surge component was calculated for each AEP as the difference between the specified total water level and the established tidal water level.

Where appropriate, tidal boundary profiles were extracted from the RPS Irish Surge and Tidal Model (ISTM) in order to represent a realistic tidal regime of the area. Using information from the ISTM, as well as observed extreme events where available, RPS have established that a typical profile of a surge event could be adequately represented in this Study by a positive sine curve of 48 hours duration. Each sine curve was scaled appropriately to achieve a surge residual of the relevant magnitude.

The relevant tidal curve was combined with the appropriate residual surge profile to obtain the total combined water level time series as required for the relevant AEPs. It was assumed that the peak of the surge would coincide with the peak of the tide at the boundary locations.

Each time series includes a number of tidal cycles, with one preceding the onset of the surge event to assist in developing stable conditions within the models, prior to modelling the onset and progression of inundation during the surge event.

3.4.4 Coastal Simulations, Joint Probability and Sensitivity

Upon development of a completed and successfully calibrated model, relevant simulations were undertaken in order to determine the worst case scenario flooding for each AEP.

As a starting point, RPS reviewed both coastal dominated and fluvial dominated scenarios for each AFA, combining low probability events from one source, with a more frequently occurring 50% AEP event from the other. It was assumed that in order for such an event to be extreme, the likelihood of at least some activity from the other source was high, before joint probability was considered further.

As such, coastal events of 10%, 0.5% and 0.1% AEP were combined with a fluvial event of 50% AEP in order to produce joint return periods of 10%, 0.5% and 0.1% AEP for a coastal dominated scenario.

Conversely, fluvial events of 10%, 1% and 0.1% AEP were combined with a coastal event of 50% AEP for joint return periods of 10%, 1% and 0.1% AEP for a fluvial dominated scenario.

Where there were significant areas of overlap between these outputs, and where other historical information, Preliminary Flood Risk Assessment (PFRA) data and ICPSS flood extents indicated a relationship, the requirement for joint probability analysis was considered during a screening analysis, as outlined in the Hydrology Report. However, due to the lack of available historical gauge information, the determination of dependence relied on the application of intuition and experience of the RPS Coastal Modelling team. A better method, and one that should be employed where suitable data exists, is to examine several years of simultaneous tidal level and river flow data as this allows the correlation analysis to be performed in a scientific and objective manner. In this case the correlation can be determined through the use of graphical techniques such as scatter diagram analysis. Further details are provided within the South Eastern CFRAM Study NTCG GN20 Joint Probability Guidance.

Where necessary, further simulations were set up to determine flood extents for medium/medium events, where flooding was not dominated solely by fluvial or coastal events, but was a combination of less extreme events from both sources for a given joint AEP. Further details are provided in Chapter 4 for the AFAs where this analysis was undertaken.

Sensitivity tests were undertaken for the principal parameters used within the model to identify the degree of variability within the model output associated with the model inputs. This included variation in the joint probability and temporal variations, along with parameters such as eddy viscosity and bed resistance. In some AFAs, relative timing between fluvial and coastal peaks was critical in the determination of flood extents, and in general it was assumed the events from both sources would peak together at the location affected most by both fluvial and coastal flooding. As such, timings were adjusted and using an iterative approach, the worst case flood outlines, for a particular combination of events, were established.

3.5 HYDRAULIC MODEL CALIBRATION

3.5.1 Overview

The use of flood event data draws on the historic data analysis undertaken at the Inception Stage of the CFRAM Study (refer to Inception Report) whereby key flood events were identified for use in the calibration of each model. The following aspects contributing to model calibration were also discussed in the hydrology report, with further details provided below.

Specific details on the use of past flood event data for model calibration is provided in Chapter 4 per AFA/Model. Generally, the principal model parameters that are reviewed and amended during the model calibration process are identified below:

- Bed and floodplain roughness coefficients;
- Structure roughness and head loss coefficients;
- Timing of hydrographs;
- Magnitude of hydrographs;
- Incorporation of additional survey information (e.g. additional cross-sections or missed structures).

The choice of parameter that should be adjusted in order to calibrate the model to better represent reported historical flooding will depend on the desired output i.e. whether there is too much or too little flooding in a particular area of the model. The chosen parameter may require adjustment locally at a particular structure or reach of watercourse or globally affecting the entire model. The decision is based on the experience of the modeller and can be an iterative process until selection of the right combination of parameters (within acceptable bounds) generates a flood extent which best represents the flooding mechanisms in the AFA. This process is undertaken taking due cognisance of the limitations of the hydraulic model – see Section 3.10. Details of the parameters adjusted during calibration of each model are provided in Chapter 4.

3.5.2 Rating Review of Hydrometric Stations

In HA07 there were seven stations specified for rating review through hydraulic modelling as shown in Table 3.5. The full methodology and results and impacts of the rating review analysis are included in the Hydrology Report. From a hydraulic modelling perspective the outcomes of the rating reviews were identified in the Hydrology Report as having a potentially high impact on the associated hydraulic model calibration since this depends on the upper limits of a gauge rating i.e. observed historical flood event flow data. This could be changed based on the results of rating reviews i.e. if significant uncertainty is identified in the current rating and it is deemed appropriate to revise it using the CFRAM Study hydraulic analysis rating curve. Table 3.5 identifies two stations for which significant uncertainty

with the current rating was identified by the rating review. Following this assessment, the hydrological analysis was reviewed and updated where required – further details are found within Chapter 4 of the HA07 Hydrology Report.

Table 3.5: Hydrometric Station Rating Reviews

Station Number	Station Name	Final Station Rating Quality Classification	AFA/HPW Model	Significant Uncertainty Identified in current rating
07003	CASTLERICKARD	Pre 1970: A1 Post 11/07/1975: B	Johnstown Bridge	No
07005	TRIM	A1	Trim	No
07006	FYANSTOWN	Post 04/11/1986: A2 Pre 21/08/1982: B	Navan	No
07009	NAVAN WEIR	A1	Navan	No
07010	LISCARTAN	Pre 02/72: A1 02/72 to 20/05/82: A2 20/05/1982 to date: A2	Navan	No
07023	ATHBOY	NOT REVIEWED UNDER FSU	Athboy	Yes
07044	BALLIVOR	NOT REVIEWED UNDER FSU	Ballivor	Yes

3.5.3 Use of NAM modelling flow outputs

Full details of the use of hydrological rainfall run-off (NAM modelling) are provided in the Inception and Hydrology Reports. The overall objective was to provide an additional layer of simulated flow data at gauging stations where an augmented AMAX series was of potential benefit to the core statistical based hydrology analysis in determining design flows for each model (refer to Figure 3.6). Another potential benefit of the rainfall runoff models was identified in that a further layer of simulated hydrometric data would be available for calibration of the hydraulic models. Events which may be outside the continuous flow record period of the gauge would now be available through the simulated time series flow data at hydrometric stations where NAM modelling was undertaken. No continuous level information is available as the models are spatially dimensionless (i.e. they are not hydraulic models with inputted topographical survey information). This potential benefit was utilised in the hydraulic modelling calibration as summarised in Table 3.6 and detailed under Chapter 4.

Table 3.6: Use of Simulated Flow Trace (NAM outputs) at Hydrometric Stations for Hydraulic Model Calibration

Hydrometric Station	Model	Simulated Flow Trace used for flood event calibration?
07001	Athboy Baltray	Yes
07003	Castlerickard & Johnstown bridge	Yes
07005	Trim	Yes
07007	Castlerickard & Johnstown bridge	Yes
07009	Navan	Yes
07010	Navan	Yes
07012	Drogheda and Baltray	Yes
07023	Athboy	No
07044	Ballivor	Yes

3.5.4 Consultation Activities

Consultation activities which occurred from early to late 2015 on the draft flood maps included:

- Consulting with the relevant Local Authority representatives during the development of the draft flood mapping;
- Holding a series of Public Consultation Days, including a dedicated Elected Member briefing session, to outline the flood mapping process and to elicit feedback on the draft flood maps;
- Holding a workshop with the members of the Eastern CFRAM Study Stakeholder Group to outline the flood mapping process and to elicit feedback on the draft flood maps;
- Uploading the draft flood maps to the project website and inviting feedback on the draft flood maps.

Further details on the above consultation activities are contained within the Draft Flood Mapping Phase Summary Report (IBE0600Rp0033_Mapping Phase Summary Report_D02).

A formal consultation on the draft flood maps was launched by Mr. Simon Harris T.D., Minister of State at the Department of Public Expenditure and Reform with special responsibility for the Office of Public Works, under SI 122 of 2010. This consultation occurred between 20th November 2015 and 23rd December 2015. The draft flood mapping was available for viewing within an online mapping tool and was also put on display at Local Authority offices. The SI consultation provided a mechanism for Technical Objections under SI 122 of 2010.

All of the submissions, observations, comments and technical objections received in relation to the consultations activities described above were taken on board during the finalisation of the flood mapping. Further details on where the submissions received resulted in amendments to the hydraulic analysis are available in Chapter 4.

3.6 HYDRAULIC MODEL SENSITIVITY AND PERFORMANCE

3.6.1 Sensitivity Analysis

Sensitivity tests have been conducted for each model. The parameters selected were dependent on the specific model but generally included those listed below. The model output for each sensitivity model simulation was compared with the verified model, with further details and a discussion on the sensitivity of the selected parameter given in the relevant section of Chapter 4:

- Roughness coefficients: Completed for all models. This involved adjusting the roughness coefficients within the 1D and 2D model domains to the upper and lower bounds as defined in the OPW Guidance Note 22. The minimum and maximum values used are found in Table 3.2 and Table 3.7 for the 1D models and 2D models respectively.

Table 3.7: CORINE Description and corresponding Manning's Sensitivity Values

CORINE - Description	Manning's n Value		
	Minimum	Normal	Maximum
Continuous urban fabric	0.008	0.011	0.012
Discontinuous urban fabric	0.035	0.045	0.05
Industrial and commercial units	0.012	0.014	0.016
Road and rail network	0.011	0.013	0.015
Sea ports	0.012	0.014	0.016
Airports	0.011	0.013	0.015
Mineral extraction sites	0.028	0.03	0.035
Dump	0.04	0.05	0.07
Construction sites	0.03	0.04	0.05
Green urban areas	0.028	0.03	0.035
Sport and leisure facilities	0.028	0.03	0.035
Non-irrigated arable land	0.022	0.035	0.04
Permanently irrigated land	0.028	0.03	0.035
Fruit trees and berries plantations	0.05	0.07	0.13
Pastures	0.022	0.035	0.04
Annual crops associated with permanent crops	0.022	0.035	0.04
Complex cultivation patterns	0.03	0.04	0.05
Land principally occupied by agriculture with significant areas of natural vegetation	0.045	0.06	0.08
Agro-forestries	0.045	0.06	0.08
Broad-leaved forests	0.05	0.07	0.13
Coniferous forests	0.045	0.06	0.08

CORINE - Description	Manning's n Value		
Mixed forests	0.047	0.065	0.11
Natural grassland	0.022	0.035	0.04
Moors and heathlands	0.035	0.045	0.05
Transitional woodland scrub	0.045	0.06	0.08
Beaches, dunes, sand	0.023	0.025	0.032
Bare rocks	0.018	0.02	0.023
Sparsely vegetated areas	0.023	0.025	0.032
Burnt areas	0.023	0.025	0.032
Inland marshes	0.023	0.025	0.032
Peat bogs	0.045	0.06	0.08
Salt marshes	0.028	0.03	0.035
Salines	0.028	0.03	0.035
Intertidal flats	0.018	0.02	0.023
Stream courses	N/A	0	N/A
Water bodies	N/A	0	N/A
Coastal lagoons	N/A	0	N/A
Estuaries	N/A	0	N/A
Sea and ocean	N/A	0	N/A

- Critical structure coefficients: Completed for models containing a critical structure which is likely to have a significant impact on local receptors. The factor determining the energy loss occurring for flow through the structure was increased. For MIKE models, this parameter is dimensionless, with a recommended maximum value of 0.9 (as per discussions with DHI Software Managers). As discussed in Section 3.3.3, long culverts have been modelled as a series of closed cross-sections meaning that these structures do not have an explicit head loss parameter. Where a critical structure has been modelled as a long culvert, the roughness coefficient was increased to an upper bound value (based on Table 3.2) during the sensitivity analysis, in order to replicate the increase in head loss effect.
- Flow inputs: Completed for all models. The outputs from the assessment of the sensitivity and uncertainty in the hydrological analysis (see Hydrology Report, Chapter 8) have been converted into a score for each model. This score has been used to derive factorial adjustments to the peak flow estimates as per the range of adjustments set out in Guidance Note 22.
- Operation of dynamic structures: Completed for models where the operation of a dynamic structure could potentially have a significant impact on local receptors. The operation of the structure was assumed to be the opposite of the operation assumed in the design simulations for this sensitivity simulation.
- Downstream boundary conditions: Completed for all models where the downstream boundary could potentially impact on simulated water levels within the AFA. The water level generated

from the 1% MRFS AEP event replaced the current scenario water level boundary in this sensitivity simulation.

- **Representation of buildings in 2D model domain:** Completed for models where the representation of buildings could potentially have a significant impact on local receptors. The building representation on those AFAs with a low level of fluvial flood risk is considered to have a low influence on modelled flood extents and depths – the screening assessment has concluded that these AFAs do not require the building representation sensitivity test to be undertaken. The sensitivity test has been undertaken where the screening assessment identified AFAs with a significant level of fluvial flood risk or where flow paths are potentially being blocked by the presence of buildings. This assessment has been supported by information acquired through the data collection process and public/stakeholder consultation. The 2D model topography is based on LiDAR information only (with buildings ‘unblocked’), and the roughness of the building footprint increased (Manning’s n of 0.3) in this sensitivity simulation.
- **Timing of tributaries:** Completed for models where the total discharge of the tributary and the main river is greater than the discharge simulated under the ‘Flow Input’ simulations described above. The timing of the hydrograph of the tributary was moved by 10% of the overall duration in this sensitivity simulation.
- **Flow Volume:** Completed for models where it was concluded that there was high uncertainty associated with the hydrological analysis (see Hydrology Report, Chapter 8). The flow volume was increased by a factor of 2 for this sensitivity simulation.

Note that where the sensitivity to a particular parameter is not discussed within Chapter 4, this is due to a screening assessment determining that a sensitivity analysis was not required. The screening process followed the methodologies outlined above for each parameter using GIS tools and other sources of information e.g. the Hydrology Report.

3.6.2 Model Performance

Details of the performance of each model, including a review of any significant instabilities, are provided within each ‘Summary of Calibration’ section in Chapter 4. A mass balance check has also been carried out on the 1% AEP model to ensure that the total volume of water entering and leaving the model at the upstream and downstream boundaries balances the quantity of water remaining in the model domain at the end of a simulation. As a general rule of thumb, mass errors should be less than 2%. If the mass error is greater than 2%, the cause and location of the mass error within the model schematisation should be identified and the consequence of this error assessed and improvements to the model considered. If the mass error is greater than 5%, then it suggests that the model schematisation is not robust and needs to be reviewed (Environment Agency, 2010). For MIKE 2011 models, this is a manual calculation completed using Microsoft Excel. For MIKE 2012 and Infoworks ICM models, the software can generate the mass balance automatically.

3.7 FUTURE SCENARIOS

The OPW has produced a draft guidance note “Assessment of Potential Future Scenarios for Flood Risk Management” (The OPW, 2009). The document gives guidance on the allowances for future scenarios based on climate change (including allowing for the isostatic movement of the earth’s crust), urbanisation and afforestation. Table 1 from the guidance has been adapted for the purposes of this Study to take into account catchment specific effects which were used in the hydrology analysis as the basis for the design flow adjustment for the mid range (MRFS) and high end (HEFS) future scenarios (refer to Hydrology Report, Chapter 8).

The future potential changes which may affect the outputs of the CFRAM Study were identified and described in the Hydrology Report under the following headings:

- Climate Change
- Afforestation
- Land Use and Urbanisation
- Arterial Drainage
- Geo-morphology

The allowances applied to design flows and coastal boundary conditions for climate change (extreme rainfall depths, flood flows and mean sea level rise); urbanisation; and afforestation are shown in Table 3.8 and detailed in the Hydrology Report.

Table 3.8: HA07 Allowances for Future Scenarios (100 year time horizon)

	MRFS	HEFS
Extreme Rainfall Depths	+ 20%	+ 30%
Flood Flows	+ 20%	+ 30%
Mean Sea Level Rise	+ 500mm	+ 1000mm
Urbanisation	UAF ³ of 1.14 Urban W.C. UAF ⁴ of 2.5	UAF ³ of 1.28 Urban W.C. UAF ⁴ of 2.5
Afforestation	- 1/6 Tp ¹	- 1/3 Tp ¹ + 10% SPR ²

Note 1: Reduce the time to peak (Tp) by one sixth / one third: This allows for potential accelerated run-off that may arise as a result of drainage of afforested land

Note 2: Add 10% to the Standard Percentage Run-off (SPR) rate: This allows for increased run-off rates that may arise following felling of forestry

Note 3: UAF (Urban Adjustment Factor) applied to 'greenfield' flow estimates.

Note 4: UAF (Urban Adjustment Factor) for small urban tributaries within AFA extents assume 85% urbanisation. Assessed on a case by case basis.

The climate change allowances are applied to all models. Urbanisation allowances are applied on a case by case basis as required, the factors themselves having been derived during the hydrology analysis by looking at historic urbanisation growth indicators and estimating appropriate growth factors for MRFS and HEFS. The outputs of future scenarios modelling for each AFA/HPW are used to assess the sensitivity of the AFA to future change within Chapter 8 of the UoM07 Preliminary Options Report (IBE0600Rp0037).

Arterial Drainage was identified as a potential future scenario that required further consideration in HA07. The analysis of gauge stations in HA07 which have data from both pre- and post- arterial drainage scheme implementation shows that the Boyne Catchment Drainage Scheme has on average increased the Q_{med} by 50%. This is in line with previous research carried out on Irish catchments which suggested that arterial drainage schemes can lead to significant changes in peak discharge of up to 60% (Bailey and Bree 1981).

The hydrological analysis and design flow estimation are largely based on post arterial drainage scheme gauge data and as such can be considered to represent the average of the post arterial drainage scheme average. As the post arterial drainage scheme increases peak flows, this can be considered to be a robust approach. As such it is not considered necessary to make any additional flow allowances for the future scenarios in relation to the arterial drainage scheme.

Geo-morphological changes ultimately apply to the performance of flood risk management options and as such, this will be considered further and reported on in the next stage of the CFRAM Study.

3.8 DEFENCE FAILURE SCENARIOS

For each effective flood defence asset, an assessment was carried out to identify locations where there might be a vulnerability to breach. The criteria used to locate breach vulnerable areas was to identify locations where the retained depth of water above ground level exceeds one metre up to the design event (1% AEP for fluvial and 0.5% AEP for coastal). Where multiple locations were identified in an AFA, two locations were selected and therefore two scenarios were simulated. The selection of these two areas was based on the condition of the defences (which parts of the defences would be most likely to fail), whether any defences have failed in the past, the topography behind the defence (would the flood water inundate a large area behind the defence) and what receptors would be at risk if the breach were to occur.

In HA07, there were no defences identified which met the criteria. Consequently, there was no analysis undertaken to identify and assess the flood hazard that may be caused, or significantly increased, by the occurrence of a failure of any defences.

3.9 APPROACH TO FLOOD MAPPING

Along the 1D only model reaches (MPWs), the modelling software creates flood extents by interpolation of water levels at the defined cross-sections, and within the extent of the cross-sections only. For some models, this can result in an irregular, unnatural flood extent. Although the level of detail for MPWs is not as high as that achieved along HPWs, the appearance of the flood extent for these reaches can be improved by incorporating a digital elevation model e.g. the National Digital Height Model (NDHM) into the model. During generation of the model outputs, the calculated water level at each cross-section is projected onto the NDHM, following the relevant contour and creating a smoother flood extent boundary. This approach has been taken for all models except where otherwise stated in Chapter 4. Due to the lesser accuracy of the NDHM, it has only been used along MPWs where LiDAR information is not available. Within the 2D model domains, only LiDAR information has been used.

Flood mapping utilises ArcGIS to present the results of the hydrodynamic models on background mapping and to derive a series of flood hazard maps in support of the CFRAM Study. ArcGIS version 10.0 is utilised for the production of all AFA mapping. Before commencing the mapping, the raw outputs of the hydraulic models are checked and cleaned to remove outliers and islands which are not connected to the fluvial or coastal flooding mechanisms.

The approach for the generation of flood maps from the output files of MIKE FLOOD Classic Grid (rectangular mesh) models involves the use of the Statistics tool from the MIKE Zero toolbox. The maximum parameter (e.g. depth) is extracted from the dfs2 results file generated by populating the 'Maps' tab within the HD Parameters file in MIKE11. This file covers both the 1D and 2D model domains. The maximum dfs2 output file is opened in ArcGIS (using a dfs2 Plug-in) and converted to a grid raster format which is reclassified as a singularity and subsequently converted to a shapefile showing the flood extent.

For MIKE FLOOD Flexible Mesh models, the above process is repeated but the 'Maps' results file covers the 1D domain model domain only. A separate process using Data Extraction FM (within MIKE Zero) is required to extract the maximum parameter from the flexible mesh results output (dfs2 file). The Mike2Shp tool in the MIKE Zero toolbox is used to convert this file to a shape file, which gives the maximum level within each element of the mesh for that model simulation. It is edited in ArcGIS to remove values below 0.02m to provide the best representation of the flood extent. A raster file is created based on the maximum levels to generate a depth map of the floodplain. Both results files described above are then combined to generate the flood map covering both the 1D and 2D model domains.

The tidal influence boundary on the flood extent maps has been derived by comparing modelled water levels on tidally influenced rivers for the 0.1% AEP and 10% AEP tidal design runs. The most downstream point at which there is no difference in peak water level between these two design runs is the point beyond which tidally influenced flooding does not extend upstream, as the water level beyond this point is entirely governed by fluvial processes.

Before finalising each flood map, any necessary post-processing of the flood extents is completed. This includes removing bridges which aren't overtopped during the flood event from the flood extent. This is required as the output from the MIKE software does not make a distinction between the in-channel structures which overtop and those that do not (assuming all in-channel structures are flooded).

The approach outlined above is used to generate flood extent, zone, depth and velocity maps as all of the required information is contained within the model output files. The flood extent map also includes peak level and flow information, extracted from the model, and a benefitting area (as defined in the OPW Guidance Note 33) where defences are present. MIKE ECOLab is used to generate the risk to life maps, based on the maximum combination of velocity and depth reached within the model results file.

The map is set at the appropriate scale (1:5,000 or 1:25,000 for HPW and MPW respectively), additional information added (such as the river centre line) and set within the completed title block. A pdf of the map is created to ensure the map is in print-ready format.

The approach for the generation of flood maps from the output files of ICM models entails exporting the model results within the 2D mesh to a shapefile. This shapefile gives the maximum level within each element of the mesh for that model simulation. It is edited to remove values below 0.02m to provide the best representation of the flood extent. A raster file is created based on the maximum levels to generate a depth map of the floodplain (within the 2D model domain).

For the 1D (river channel) element of ICM models, all of the cross-sections are exported from the model to a shapefile. Each cross-section contains the maximum level reached during the model simulation - these levels are interpolated to generate an elevation surface shapefile for the 1D channel. The shapefile is then converted to a surface raster. The DTM information is subtracted from the elevation surface files to generate the flood depth information. The 1D and 2D raster files are combined to generate a complete extent of the floodplain from which a shapefile is produced. The shapefile is overlaid on background mapping, to produce flood extents maps. The same approach is used to generate flood velocity and risk to life maps as all of the required information is contained within the model output files.

3.10 ASSUMPTIONS, LIMITATIONS AND UNCERTAINTY

There are inherent assumptions, limitations and uncertainty associated with hydraulic modelling which are beyond the scope of this report. The assumptions, limitations and uncertainty which are specific to each individual model are discussed in detail under Chapter 4. Each issue is discussed, with the requirement for the assumption justified. The issues addressed will include:

- schematisation decisions regarding out-of-bank flow routes;
- culvert/bridge schematisation (including skew angle considerations);

- sweetening flow assumptions;
- comments and notes throughout to reflect data sources; changes to parameters from default;
- explanation of parameters used that are outside of the expected ranges; and
- any other atypical assumptions made.

3.11 DELIVERABLES

As an accompaniment to this report, the following deliverables are provided:

- All survey digital data files (including AutoCAD files / PDFs of cross-sections, long-sections and plan views, ascii files containing cross-section data, photographs at each cross-section and videos at structures);
- Digital hydraulic model files;
- Defence Asset Database;
- Digital copies of the GIS-format and Print-ready format Flood Hazard Maps.

4 MODEL SPECIFIC DETAILS

The following sections provide the specific details of each model within UoM07:

4.1 ATHBOY

4.2 BALLIVOR

4.3 DROGHEDA & BALTRAY

4.4 EDENDERRY

4.5 JOHNSTOWN BRIDGE

4.6 LONGWOOD

4.7 MORNINGTON

4.8 NAVAN

4.9 TRIM

4 HYDRAULIC MODEL DETAILS

4.1 ATHBOY MODEL

Name	Local Authority	Unique ID	SSA	Status	Date
Athboy	Meath	70028	AFA	Final	08/05/2017

4.1.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600 Rp0001_Flood Risk Review) highlighted Athboy as an Area for Further Assessment (AFA) for fluvial flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

Athboy is located on the Athboy River (also known as the Tremblestown River), a tributary of the River Boyne. The Athboy AFA is affected by the Athboy River and a small unnamed tributary which flows from the north west into the Athboy River before reaching Athboy town (referred to as the Athboy River Trib). Downstream of the AFA a number of small tributaries join the Athboy River but the largest portion of the catchment contributes to the model upstream of Athboy. Both the Athboy River and the Athboy River Trib are included in the Arterial Drainage Scheme (ADS) and may be subject to periodic dredging.

The Athboy model (Model 3 within UoM 7) is the only model located on the Athboy River. The model ends at the confluence with the River Boyne. This location on the River Boyne is included in the Trim model.

The extents of the Athboy model encompass identified watercourses to be modelled. Reaches of the watercourse with the potential to affect Athboy directly were assigned a high priority; this encapsulated the Athboy Trib and the reach of the Athboy River starting approximately 0.5km upstream of the AFA boundary and 0.5km downstream. The remainder of the Athboy River was given a medium priority, the downstream limit of which is located at the confluence with the River Boyne.

The HPWs, ie the uppermost 3km of the modelled Athboy River and the Athboy River Trib were modelled as 1D-2D using the MIKE suite of software and LiDAR was used along these extents to model the 2D out of bank flow. The remaining reach of the Athboy River, MPW, was modelled as 1D using the MIKE suite of software. Extended cross sections and the national DTM were used to map the out of bank floodplain flood extents.

The Athboy / Tremblestown River can be considered to be well gauged with one gauging station located in the centre of the town called Athboy (07023 – EPA). This gauging station was not given a classification under FSU and following the rating review the Q_{med} changed significantly (from 15.3 to 11.8 m³/s). The EPA does note that weed growth has been a problem in the past at this station. The Q_{med} extracted from the NAM modelled AMAX series at this station is 10.0 m³/s. The Tremblestown Gauging Station (07001 –

<p>OPW) is located approximately 7km downstream of the Athboy AFA extents. This gauging station has an FSU classification of A2 but only for the record period pre May 1971. For this period the Q_{med} is 11.29 m³/s. Further examination of the rating information provided by OPW Hydrometric Section suggests that there is some confidence in the rating up to 1999. From 1999 until 2010 no spot gaugings were recorded by OPW and there is a noticeable gradual upward shift in the flow values for that period suggesting the rating is unreliable for the period.</p> <p>While all the watercourses being modelled are part of the ADS and may be subject to dredging the Athboy model represents the current status as of July 2012 when the watercourses were surveyed.</p>		
(2) Model Reference:		HA07_ATHB3
(3) AFAs included in the model:		Athboy
(4) Primary Watercourses / Water Bodies (including local names):		
<u>Reach ID</u>	<u>Name</u>	
0728	Athboy River	
0729	Athboy Trib	
(5) Software Type (and version):		
(a) 1D Domain: MIKE 11 (2011)	(b) 2D Domain: MIKE 21 - Rectangular Mesh (2011)	(c) Other model elements: MIKE FLOOD (2011)

4.1.2 Hydraulic Model Schematisation

(1) Map of Model Extents:
<p>Figure 4.1.1 Figure 4.1.2 illustrate the extent of the modelled catchment, river centre line, HEP locations and AFA extents as applicable. The Athboy River catchment contains 2 Upstream Limit HEPs, 1 Downstream Limit HEP, 4 Tributary HEPs and 2 Gauging Station HEPs.</p>

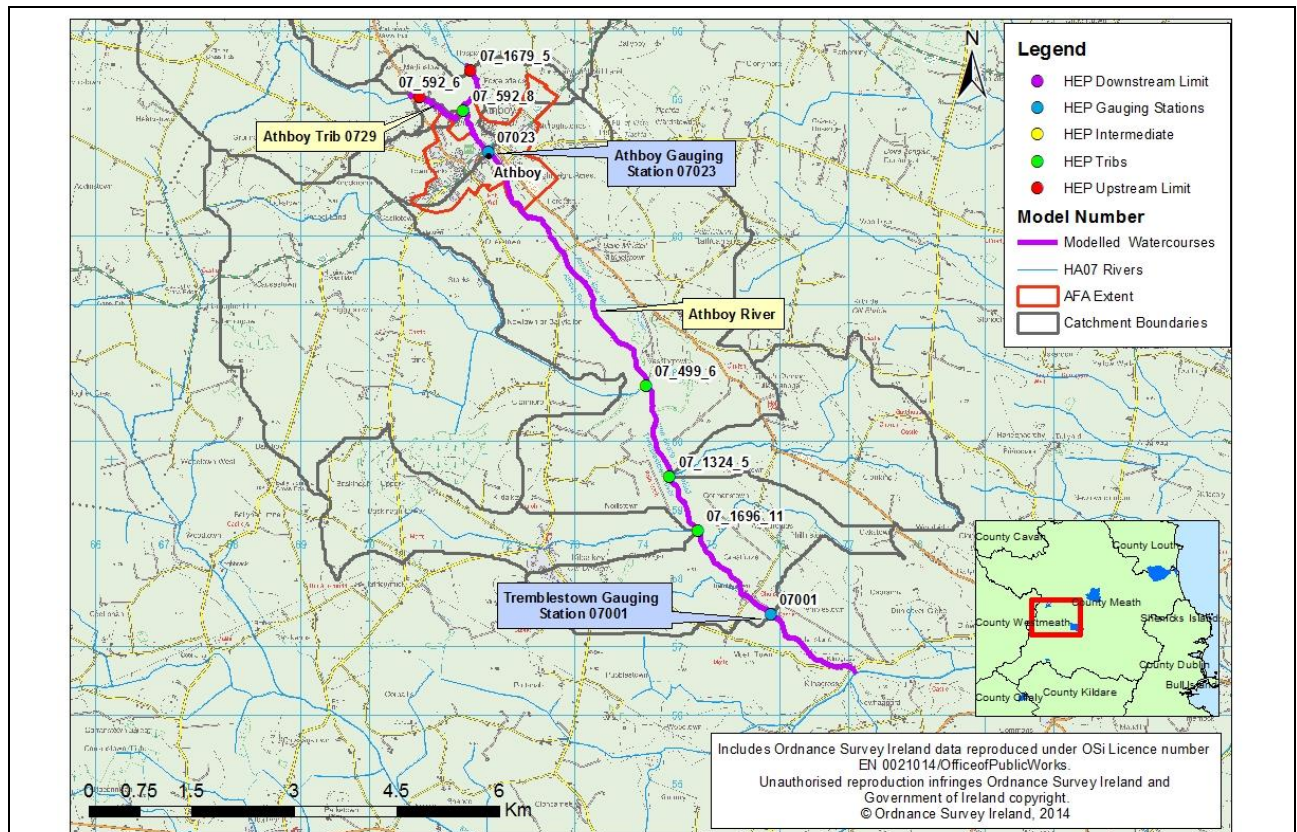


Figure 4.1.1: Athboy Model Overview

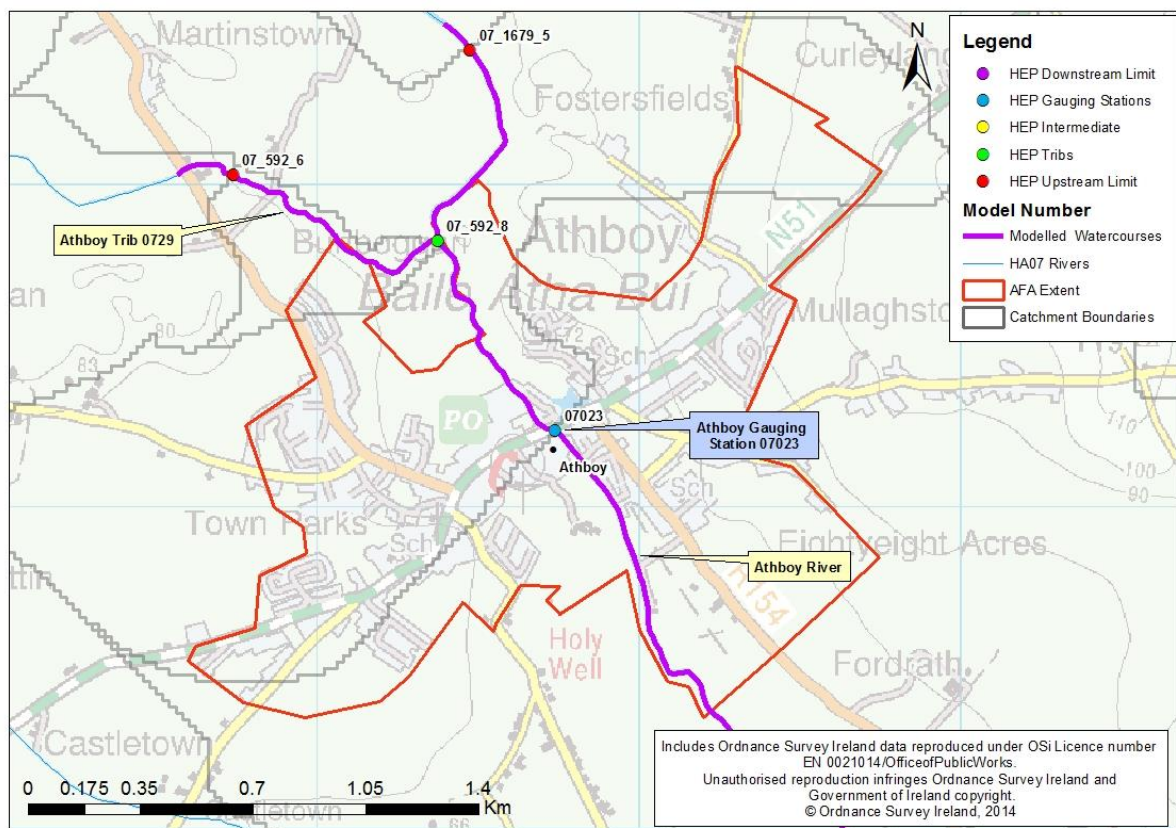


Figure 4.1.2: Athboy Model AFA Overview

Figure 4.1.3 and Figure 4.1.4 show the location of the cross sections used in the model along with the location of the 2D links and the critical structures.

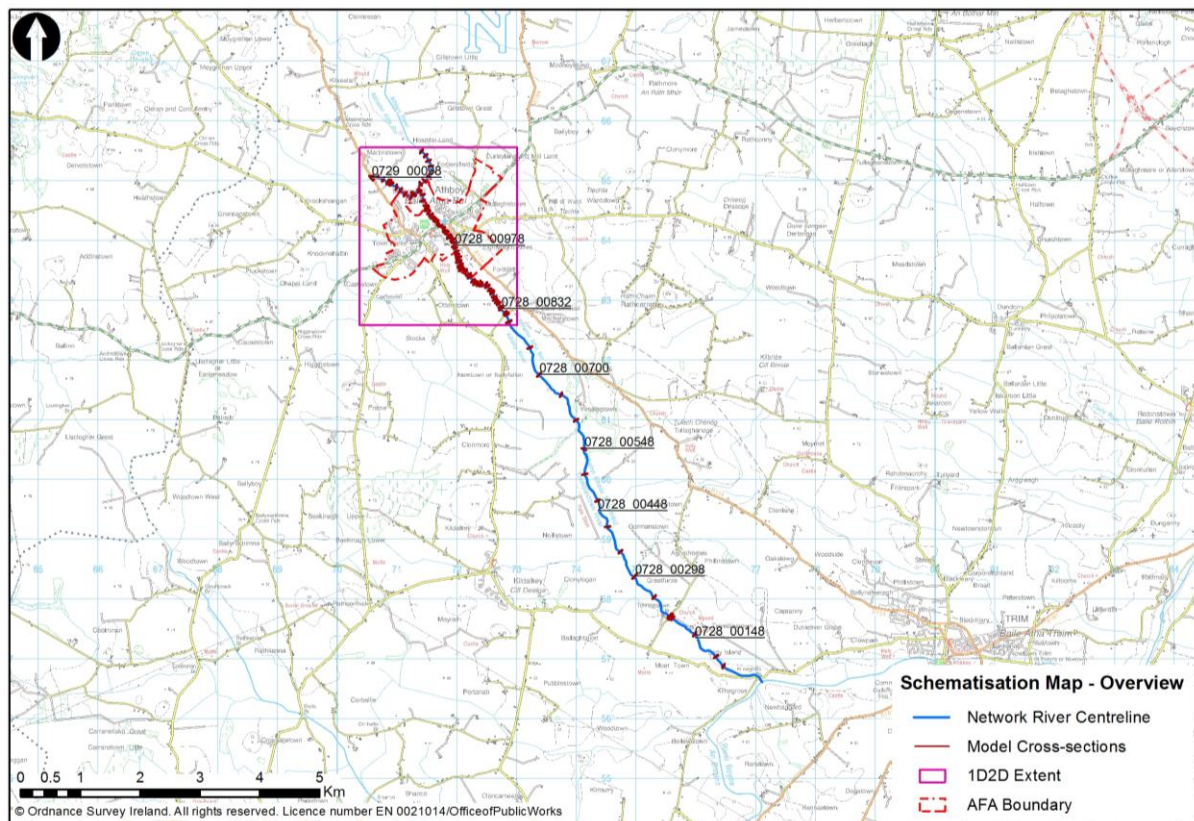


Figure 4.1.3: Overview of Model Schematisation

For clarity in viewing cross-section locations, the model schematisation diagrams show the full extent of the surveyed cross-sections. Note that the 1D model considers only the cross-section between the 1D-2D links.

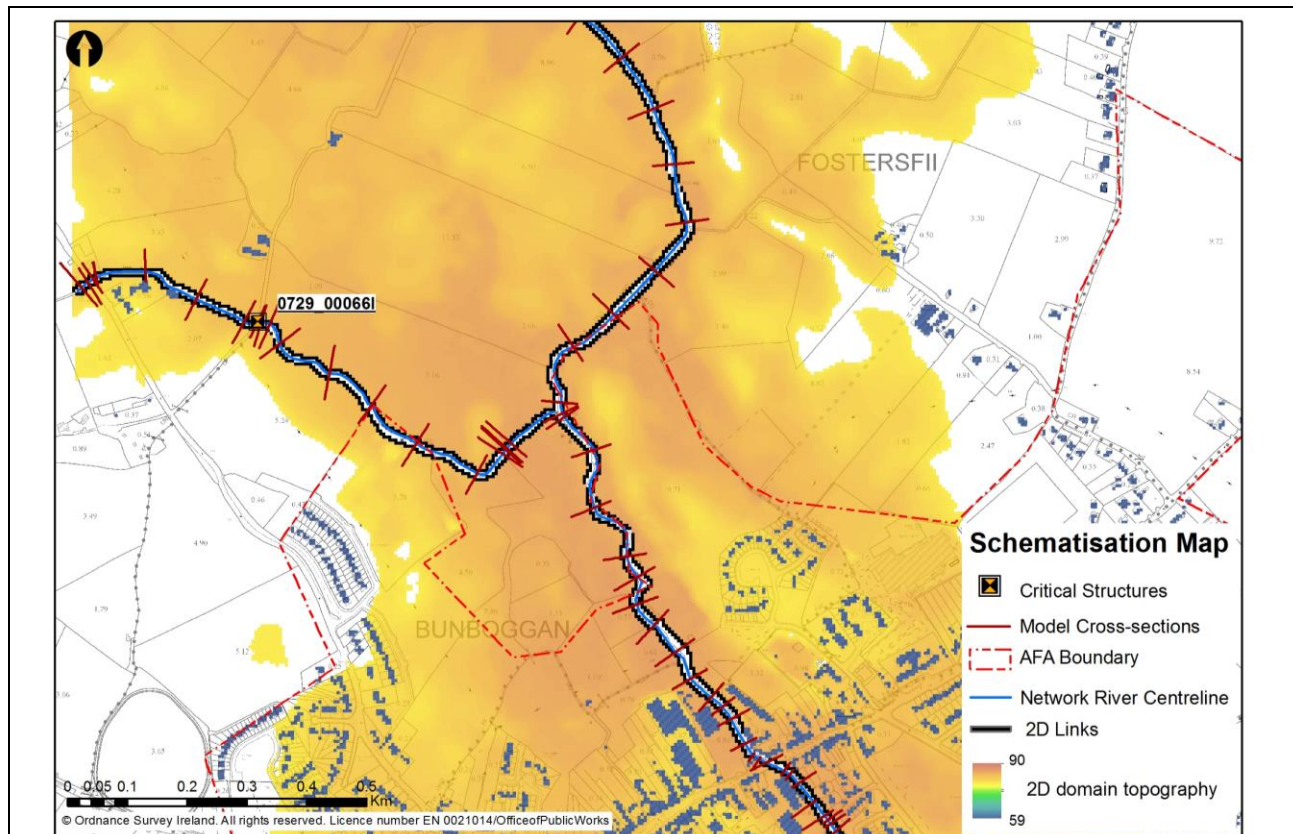


Figure 4.1.4: Model Schematisation AFA Overview

(2) x-y Coordinates of River (Upstream extent):

River Name		x	y
0728	ATHBOY RIVER	271393	265501
0729	ATHBOY TRIB	270565	265032

(3) Total Modelled Watercourse Length:		12.742km	
(4) 1D Domain only Watercourse Length:	9.05km	(5) 1D-2D Domain Watercourse Length:	3.69km
(6) 2D Domain Mesh Type / Resolution / Area:		Rectangular / 5 metres / 6.46km ²	
(7) 2D Domain Model Extent:			

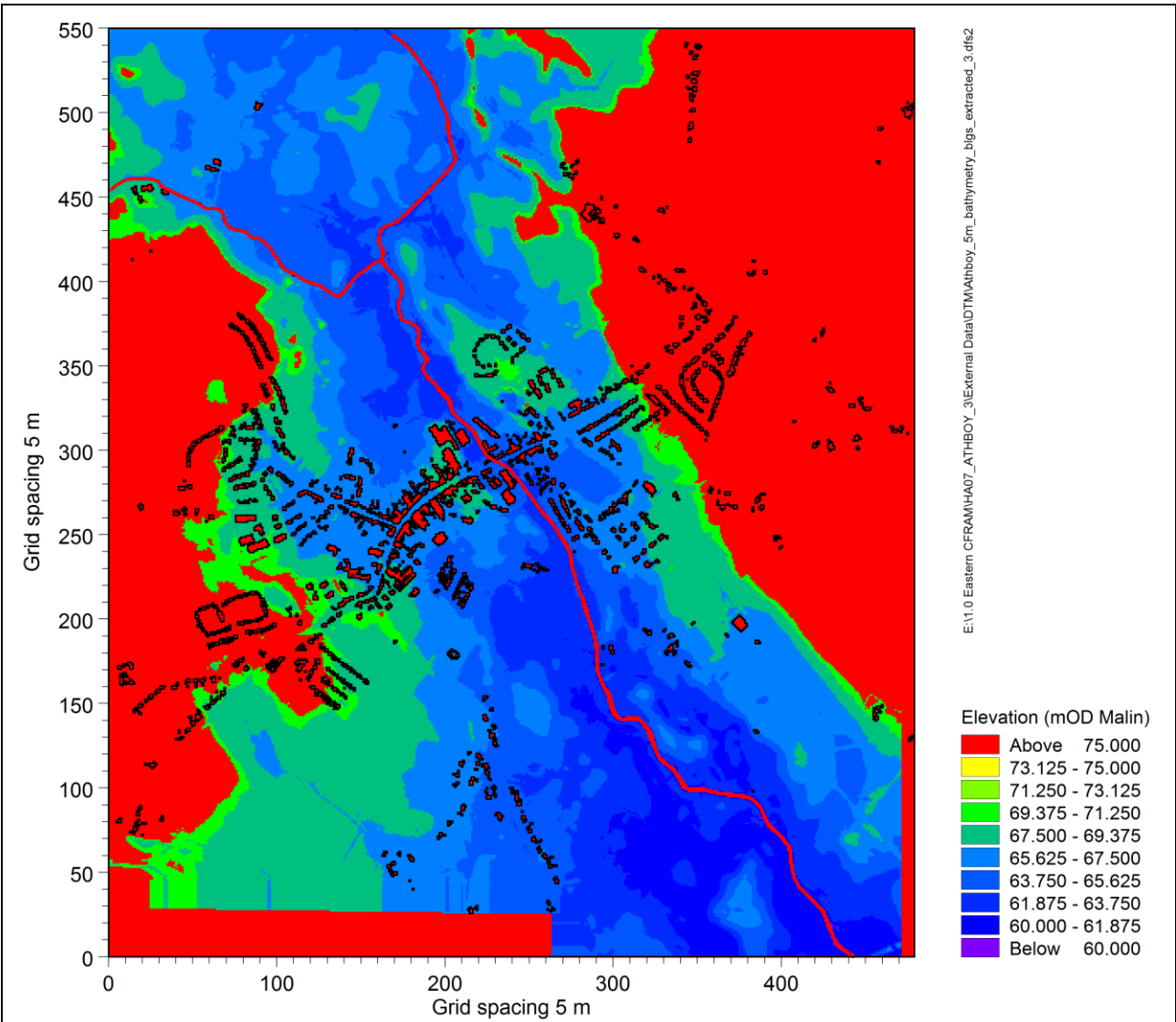


Figure 4.1.5: Athboy Model AFA Overview

Figure 4.1.5 illustrates the bathymetry file showing the modelled extents of Athboy and the general topography of the catchment. The 2D domain was generated using LiDAR survey data and created as a 5m grid rectangular mesh, see Chapter 3.2 for more detail on grid mesh size. No post processing was required to the bathymetry file.


(8) Survey Information

(a) Survey Folder Structure:

First Level Folder	Second Level Folder	Third Level Folder
Murphy_E07_MO3_WP1_0728A_120702 Athboy Murphy – Surveyor Name E07 – Eastern CFRAM Study Area	GIS and Floodplain	Structure Register
	Photos	Surveyed Cross Section Lines
	Ascii	
	Drawings and PDFs	

Hydrometric Area 07 MO3 – Model Number 3 WP1 – Work Package 1 0728A – River Reference 120702 – Date Issued 2 July 2012	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	
(b) Survey Folder References:		
Reach ID	Name	File Reference
0728	ATHBOY RIVER	Murphy_E07_M03_WP1_0728A_120702 Murphy_E07_M03_WP1_0728B_120620 Murphy_E07_M03_WP1_0728C_120620 Murphy_E07_M03_WP2_0728A_ADD_120927
0729	ATHBOY TRIB	Murphy_E07_M03_WP1_0729_120620
(9) Survey Issues:		
No Survey Issues		

4.1.3 Hydraulic Model Construction

(1) 1D Structures (in-channel along modelled watercourses):	See Appendix A Number of Bridges and Culverts: 6 Number of Weirs: 3
	
<p>Figure 4.1.6: Photograph of Critical Structure on the Athboy Tributary</p> <p>Critical structure identified on the Athboy Tributary at chainage 340m. The culvert surcharges and causes</p>	

out of bank flooding on the left hand bank.					
(2) 1D Structures in the 2D domain (beyond the modelled watercourses):		None			
(3) 2D Model structures:		None			
(4) Defences:					
Type	Watercourse	Bank	Model Start Chainage (approx.)	Model End Chainage (approx.)	
No Formal					
No Informal					
(5) Model Boundaries - Inflows:					
Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report - Section 4.3 and Appendix D). The boundary conditions implemented in the model are shown in Table 4.1.1.					
Table 4.1.1 Location and type of boundary conditions applied to the Athboy model					
Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Boundary ID
Open	Inflow	ATHBOY RIVER	0	0	07_1679_5
Open	Inflow	ATHBOY TRIB	0	0	09_592_6
Point Source	Inflow	ATHBOY RIVER	6002	0	07_499_6
Point Source	Inflow	ATHBOY RIVER	7450	0	07_1324_5
Point Source	Inflow	ATHBOY RIVER	8405	0	07_1696_11
Open	Q-h	ATHBOY RIVER	11767	0	
Distributed Source	Inflow	ATHBOY RIVER	1642	10112	Top-up between 07023 & 07001
Distributed Source	Inflow	ATHBOY RIVER	10113	11695	Top-up between 07001 & 07_971_6

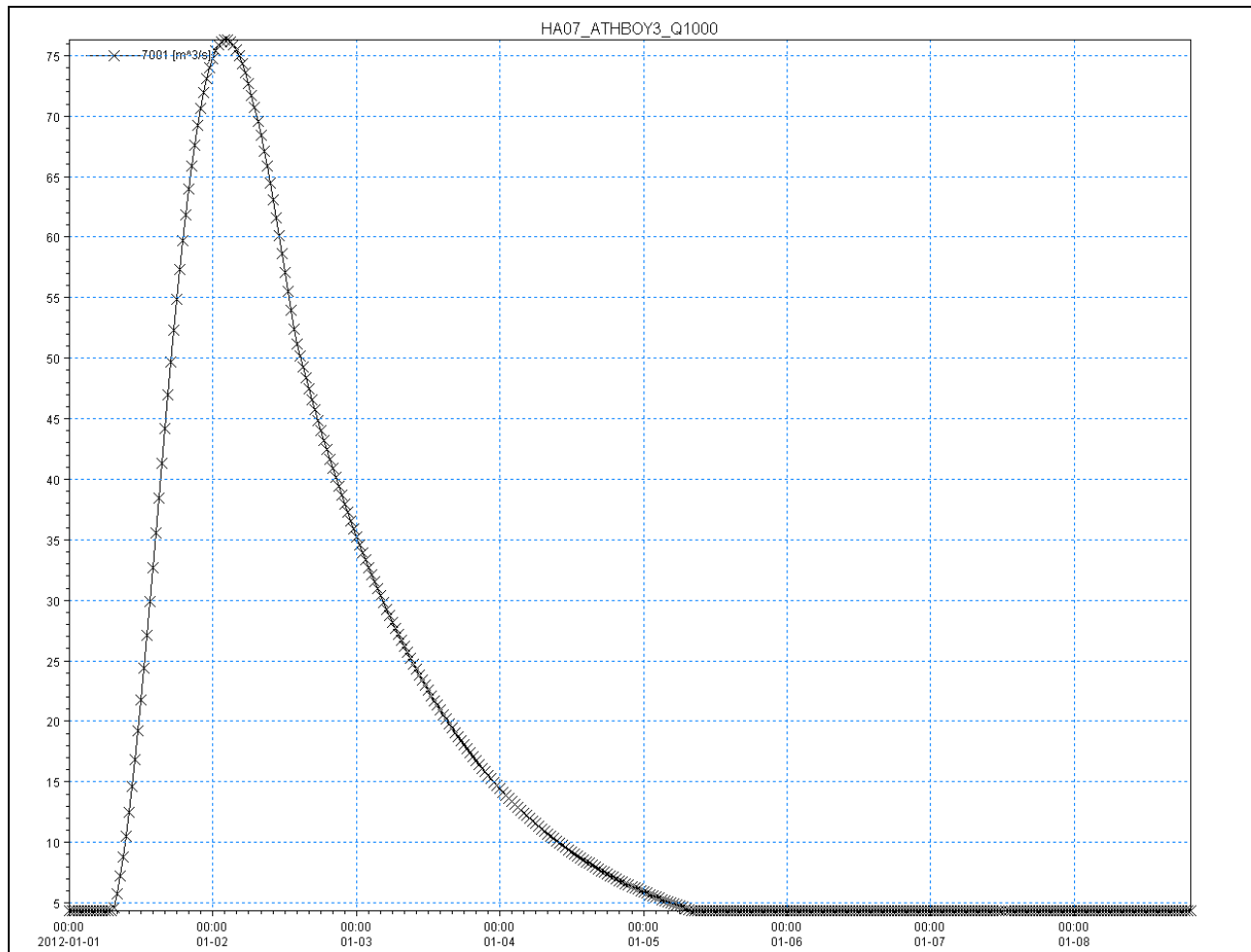


Figure 4.1.7 Inflow Hydrograph to the Athboy River during the 0.1%AEP Event at HEP 07_1679_5

The hydrology was reviewed during the rating review at Athboy Town Gauging Station and was found to produce consistent results in the full Athboy model. There was therefore no change made to the hydrology. Please view Section 4.1.5(2) which discusses model updates for Final deliverables.

**(6) Model Boundaries –
Downstream Conditions:**

The downstream boundary condition is a Q-h relationship, generated based on Manning's equations and uses the downstream extent of the model with a slope of 0.001. This is located at the confluence with the River Boyne.

To ensure continuity between adjacent models, in this case between the Athboy model and the Trim model located downstream, a check was carried out to ensure that the flow output from the Edenderry model matched that of the flow in the Trim model at the location of the Athboy River confluence. As such the 1%AEP hydrographs at the downstream extent of the Edenderry model and in the Trim model were compared as shown in Figure 4.1. 8

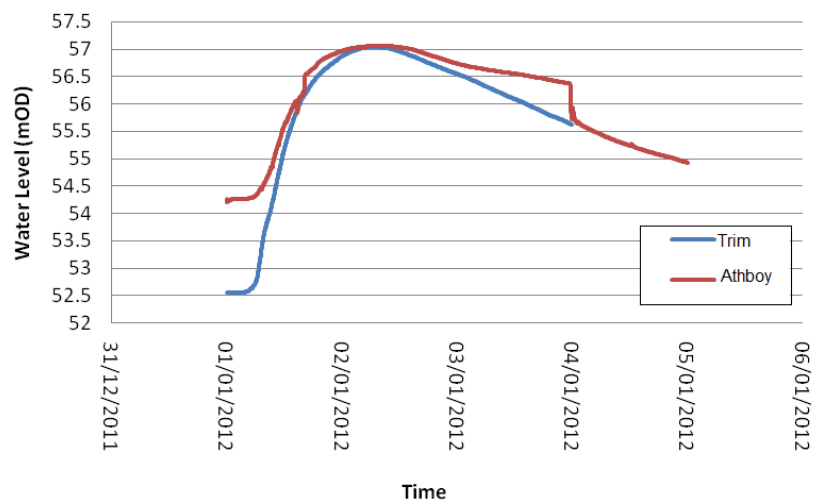


Figure 4.1. 8: Comparison of Hydrographs between Adjacent Models Edenderry and Trim

A good correlation was found in peak water level and duration giving confidence that the downstream boundary of the Athboy River is representative of the water levels in the River Boyne.

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.020	Maximum 'n' value: 0.100
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.035
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.035 (Inverse of Manning's 'M')	Maximum 'n' value: 0.045 (Inverse of Manning's 'M')

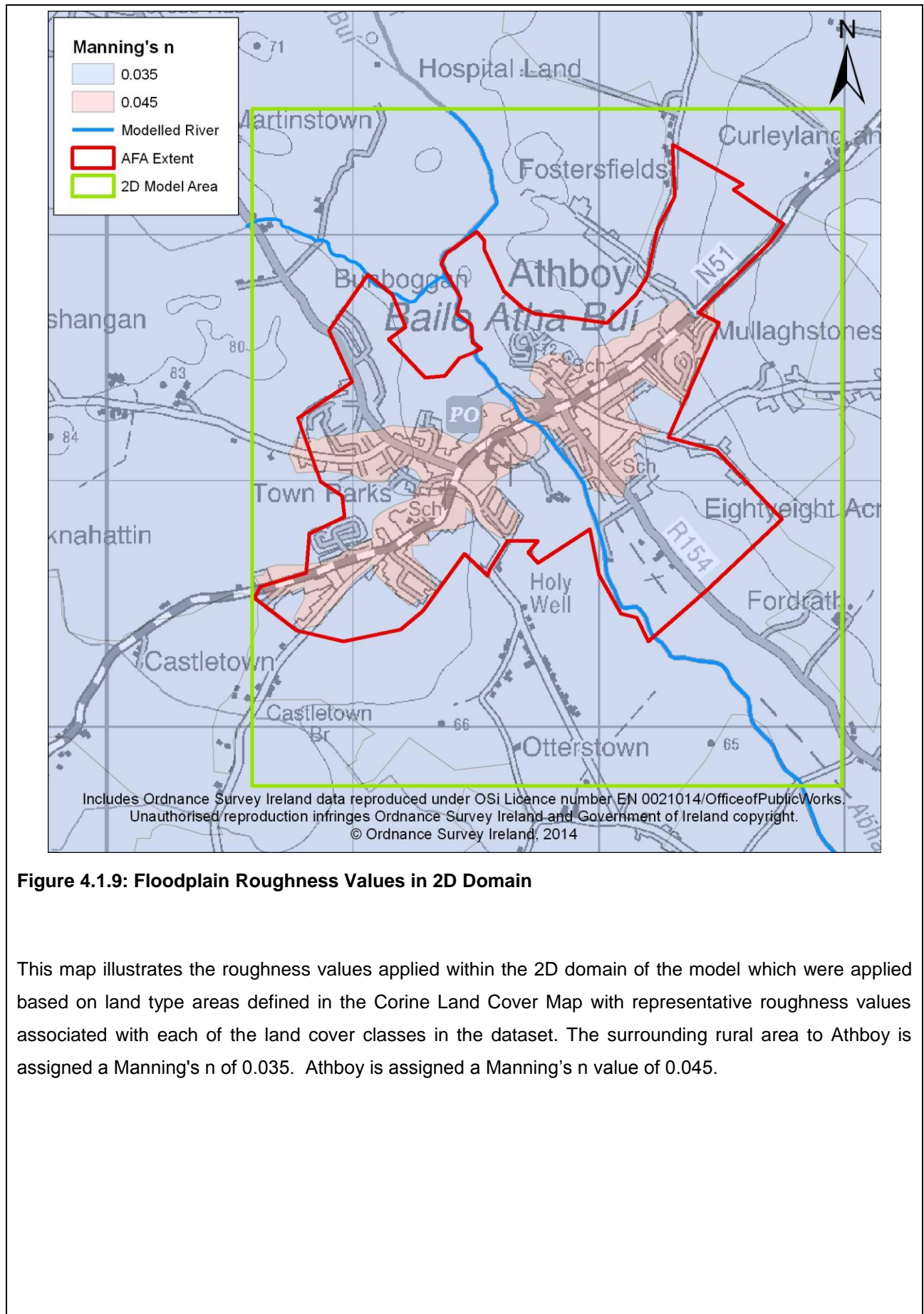


Figure 4.1.9: Floodplain Roughness Values in 2D Domain

This map illustrates the roughness values applied within the 2D domain of the model which were applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset. The surrounding rural area to Athboy is assigned a Manning's n of 0.035. Athboy is assigned a Manning's n value of 0.045.

(d) Examples of In-Bank Roughness Coefficients**Athboy River**

Figure 4.1.10: Example of roughness coefficients on the Athboy River at cross section 0728_01125

Manning's $n = 0.100$

Thick hedgerow with overhanging branches along left bank

Athboy Trib

Figure 4.1.11: Example of roughness coefficients at culvert 0729_00065J on the Athboy Trib

Manning's $n = 0.020$

Concrete culvert



Figure 4.1.12: Example of roughness coefficients at bridge 0728_0007D on the Athboy River

Manning's $n = 0.035$ along river bed and block work
Manning's $n = 0.120$ to represent low reaching branches

Low hanging tree branches partly blocking the left arch and fully blocking the right arch



Figure 4.1.13: Example of roughness coefficients at bridge 0728_00204D on the Athboy River

Manning's $n = 0.035$ along river bed and block work
Manning's $n = 0.070$ to represent the brush around the left and right arches

Heavy brush restricting flow through both arches

4.1.4 Sensitivity Analysis

A number of sensitivity simulations have been conducted with the purpose is to assess the sensitivity and impact of the 1% AEP fluvial hydraulic model where appropriate within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness increased to upper bound values and flood plain roughness decreased to lower bound values – The change in channel and floodplain roughness values resulted in a high increase in flood extents within Athboy AFA, as shown in Figure 4.1.14. The Athboy model therefore has a high sensitivity to adjusting the roughness parameters. This high increase in flood extents results in no further impact to AFA receptors.

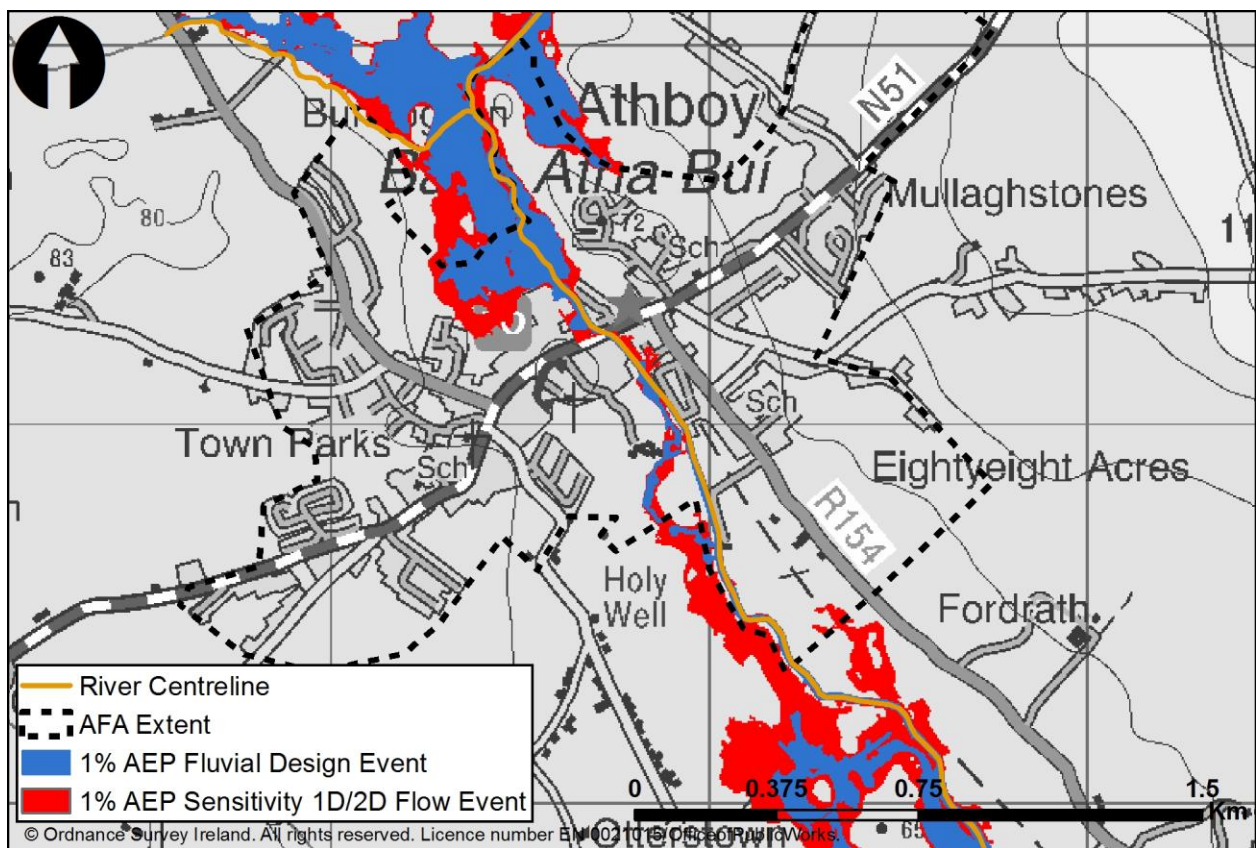


Figure 4.1.14 Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in roughness

- b) Downstream boundary increase – it has been determined for Athboy that the downstream water level boundary has no impact on the AFA. Therefore, a downstream water level boundary sensitivity run was not required.
- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Athboy model was assessed as having medium uncertainty/sensitivity and therefore factors of 1.57 and 1.67 were applied to the design flows for the sensitivity (refer to the Hydrology Report IBE0600Rp0012 for further detail). There is uncertainty in the gauge data at Athboy due to ratings and catchment run-off characteristics consequent of arterial drainage. Figure 4.1.15 shows

that the Athboy model has a high sensitivity to increased inflow parameters; this is reflected by the increase of flood extents. A significant increase of the flood extent is observed around Lower Bridge Street, particularly those properties located within a close proximity to the Athboy River to the north of the AFA. These changes have a high impact on receptors as approximately 7 additional buildings are affected. This is a relative 175% increase when compared to the 1% AEP design results.

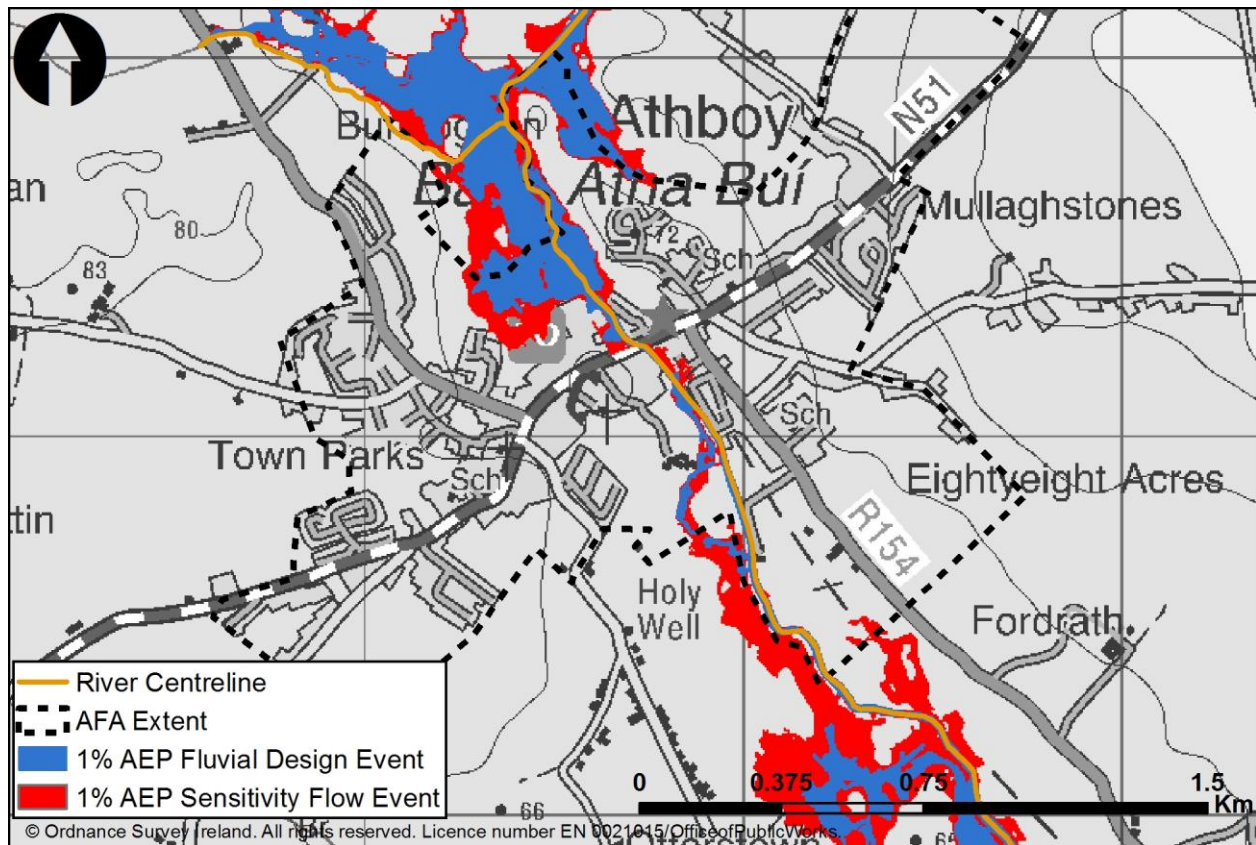


Figure 4.1.15 Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in flow

- d) Flow volume – A sensitivity adjustment factor of 2 has been applied to flood duration to assess the effect on the model. Flood durations in this case have been derived from observed data with some uncertainty at flood flows. Overall there is a low increase in flood extents within the AFA, with the largest increase occurring outside of the AFA boundary as shown in Figure 4.1.16. The Athboy model is considered to have a low sensitivity to flow volume parameters, as no additional properties impacted.

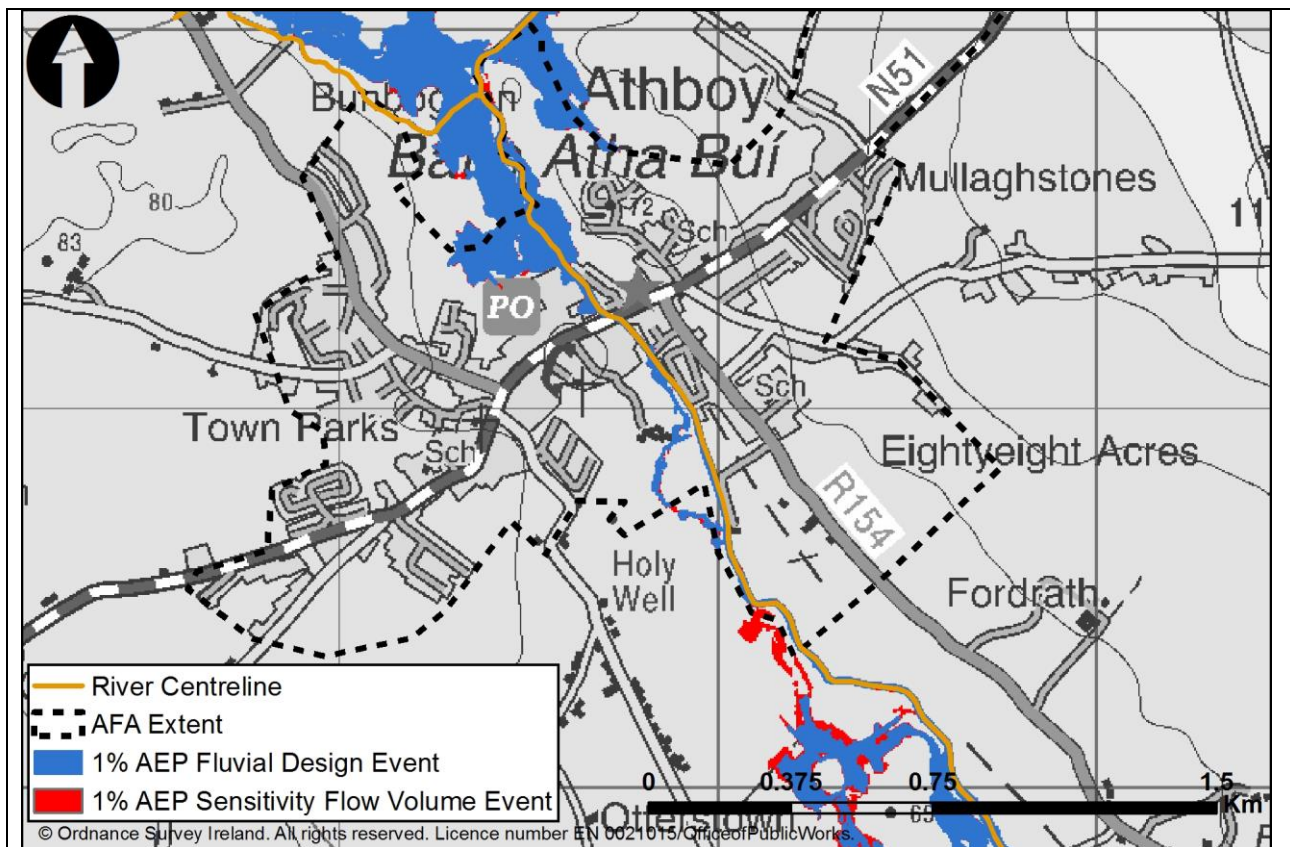


Figure 4.1.16: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Volume Event

- e) Afflux/Head loss at key structures – A single simulations were carried out to assess the sensitivity of flood extents to changing the head loss coefficients of key structure 0728_01020D. This structure allows the N51 to cross over the Athboy River and is almost located at the centre of the Athboy AFA. As presented in Figure 4.1.17 the Athboy model indicates a low sensitivity to parameter changes as there was little change in flood extents and no impact to receptors.

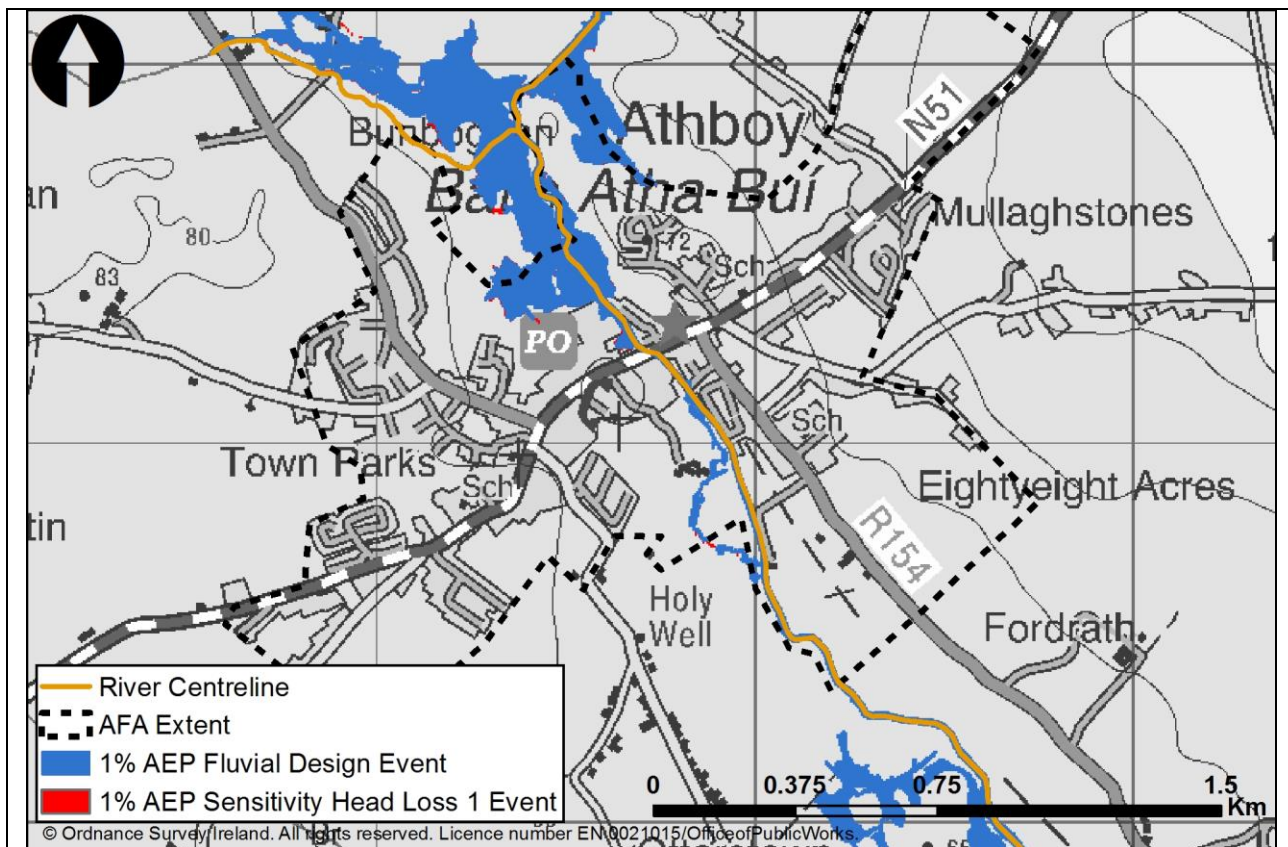


Figure 4.1.17: Comparison between 1% AEP Design Event and 1% AEP Sensitivity (Head Loss 1)

- f) Building representation – Building representation was modelled by adjusting the roughness of cells within the building footprint to a Manning's n of 0.3. The topography within the 2D model domain was based on LiDAR - the cells within building footprints remained 'unblocked'. Figure 4.1.18 shows that the Athboy model has low sensitivity to building representation, as revealed by the overall low increase in the 1% AEP flood extent. This negligible change results in no further impact on to receptors located within the AFA.

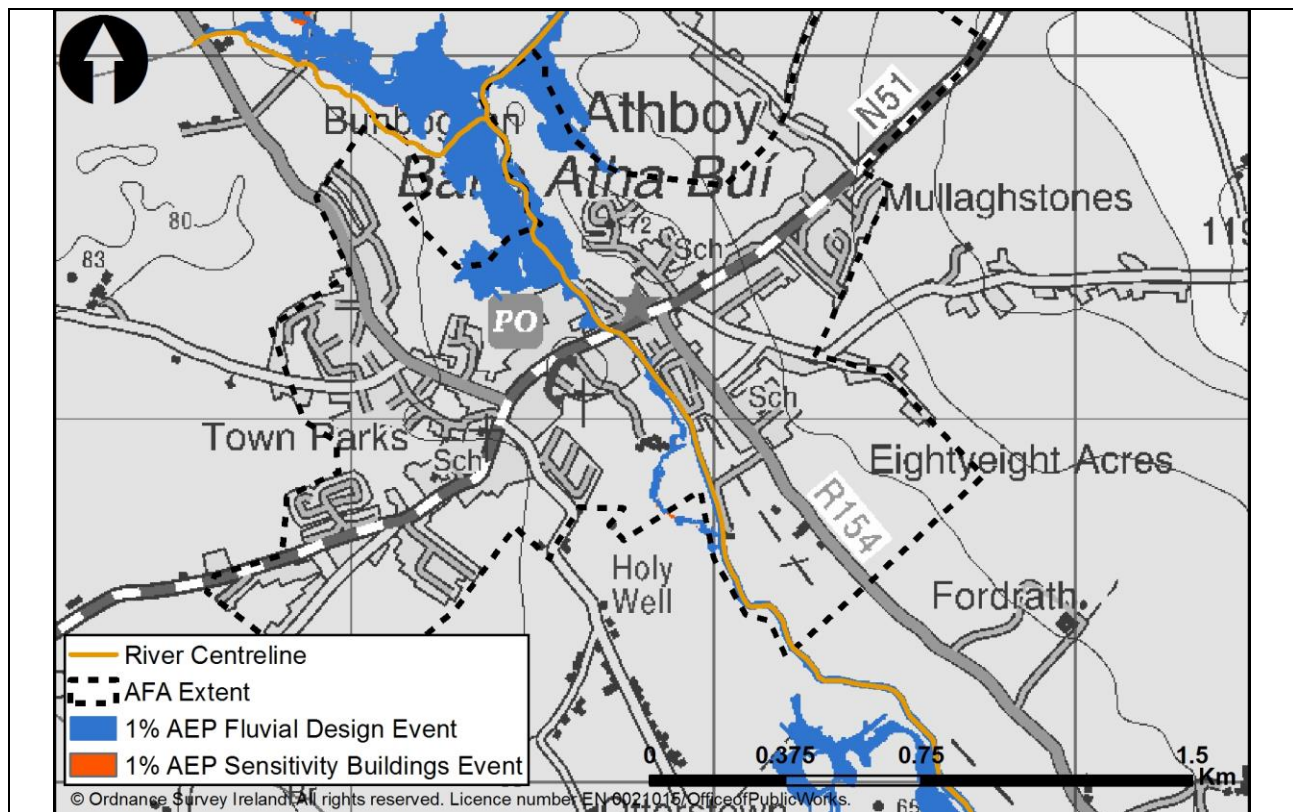


Figure 4.1.18: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Buildings Event

Table 4.1.2 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Athboy model. Of these parameters assessed, the model demonstrates a high sensitivity to inflow. Table 8.1 in the Hydrology Report states that there is medium to low uncertainty associated with the hydrological inputs for this model, principally due to a changing gauged catchment attributed to arterial drainage. The model demonstrates lesser sensitivity to the other parameters, with the resulting analysis identifying low increases in flood extents and low impact to receptors within the AFA

Table 4.1.2: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	High	-
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	High	High
1% AEP Sensitivity Flow Volume Event	Low	-
1% AEP Sensitivity Head Loss 1 Event	Low	-
1% AEP Sensitivity Buildings Event	Low	-

4.1.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (From IBE0600Rp0004_HA07 Inception Report unless otherwise specified):	
(a) Aug 2008	<p>Flooding occurred in Athboy and Edenderry in August 2008. Heavy rainfall on 16th August resulted in the River Boyne overtopping its banks in Navan. The level reading at Trim Hydrometric Station was the 7th highest on record. In the Athboy environs, at least one house in the Castletown area was flooded on 16th August. The peak flow recorded at the Athboy Hydrometric Station reached 24.42m³/s derived from a recorded level of 1.92m</p> <p>The estimated flood event that occurred in 2008 was between a 10% and 6.67% AEP event (ref: Eastern CFRAM Study HA07 Inception Report). The modelled water depth at the Athboy Gauging Station during a 10% AEP event is estimated to be 1.83m and during a 1% AEP event to be 2.19m. This correlates well with the recorded level of 1.92m and suggests that the event was closer to a 10% AEP event rather than a 1% AEP event which the estimation as detailed in the HA07 Inception Report confirms.</p> <p>The house which flooded in the Castletown area was found to be outside the model extent and flooded by a watercourse not being modelled under the CFRAM study. There were no other records of properties flooding in the Athboy area. The modelled flood extents for the 10% and 1% AEP events also show no properties flooding in Athboy.</p>
(b) Dec 1978	<p>Information was found for a flood event which occurred in Drogheda, Navan, Trim, Mornington and Balray in December 1978. A maximum flow of 130m³/s was recorded at Trim hydrometric station (07005) where the River Boyne overflowed. Griffin Park, Athboy Road, Market Street, Haggard Street, High Street, St. Joseph's home and St. Mary's secondary school were all mentioned as being flooded.</p> <p>There was no record of flooding in Athboy town during this flood which was estimated to be between a 50% - 20% AEP event in the Athboy River. This is consistent with the predicted flood extents in the model in that no properties are shown as flooded between the 10% - 1% AEP events.</p>
(c) Dec 1954	<p>A flood event was found to have occurred in Drogheda, Mornington, Navan and Trim in December of 1954. In Trim, the River Boyne overflowed its banks and the water level rose above the parapets of the "new bridge". Press articles reported flooding of houses on Mill Lane and Athboy Road.</p> <p>There was no record of flooding in Athboy town during this flood. The hydrometric data for the Athboy River extends as far back as 1975. An estimate of the flood event is not readily achievable and the event is therefore of limited use in verification</p>

	of the model.
<p>Summary of Calibration</p> <p>The lack of historical flooding information at Athboy gives little quantitative data to calibrate the model to the larger flood events. The model was calibrated to the available spot gaugings at Athboy Town gauging station and the roughness values of the model adjusted accordingly.</p> <p>A comparison between estimated and modelled flows at HEP points was carried out and the details of which are presented in section 4.1.5 (5).</p> <p>A mass balance analysis was carried out to assess the difference between the discharge into the model and the volume of water stored with the discharge out of the model. Results showed a difference of 0.42%. This is within the acceptable limits as stated in the Environment Agency's Fluvial Design Guide.</p> <p>Whilst anecdotal information and available data has been used to the best extent possible, overall there is little or poor data to calibrate the model to and observation of more events would be necessary to reduce the uncertainty in model results.</p>	
<p>(2) Post Public Consultation Updates:</p>	
<p>Following informal public consultation and formal S.I. public consultation periods in 2015, analysis of the results of the rating reviews and their impact on the hydrological analysis for the UoM07 watercourses was finalised. The updated inflow hydrograph at HEP 07_1679_5 is shown in Figure 4.1.19. This resulted in the hydrology and mapping outputs being updated, however there were no changes made to hydraulics. A revised set of flood hazard and risk mapping has been issued as Final to reflect this change.</p>	

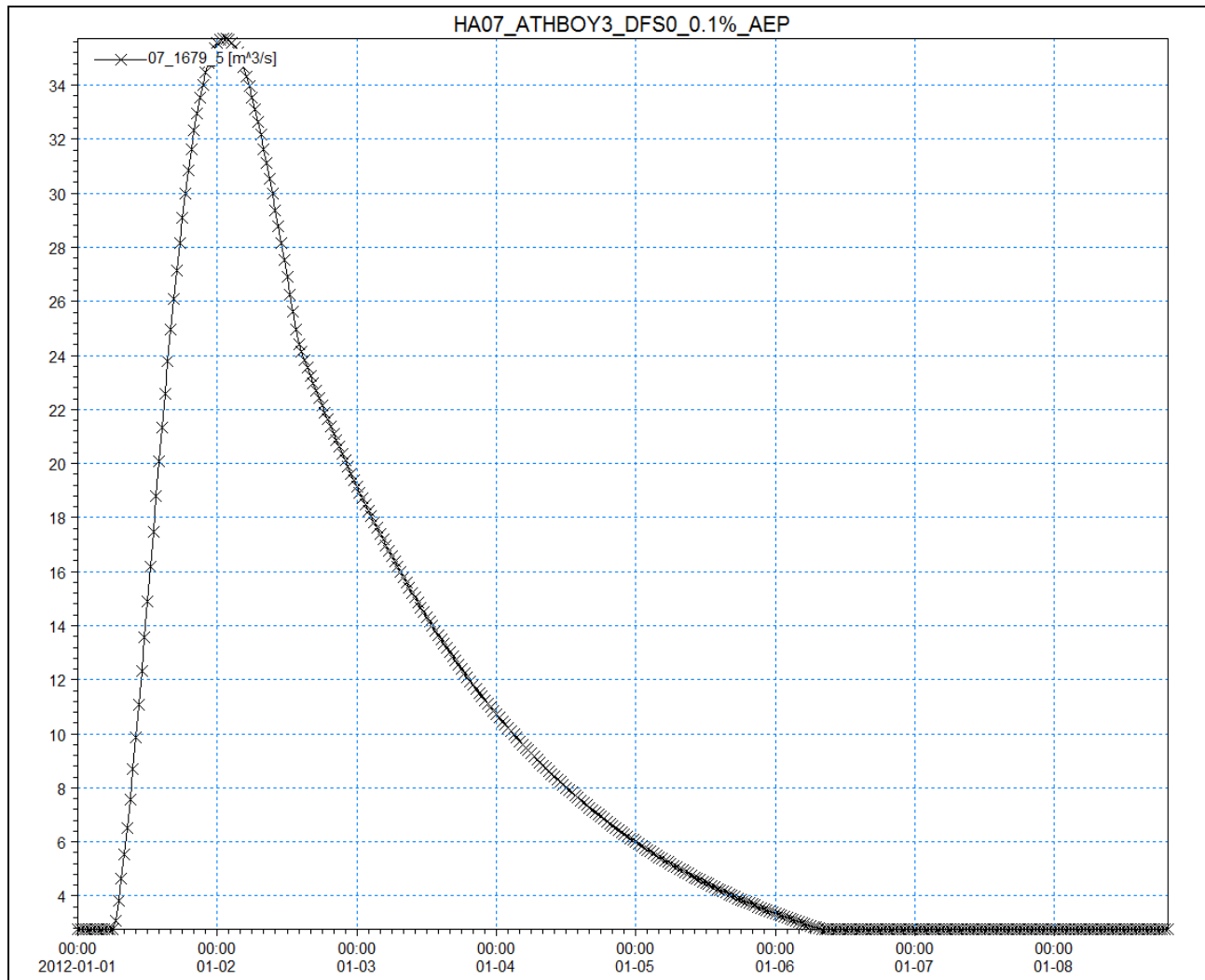


Figure 4.1.19 shows the inflow hydrograph to the Athboy River during the 0.1% AEP event at HEP 07_1679_5

(3) Standard of Protection of Existing Formal Defences:

Defence Reference	Type	Watercourse	Bank	Modelled Standard of Protection (AEP)
No Formal Defences				

(4) Gauging Stations:

There are two gauging stations within the model extent, both have water level and flow information available.

(a) Athboy (07023)

Operated by the EPA. It is located in Athboy Town downstream of the main bridge at Bridge Street.

An attempt was made to verify the model to the recorded gaugings. Only gaugings post 2002 were used in this assessment. This is a similar approach taken by the EPA who took no rating between 1988 and

2002 mainly due to rocks being put in the river by fisheries/angling club to artificially raise the water level. A good correlation was achieved at low flows with all spot gaugings being within acceptable limits (less than 200mm). It is concluded that the model's in bank flow is calibrated to the observed spot gaugings within acceptable tolerances. This gauging station is subject to a rating review – for details please refer to IBE0600Rp0012_HA07 Hydrology Report.

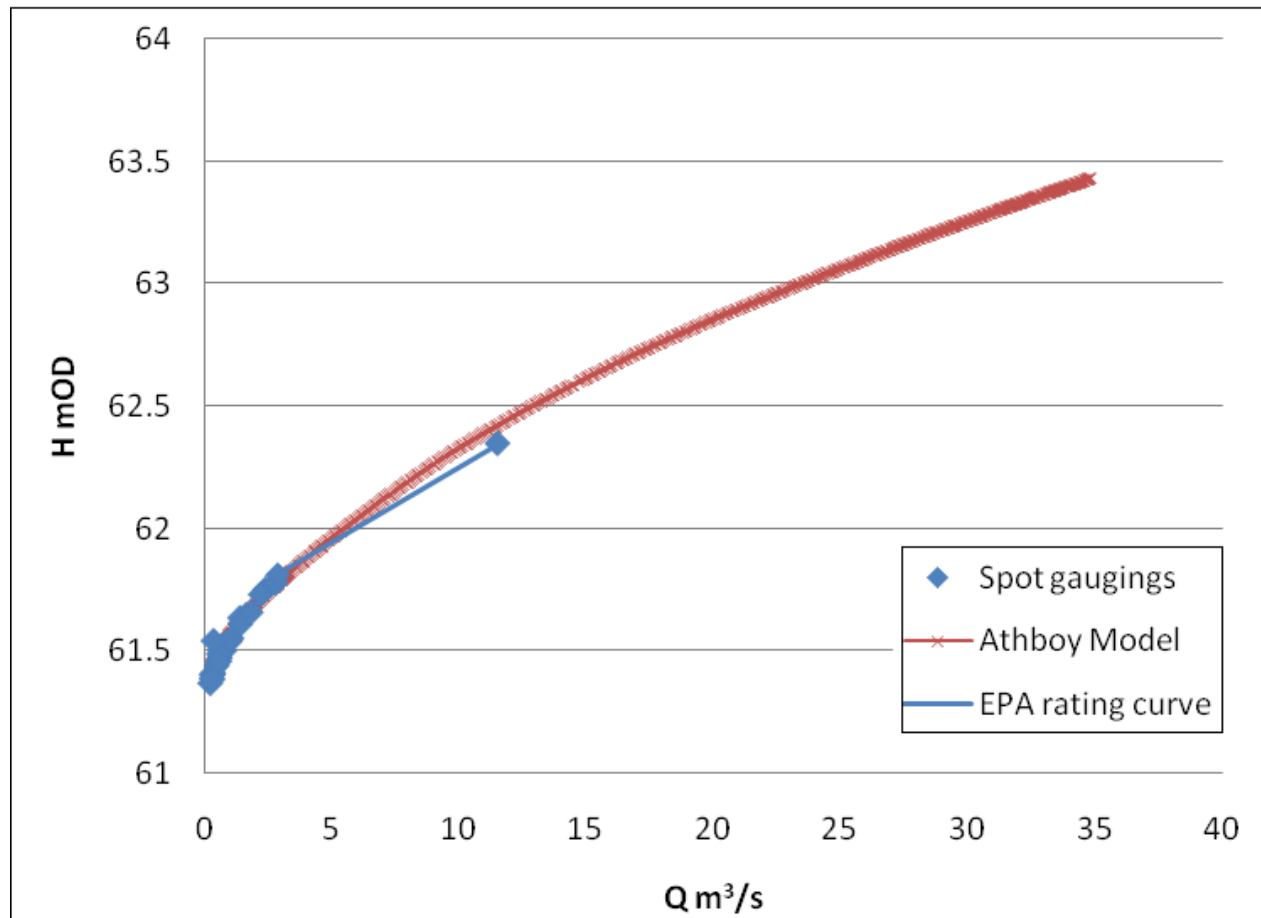


Figure 4.1.20: QH Relationship Comparison between Athboy model and EPA Rating Curve at Athboy River Gauging Station

(b) Tremblestown (07001)

Operated by the OPW. The gauging station is located in the rural area of Tremblestown downstream of Athboy and near the confluence to the River Boyne. An attempt was made to verify the model to the recorded gaugings. A large scatter was found in the spot gaugings, when investigated it was found that for any given flow the water level was rising as the spot gauging were being recorded from 1975 to 1999 as shown in Figure 4.1.21. The likely cause of this is that the Athboy River was dredged around 1975 as part of the ADS and then was left to revert back to its original bed level through the process of siltation. The Athboy model QH curve shows a correlation with the original spot gaugings recorded after the ADS was first carried out suggesting that the channel has been dredged between 1999 and present day. OPW confirmed that dredging works were carried out in 2004/05 which support and verify the findings of the Athboy model.

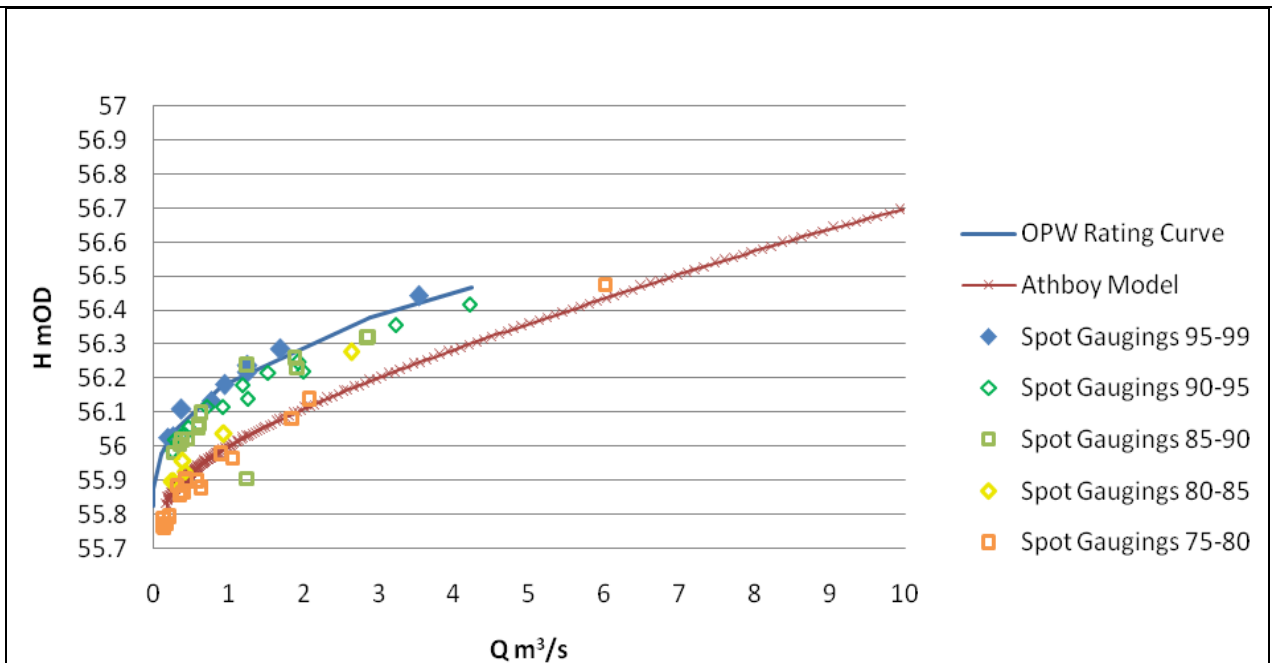


Figure 4.1.21: QH Relationship Comparison between Athboy model and OPW Rating Curve at Tremblestown Gauging Station

(5) Validation with MIKE NAM:**(a) Athboy (07023)**

It can be seen from **Table 4.1.3** that comparison between the NAM model and observed data cannot be made as none of the events have concurrent data. There is significant discrepancy between the observed peak flow on the 16th August 2008 and the model simulated peak flow. This is due to uncertainty in the observed peak flow which is beyond the reliable limit of the rating ($11.5\text{m}^3/\text{s}$).

Table 4.1.3: Comparison of Observed, Hydrologically (NAM) and Hydraulically Modelled Data for Flood Events

	Observed Peak		MIKE FLOOD Simulated Peak at Observed WL	MIKE NAM Simulated Peak	MIKE FLOOD Simulated Peak at NAM Discharge	Water Level Difference
Flood Event	Water Level (mOD)	Discharge (m^3/s)	Discharge (m^3/s)	Discharge (m^3/s)	Water Level (mOD)	(m)
18/08/2008 (10% to 6.67% AEP)	62.848	24.92	18.64	-	-	-
28/12/1978 (50% to 20% AEP)	-	-	-	11.268	62.42	-
09/12/1954 (5% to 10% AEP)	-	-	-	15.172	62.62	-

(6) Comparison of Flows:

Table 4.1.4 provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

Table 4.1.4: Modelled Flows and Check Flows

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m3/s)	Model Flow (m3/s)	Diff (%)
ATHBOY RIVER 00.00	10%	16.21	16.21	0.00
07_1679_5	1%	24.48	24.81	1.35
	0.1%	35.73	33.88	-5.18
ATHBOY RIVER 1642.37	10%	17.75	17.67	-0.45
07023	1%	26.95	27.00	0.19
	0.1%	39.66	33.86	-14.63
ATHBOY RIVER 10112.25	10%	35.87	34.74	-3.16
07001	1%	56.11	60.48	7.76
	0.1%	76.32	86.21	12.95
ATHBOY RIVER 11767	10%	35.91	35.67	-0.66
07_971_6	1%	56.17	61.82	10.05
	0.1%	76.39	85.79	12.30
ATHBOY TRIB 00.00	10%	1.79	1.79	0.00
07_592_6	1%	3.35	3.32	-0.81
	0.1%	6.07	6.22	2.44
ATHBOY TRIB 975.00	10%	1.98	1.75	-11.59
07_592_8	1%	3.7	3.20	-13.62
	0.1%	6.71	6.10	-9.12

The table above provides details of flow in the model at every HEP inflow, check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

The percentage difference in the Athboy River between the model and check flows is relatively small.

The modelled flows at the upstream of the Athboy Tributary correlate well with the hydrological estimated flows. However there is a difference of approximately 10% in flows at the downstream extent. This is due to the extensive overland flow and attenuation that occurs from the Athboy Tributary which slows the flow and therefore reduces the overall peak flow along the watercourse as shown in the figure below.

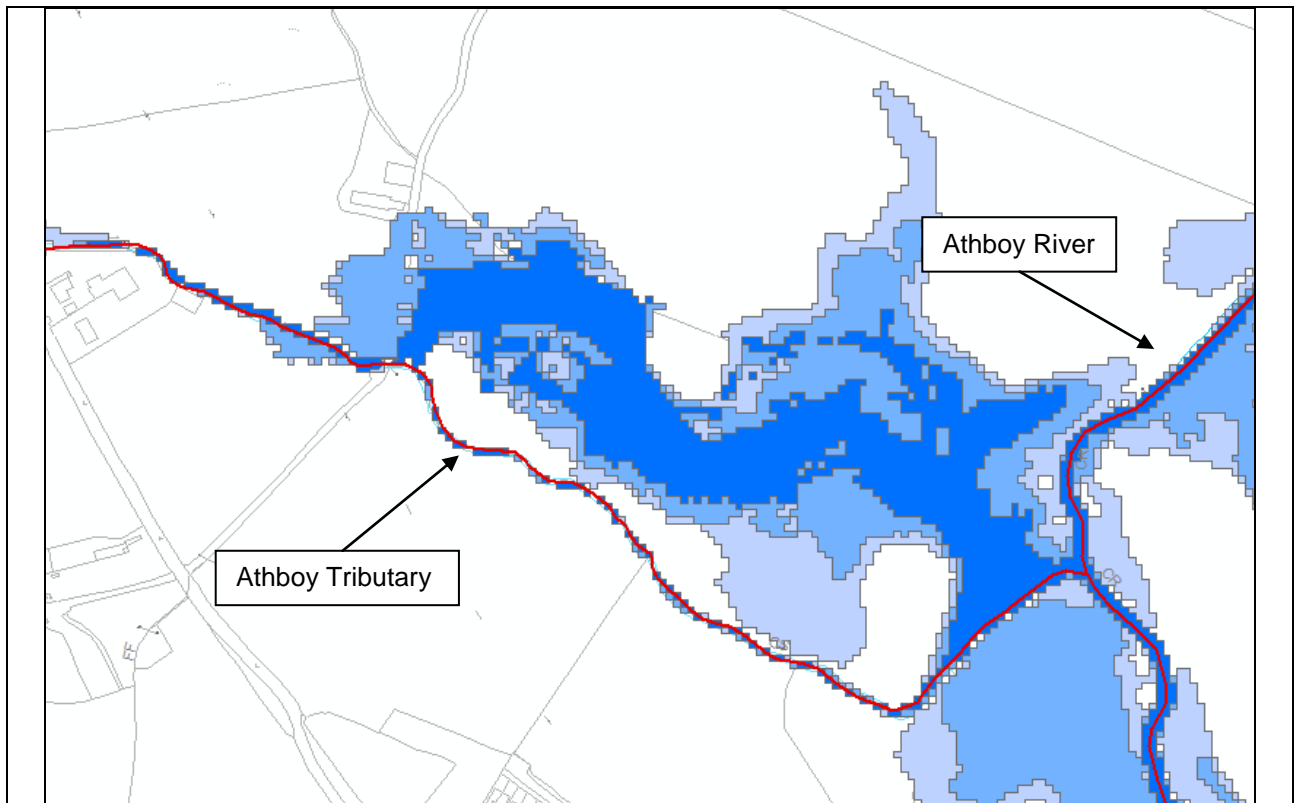


Figure 4.1.22: Flood Extents of the Athboy Tributary and Athboy River

(7) Other Information:

The OPW provided some additional information obtained during a site walkover in February/March 2014 as follows:

- Debris is a potentially significant issue to be considered as part of maintenance / optioneering
- River sections are very overgrown upstream of the bridge and debris could cause blockage, the river was dredged in 2004/05 and the millrace has been filled in and two arches of the bridge are not operational
- The most recent significant flooding was over 50 years ago however flooding in the upstream agricultural portion of the AFA occurs on average every second year
- Flood risk receptors may potentially include the factory, apartments and residential properties at the iron footbridge

The watercourses in the Athboy model are part of the arterial drainage scheme. As such they were assessed and modified to provide improved drainage to land. Part of this scheme involved delineating the benefiting land. This involved a walkover survey by land valuers to identify flood prone and marshy land that would benefit by the improved drainage. An indication can therefore be obtained from the ADS benefiting lands as to the extent of a river's floodplain and therefore be used as verification on the Athboy model. A review was carried out for the Athboy River and tributary.

Generally the flood extents follow the ADS benefitting lands although extents differ where the model does not take account of certain watercourses. In addition it should be noted that the delineation of ADS benefitting land includes land improved by drainage and not solely flood prone land.

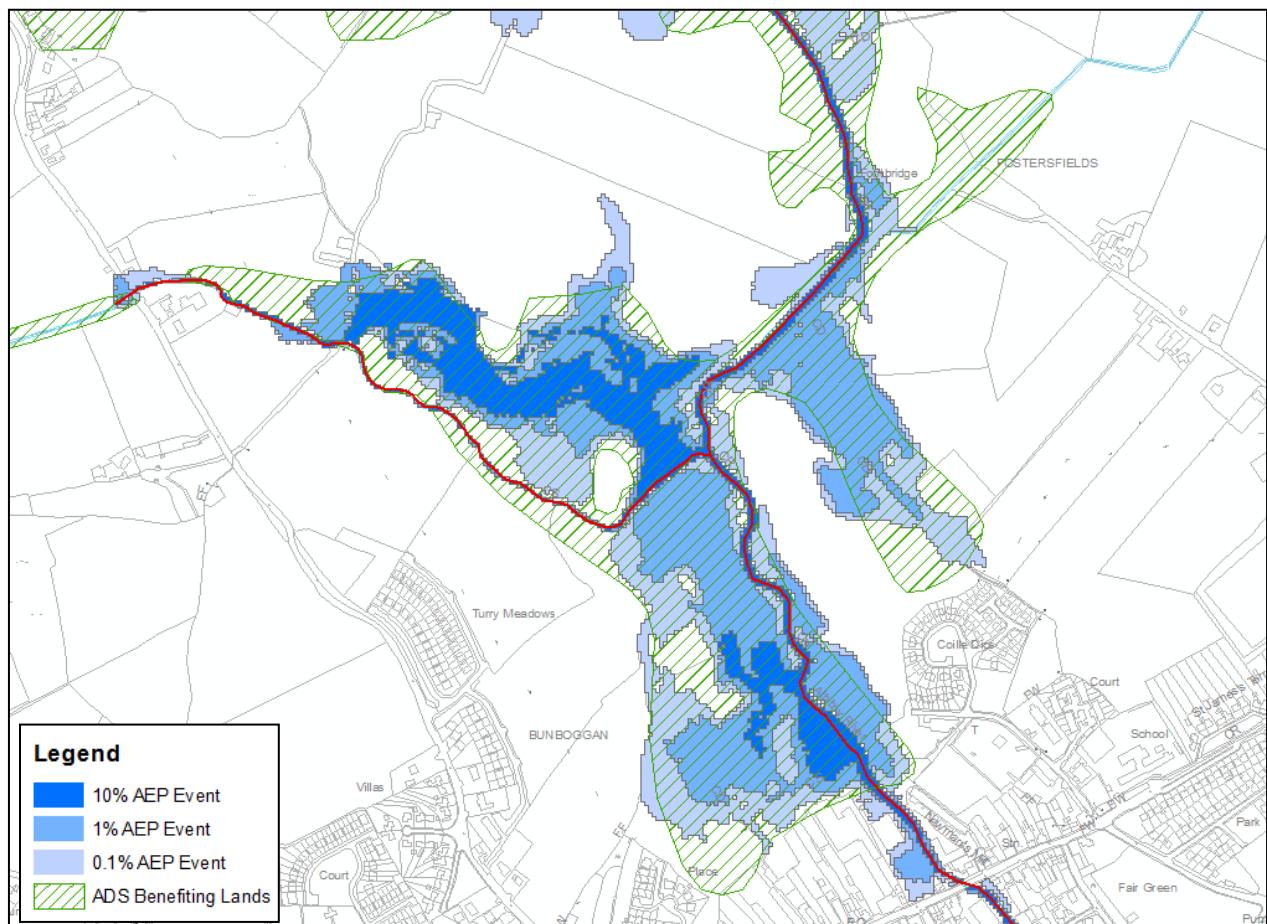


Figure 4.1.23: ADS Benefitting Lands

At the upstream reach of the model the modelled flood extents agree with the ADS benefitting lands where there is a wide floodplain upstream of Athboy town and shown in Figure 4.1.23: ADS Benefitting Lands Figure 4.1.24.

The modelled flood extents show good correlation with the ADS benefitting lands through Athboy, where no flooding is shown, and downstream of the town where the floodplain opens up again.

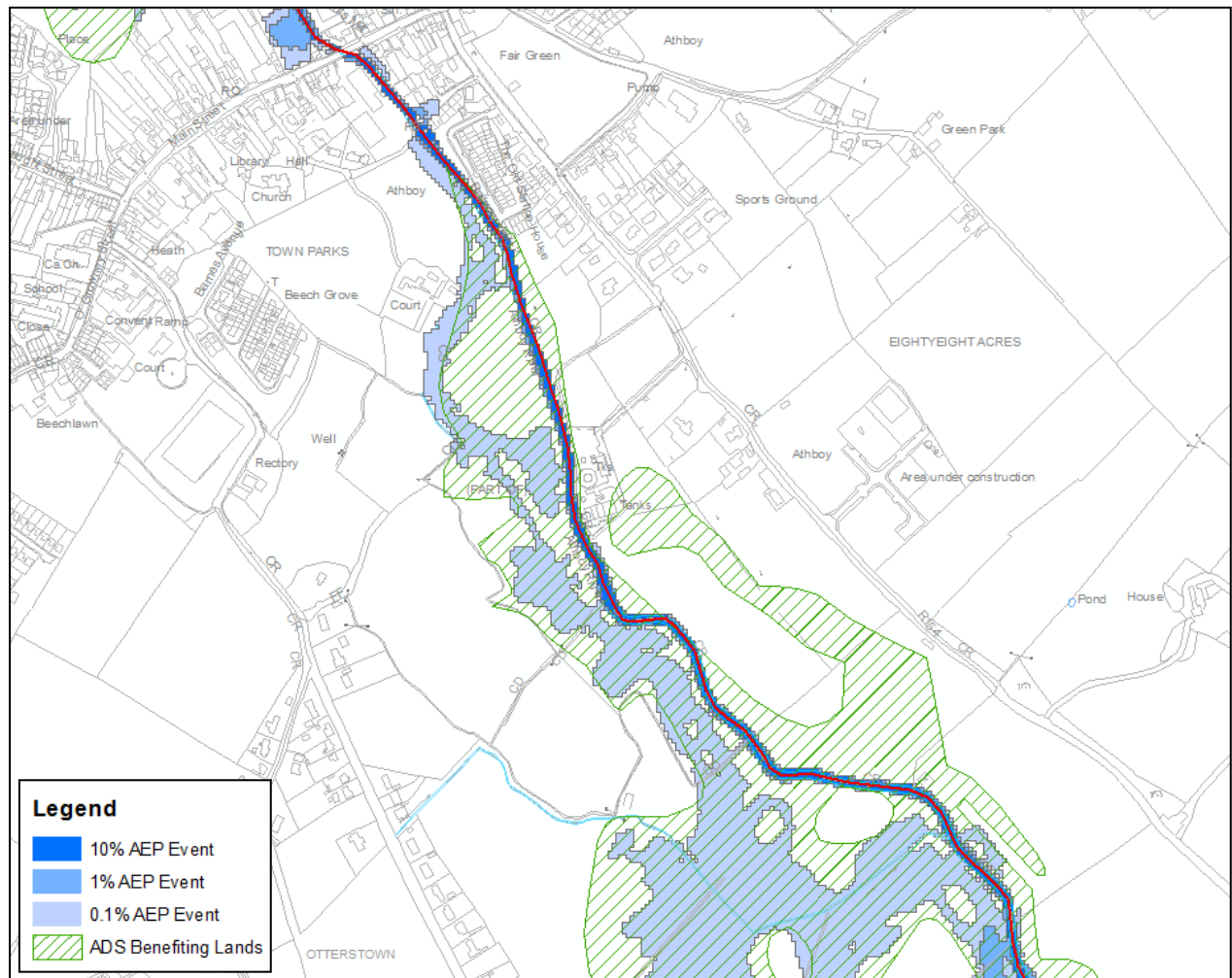


Figure 4.1.24: Comparison between ADS Benefitting Lands and Modelled Flood Extents Downstream of Athboy

Along the MPW, which was modelled in 1D, the flood extents cover a smaller area than the ADS benefitting lands as shown in Figure 4.1.25. Extents differ where the model does not take account of certain unmodelled watercourses in addition it should be noted that the delineation of ADS benefitting land includes land improved by drainage and not solely flood prone land.

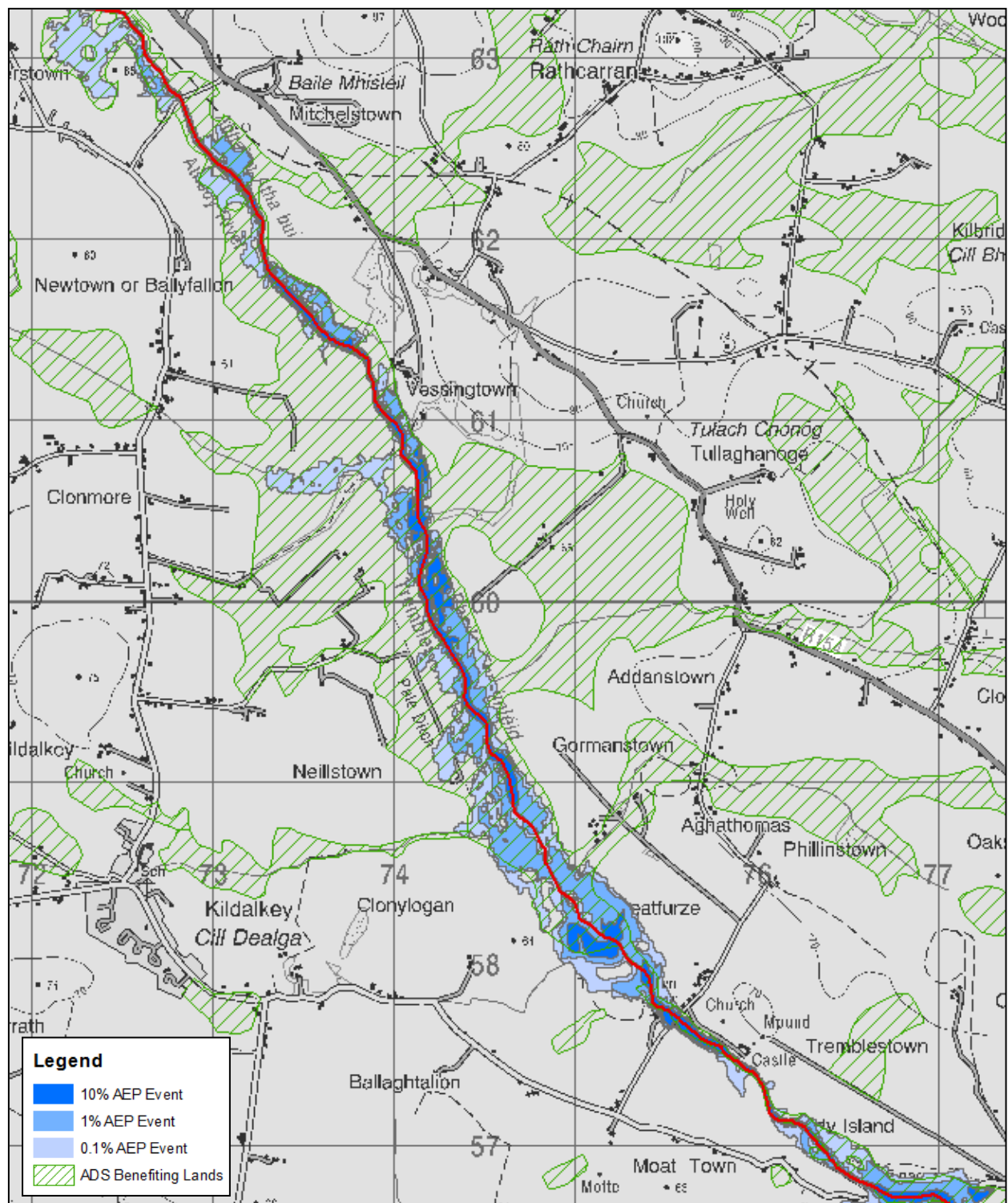


Figure 4.1.25: Comparison between ADS Benefitting Lands and Modelled Flood Extents at the Lower Reach of the Athboy Model

4.1.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

A review of all channels was carried out in order to estimate the roughness coefficients. Along the upper reaches of the Athboy River the banks are delineated with dense hedgerows that occur quite low, close to the channel bed. A higher roughness coefficient was assumed to allow the effects of this to be accounted for.

(2) Hydraulic Model Limitations and Parameters:

(a) A 2 second time-step for both the MIKE 11 and MIKE 21 models has been selected in order to achieve a successful model simulation for all return periods.

(b) It was noted that model instability occurs in the Athboy River at Chainage 9736m where bridge 0728_00204D is located. The reason for the instability was due to the small spacing of cross sections upstream and downstream of the bridge and that the cross sections' geometry changes abruptly. To smooth off the instabilities the cross sections in question were moved in order to increase the spacing, this was carried out with a trial and error approach where the spacing was increased and tested until the instability was removed. The geometry of the cross section was not edited and the location of the bridge remained unchanged. While this approach may reduce the accuracy of the model the water level remained unaffected by these changes. This is due to the bridge acting as a control structure, dictating the Q-H relationship upstream and downstream. The resulting peak water levels for any given flood event remained unaffected by these changes.

(c) The cross sections on the MPW (Ch 5,801m – Ch11,767m) were extended beyond the limit of the topographical survey using the NDHM. A review was carried out post cross section extension to compare the topographical survey level with the NDHM. It was found that for many cross sections a discontinuity was occurring between the two survey datasets. These were reviewed however no consistent error could be found. For many cross sections one banks tied in well with the NDHM and the other had a 0.5m jump. While most cross sections suggest the NDHM is overestimating the ground level there are a few cases where it is underestimating. For other cross sections it would seem that the NDHM is offset horizontally rather than any issue with the vertical data. To this end each cross section was reviewed individually and adjusted so the NDHM tied in with the topography survey. The assumption being that the topographical survey is the more accurate. Where the NDHM data could be offset horizontally to tie in with the topographical survey this was done in the first instance. Secondly where the NDHM seemed to be vertically offset the entire cross section extension to the left or right of the original cross sections was adjusted accordingly to tie it in. Lastly where the discrepancy is minor or the previous two methods are deemed inappropriate the NDHM data was smoothed to tie in with the topographical data.

Hydraulic Model Parameters:	
MIKE 11	
Timestep (seconds)	2
Wave Approximation	Higher Order Fully Dynamic
Delta	0.5
MIKE 21	
Timestep (seconds)	2
Drying / Flooding depths (metres)	0.2 / 0.3
Eddy Viscosity (and type)	0.25 (Velocity Based)
MIKE FLOOD	
Link Exponential Smoothing Factor (where non-default value used)	All Rivers, 1.0
Lateral Length Depth Tolerance (m) (where non-default value used)	Default, 0.1
(3) Design Event Runs & Hydraulic Model Handover Notes:	
<p>The Athboy River is part of the ADS and as such had its channel deepened in 1975.</p> <p>In light of the information gathered at the gauging stations, see section 4.1.5, an assessment was carried out to compare the original cross sections before the ADS, the cross sections immediately after the channel was deepened and/or widened and the cross sections as they are in present day. The historical cross sections are spaced widely and only one cross section was found to be in the vicinity of Athboy town, the location of which is shown in Figure 4.1.26. Figure 4.1.27 shows how much the channel was deepened from its original bed level and then how the bed level has reverted back to its original level by 2012. Historical information records that rocks were introduced into the reach of the river around the Athboy gauging station some 400m upstream of this location in an effort to raise the water levels for angling purposes. This may have increased the rate of sedimentation solely in this location and other areas along the Athboy River may still be at the ADS design bed level. Without a more detailed study this is uncertain however reviewing the spot gaugings at the Tremblestown gauging station, 7km downstream, as detailed in 4.1.5 (4) it would suggest the bed level reverting to its original state as well before the further dredging took place in 2004/05. The result of this assessment is to conclude that the level of protection afforded to the receptors in and around Athboy Town cannot necessarily be attributed to the ADS.</p>	

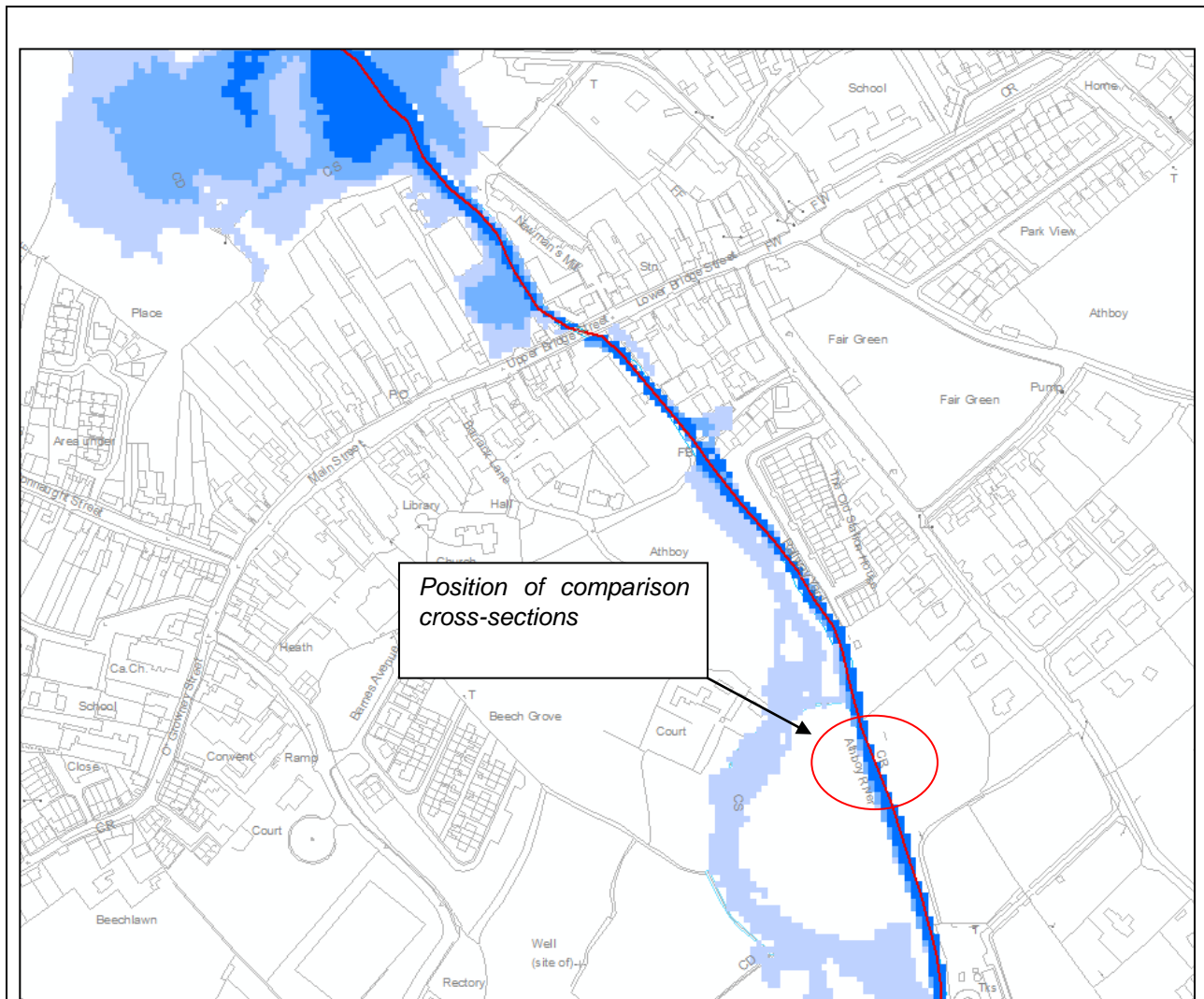


Figure 4.1.26: Comparison between ADS and Current Topographical Cross Sections

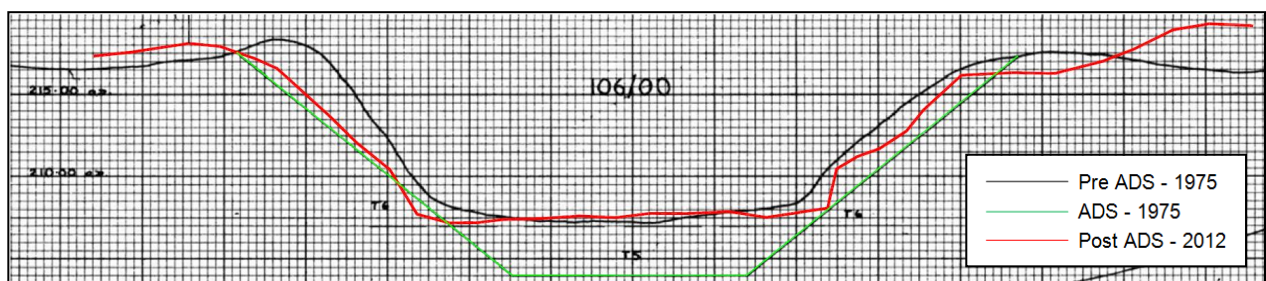


Figure 4.1.27: Comparison between ADS Cross Section and Modelled Flood Extents at the Lower Reach of the Athboy Model

The channel capacity through Athboy town is relatively large and little out of bank flooding occurs even during the 0.1% AEP event, the exception is to the factory warehouse upstream of Bridge Street on the right hand bank. The majority of flooding occurs upstream of Athboy inundating a significant area of agricultural land. This flooding is contributed to by the Athboy River Trib which spills its banks due to an undersized culvert and flows overland to the Athboy River. The land upstream of Athboy is relatively low

compared to the developed land in Athboy which acts as a control causing attenuation upstream rather than allowing increased flow to pass through the town.

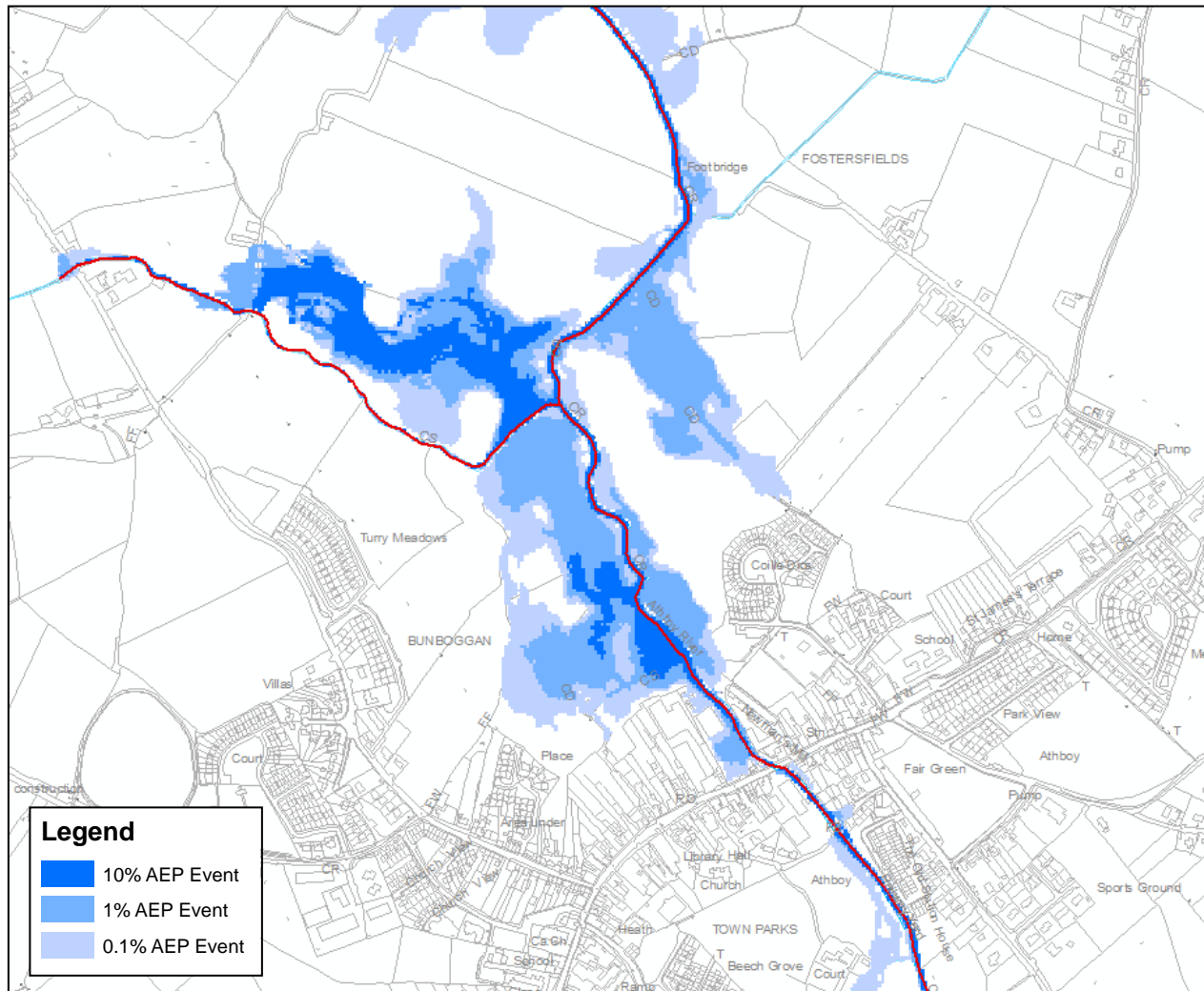


Figure 4.1.28: Flood Extent Upstream of Athboy

(4) Hydraulic Model Deliverables:

Please see Appendix A for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Mark Wilson
Model Reviewed by:	Stephen Patterson
Model Approved by:	Malcolm Brian

4. HYDRAULIC MODEL DETAILS

4.2 BALLIVOR

Name	Local Authority	Unique ID	SSA	Status	Date
Ballivor	Meath	70029	AFA	Final	08/05/2017

4.2.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600 Rp0001_Flood Risk Review) highlighted Ballivor as an AFA for fluvial flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

The Ballivor River system is a tributary of the Stoneyford River, which is itself a tributary of the River Boyne. The model represents the Ballivor River system from its upstream extents, including a number of drains and streams, which emanate from the village of Ballivor and the surrounding area, to the River Boyne, approximately 4km east of the village. The total catchment area of the model is 174.6km² but the vast majority of this contributing area (146.8km²) enters the model downstream of the AFA extents (into the Stoneyford River).

On leaving the town of Ballivor, the Ballivor River bifurcates as shown in Figure 4.2.1 at node 07_60000_1. One channel continues directly to the River Boyne, while the other discharges to the Stoneyford River. It was unclear, at the time of surveying the watercourses, how the flow is distributed between these two channels. Relative flow capacity was assessed by modelling the channels, as accurately captured by the topographical survey, which indicates that both channels convey significant proportions of flow to the River Boyne and the Stoneyford River, 52%, 48% split respectively. This was verified using site visits. For the purposes of the model the channel continuing to the River Boyne is considered part of the Ballivor River while the channel directing flow to the Stoneyford River is considered a separate branch and is called the Ballivor Split as shown in Figure 4.2.1. The floodplain on the left hand bank of the Ballivor River which flows to the River Boyne falls in the opposite direction to the watercourse creating a situation where out of bank flooding flows back towards the Ballivor Split.

All of the modelled watercourses lie within the Arterial Drainage Scheme (ADS) and while these watercourses may be subject to dredging this model represents the current status as of June 2012 when the cross section survey was carried out. There is one gauging station along the Ballivor River, the Ballivor gauging station, which is located within the AFA boundary and has flow and level data. The gauging station Ballivor (07044 – EPA) was assigned a rating of "fair" by the EPA and the rating review limited the rating curve to a stage of 0.41m which equates to a flow of 1.15m³/s. The predicted Q_{med} for the Ballivor River at the gauging station is 3.27m³/s, therefore the rating review proposed using the EPA rating curve

to calibrate the model rating up to 0.35 time the Q_{med} .

The upstream reaches of the Ballivor River (chainage 0m - 3500m) along with its three tributaries and the upper reach of the Ballivor Split (chainage 0m - 858m) are identified as high priority watercourses and are modelled as 1D-2D using the MIKE suite of software. The remainder of the Ballivor River, the lower reach of the Ballivor Split and the Stoneyford River are medium priority watercourses and have been modelled as 1D again using the MIKE suite of software. All watercourses designated HPW and therefore modelled as 1D-2D were extended a suitable distance beyond the limits of the AFA boundary, typically 500m. The reach of the Ballivor River marked as HPW was extended at the downstream end to allow the topography of the floodplain, as mentioned above, to be modelled as 1D-2D. The upper reach of the Ballivor Split was also modelled as 1D-2D, stopping at a suitable break point in the topography provided by a road and bridge at chainage 782m.

No models exist upstream of the Ballivor model. The Ballivor model extends downstream to the confluence with the River Boyne which is part of the Trim model. The downstream boundary conditions for the Ballivor model are based on the water level hydrographs generated by the Trim model at the River Boyne/Ballivor River confluence and the River Boyne/Stoneyford River confluence. An illustration of the model can be seen in Figure 4.2.1.

(2) Model Reference:		HA07_BALL2
(3) AFAs included in the model:		Ballivor
(4) Primary Watercourses / Water Bodies (including local names):		
Reach ID	Name	
0730	BALLIVOR RIVER	
0734	BALLIVOR RIVER TRIB 1 (NAME UNKNOWN)	
0731	BALLIVOR RIVER TRIB 2 (NAME UNKNOWN)	
0732	BALLIVOR RIVER TRIB 3 (NAME UNKNOWN)	
0741	BALLIVOR SPLIT	
0741	STONEYFORD RIVER	
(5) Software Type (and version):		
(a) 1D Domain: MIKE 11 (2011)	(b) 2D Domain: MIKE 21 rectangular mesh (2011)	(c) Other model elements: MIKE FLOOD (2011)

4.2.2 Hydraulic Model Schematisation

(1) Map of Model Extents:

Figure 4.2.1 and Figure 4.2.2 show the extent of the modelled catchment, river centre line, HEP locations and AFA extents as applicable. The Ballivor catchment contains 4 Upstream Limit HEPs, 2 Downstream Limit HEP, 7 Tributary HEPs and 1 Gauging Station HEPs.

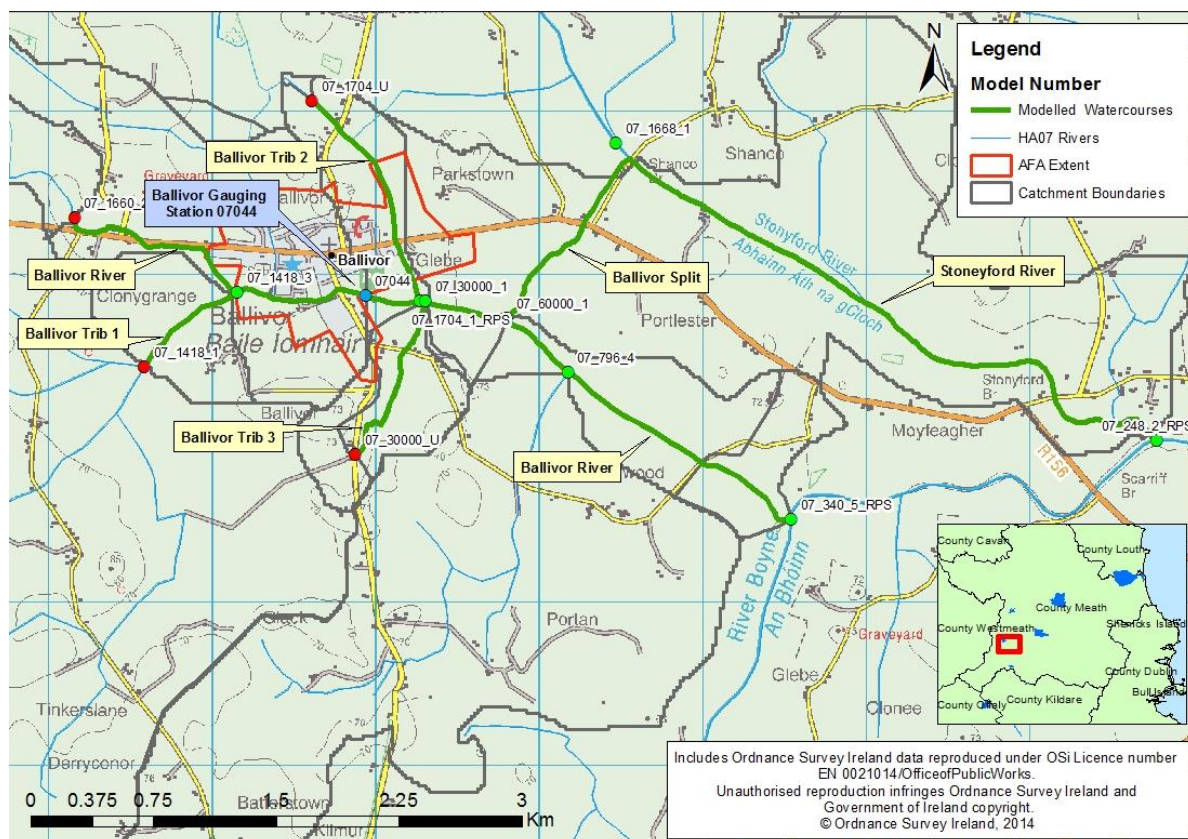


Figure 4.2.1: Ballivor Model Overview

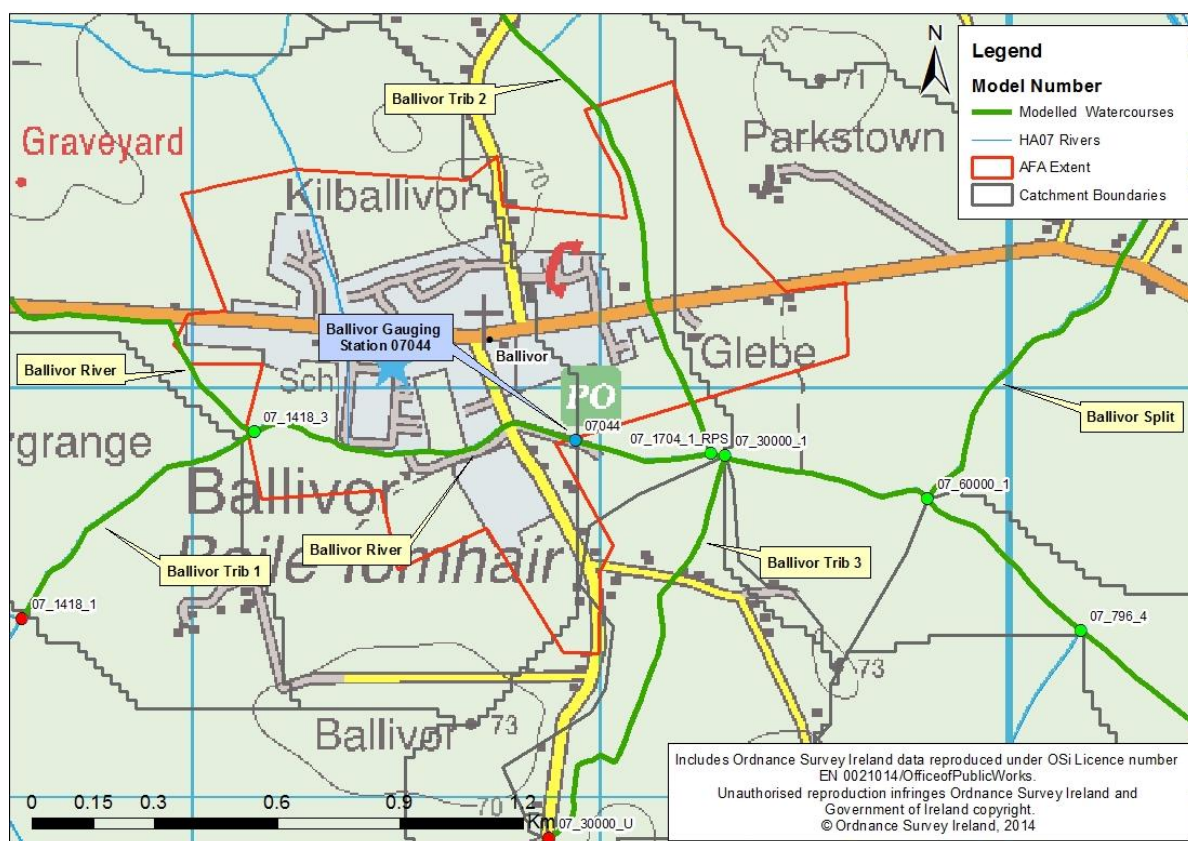


Figure 4.2.2: Ballivor Model AFA Overview

Figure 4.2.3 and Figure 4.2.4 show the location of the cross sections used in the model along with the location of the 2D links and the critical structures.

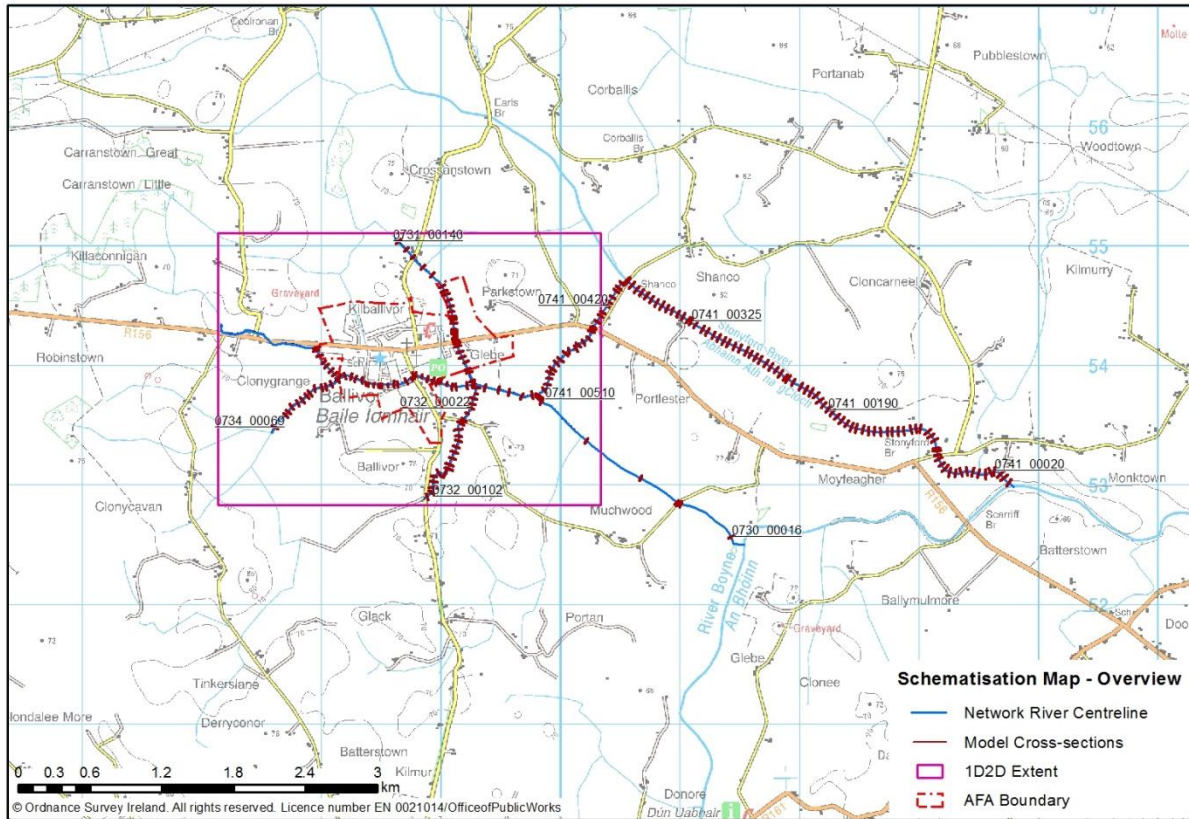


Figure 4.2.3: Overview of model schematisation

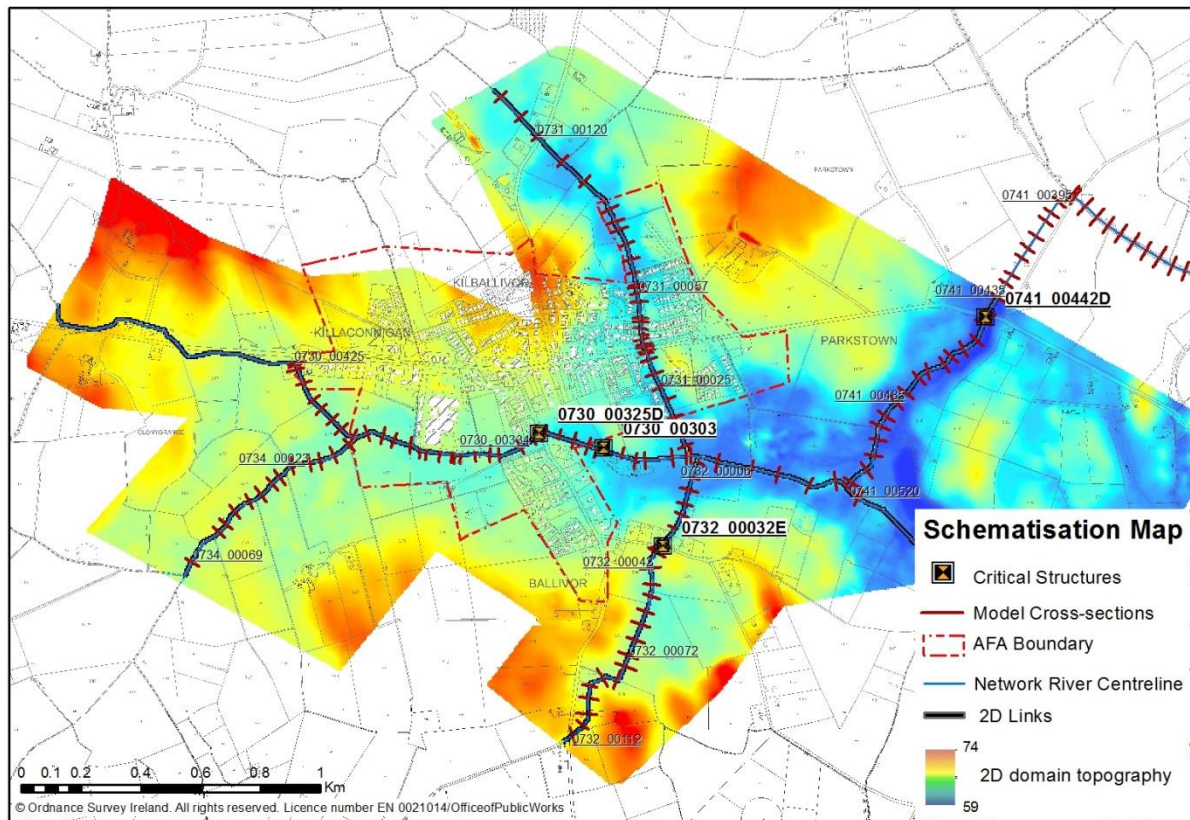


Figure 4.2.4: Model schematisation AFA overview

(2) x-y Coordinates of River (Upstream extent):

River Name	x	y
Ballivor River	267157	254349
Ballivor River Trib 1	267584	253438
Ballivor River Trib 2	268615	255065
Ballivor River Trib 3	268854	252887
Ballivor River Split	269799	253733
Stoneyford River	270543	254721

(3) Total Modelled Watercourse Length:

13.6km

(4) 1D Domain only Watercourse Length:

6.0km

(5) 1D-2D Domain

Watercourse Length:

7.7km

(6) 2D Domain Mesh Type / Resolution / Area:

Rectangular / 5 metres / 10 km²

(7) 2D Domain Model Extent:

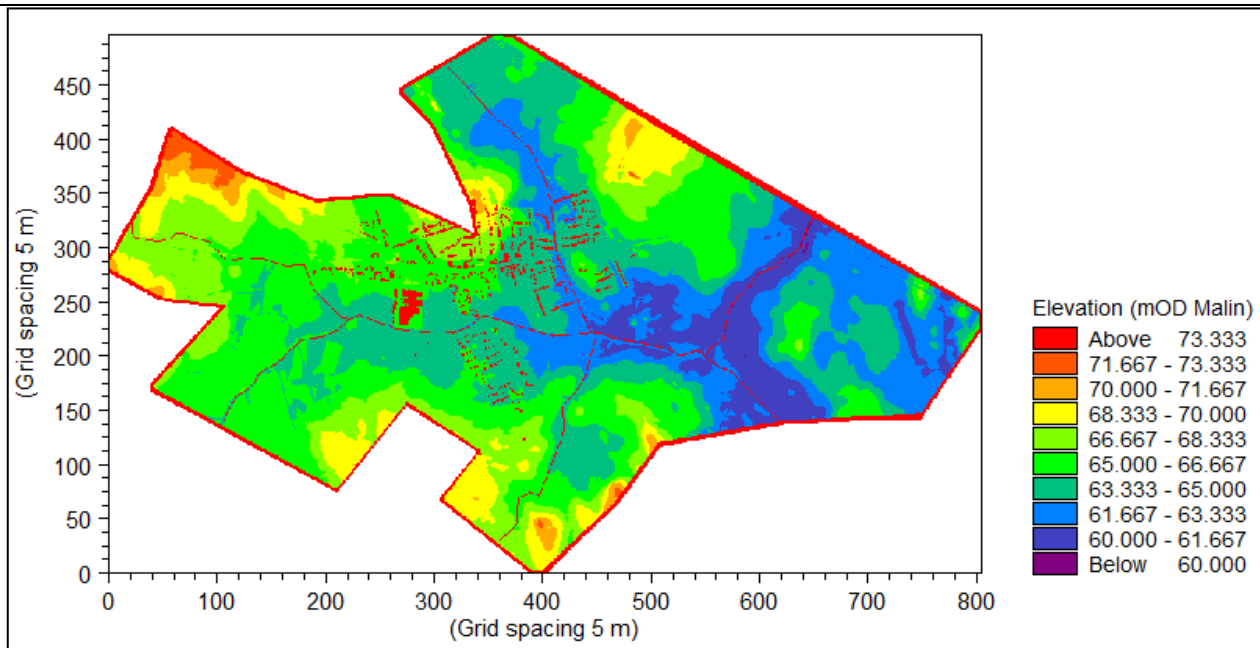


Figure 4.2.5: Ballivor Model 2D Model Domain

Figure 4.2.5 illustrates the bathymetry file showing the modelled extents of the Ballivor AFA and the general topography of the catchment. The 2D domain was generated using LiDAR survey data and created as a 5m grid rectangular mesh. No post processing was required to the bathymetry file.

(8) Survey Information

(a) Survey Folder Structure:

First Level Folder	Second Level Folder	Third Level Folder
Murphy_E07_M02_WP2_0730_120625 Ballivor Murphy - Surveyor Name E07 - Eastern CFRAM Study Area, Hydrometric Area 07 M02 - Model Number 02 WP2 - Work Package 2 120625 - Date issued (25 June 2012) 0730 - River Reference	GIS and Floodplain Photos	Floodplain Photos and Shapefiles
		Structure Register
		Surveyed Cross Section Lines
	Photos and Videos	
	Ascii	
	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	

(b) Survey Folder References:		
Reach ID	Name	File Reference
0730	BALLIVOR RIVER	Murphy_E07_M02_WP2_0730_120625
0734	BALLIVOR RIVER TRIB 1	Murphy_E07_M02_WP2_0734_120625
0731	BALLIVOR RIVER TRIB 2	Murphy_E07_M02_WP2_0731_120625
0732	BALLIVOR RIVER TRIB 3	Murphy_E07_M02_WP2_0732_120625
0741	BALLIVOR SPLIT	Murphy_E07_M02_WP2_0741_120625
0741	STONEFORD RIVER	Murphy_E07_M02_WP2_0741_120625
(9) Survey Issues:		
No survey queries arose.		

4.2.3 Hydraulic Model Construction

All structures within the 1D model that have potential to overtop, such as bridges and culverts, were simulated using an overtopping weir representative of the associated parapet or deck. This allows for flood water to overtop a surcharged structure and avoids creating an artificially high backwater profile. Overtopping weirs were applied to 9 bridges and culverts in the Ballivor model.

(1) 1D Structures (in-channel along modelled watercourses):

See Appendix B

Number of Bridges and Culverts: 12

Number of Weirs: 4



Figure 4.2.6: Photograph of Critical Structure on the Ballivor Trib 3

Figure 4.2.6 shows a critical structure identified on Ballivor Trib 3 at chainage 808m. The culvert is undersized causing a significant head loss through



Figure 4.2.7: Photograph of Critical Structure on the Ballivor River

Figure 4.2.7 shows a critical structure identified on the Ballivor River at chainage 1,916m. This structure is considered significant during the 0.1% AEP event

the structures. The resulting elevated headwater levels causes out of bank flooding during the 0.1% AEP event. In all events the flow is restricted downstream of the culvert.	where the flow is restricted and contributes to out of bank flooding upstream.																																																																																																																
(2) 1D Structures in the 2D domain (beyond the modelled watercourses):	None																																																																																																																
(3) 2D Model structures:	None																																																																																																																
(4) Defences:																																																																																																																	
None																																																																																																																	
(5) Model Boundaries - Inflows:																																																																																																																	
Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report - Section 5.2 and Appendix D). The boundary conditions implemented in the model are shown in Table 4.2.1.																																																																																																																	
The hydrology was reviewed during the rating review at Ballivor Gauging Station (07044) and was found to produce consistent results in the full Ballivor model. There was therefore no change made to the hydrology during model calibration. Please view Section 4.2.5(2) which discusses model updates for Final deliverables.																																																																																																																	
Table 4.2.1: Model Boundary Conditions																																																																																																																	
	<table><tr><th></th><th>Boundary Description</th><th>Boundary Type</th><th>Branch Name</th><th>Chainage</th><th>Chainage</th><th>Gate ID</th><th>Boundary ID</th></tr><tr><td>1</td><td>Open</td><td>Inflow</td><td>BALLIVOR RIVER</td><td>0</td><td>0</td><td></td><td>07_1660_2</td></tr><tr><td>2</td><td>Open</td><td>Inflow</td><td>BALLIVOR TRIB 1O</td><td>0</td><td>0</td><td></td><td>07_1418_3</td></tr><tr><td>3</td><td>Open</td><td>Inflow</td><td>BALLIVOR TRIB 2O</td><td>0</td><td>0</td><td></td><td>07_1704_U</td></tr><tr><td>4</td><td>Open</td><td>Inflow</td><td>BALLIVOR TRIB 3O</td><td>0</td><td>0</td><td></td><td>07_30000_U</td></tr><tr><td>5</td><td>Point Source</td><td>Inflow</td><td>BALLIVOR RIVER</td><td>3491.92</td><td>0</td><td></td><td>07_796_4</td></tr><tr><td>6</td><td>Point Source</td><td>Inflow</td><td>STONEYFORD RIVE</td><td>1280</td><td>0</td><td></td><td>07_1668_1</td></tr><tr><td>7</td><td>Distributed Source</td><td>Inflow</td><td>BALLIVOR RIVER</td><td>0</td><td>2120</td><td></td><td>Top-up flow between 07_1660_2 & 07044</td></tr><tr><td>8</td><td>Distributed Source</td><td>Inflow</td><td>STONEYFORD RIVE</td><td>1280</td><td>5154</td><td></td><td>Top-up flow between 07044 & 07_248_2_RPS</td></tr><tr><td>9</td><td>Distributed Source</td><td>Inflow</td><td>BALLIVOR RIVER</td><td>2988.98</td><td>3538.64</td><td></td><td>Top-up flow between 07_796_4 & 07_60000_1</td></tr><tr><td>10</td><td>Distributed Source</td><td>Inflow</td><td>BALLIVOR TRIB 2O</td><td>0</td><td>1400</td><td></td><td>07_1704_1_RPS</td></tr><tr><td>11</td><td>Distributed Source</td><td>Inflow</td><td>BALLIVOR TRIB 3O</td><td>0</td><td>1109</td><td></td><td>07_30000_1</td></tr><tr><td>12</td><td>Open</td><td>Water Level</td><td>BALLIVOR RIVER</td><td>5164</td><td>0</td><td></td><td>Trim_31688m</td></tr><tr><td>13</td><td>Open</td><td>Water Level</td><td>STONEYFORD RIVE</td><td>5209</td><td>0</td><td></td><td>Trim_34188m</td></tr></table>		Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID	1	Open	Inflow	BALLIVOR RIVER	0	0		07_1660_2	2	Open	Inflow	BALLIVOR TRIB 1O	0	0		07_1418_3	3	Open	Inflow	BALLIVOR TRIB 2O	0	0		07_1704_U	4	Open	Inflow	BALLIVOR TRIB 3O	0	0		07_30000_U	5	Point Source	Inflow	BALLIVOR RIVER	3491.92	0		07_796_4	6	Point Source	Inflow	STONEYFORD RIVE	1280	0		07_1668_1	7	Distributed Source	Inflow	BALLIVOR RIVER	0	2120		Top-up flow between 07_1660_2 & 07044	8	Distributed Source	Inflow	STONEYFORD RIVE	1280	5154		Top-up flow between 07044 & 07_248_2_RPS	9	Distributed Source	Inflow	BALLIVOR RIVER	2988.98	3538.64		Top-up flow between 07_796_4 & 07_60000_1	10	Distributed Source	Inflow	BALLIVOR TRIB 2O	0	1400		07_1704_1_RPS	11	Distributed Source	Inflow	BALLIVOR TRIB 3O	0	1109		07_30000_1	12	Open	Water Level	BALLIVOR RIVER	5164	0		Trim_31688m	13	Open	Water Level	STONEYFORD RIVE	5209	0		Trim_34188m
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Figure 4.2.8 provides an example of the associated upstream hydrograph on the Ballivor River at the 07_1660_2_RPS HEP.

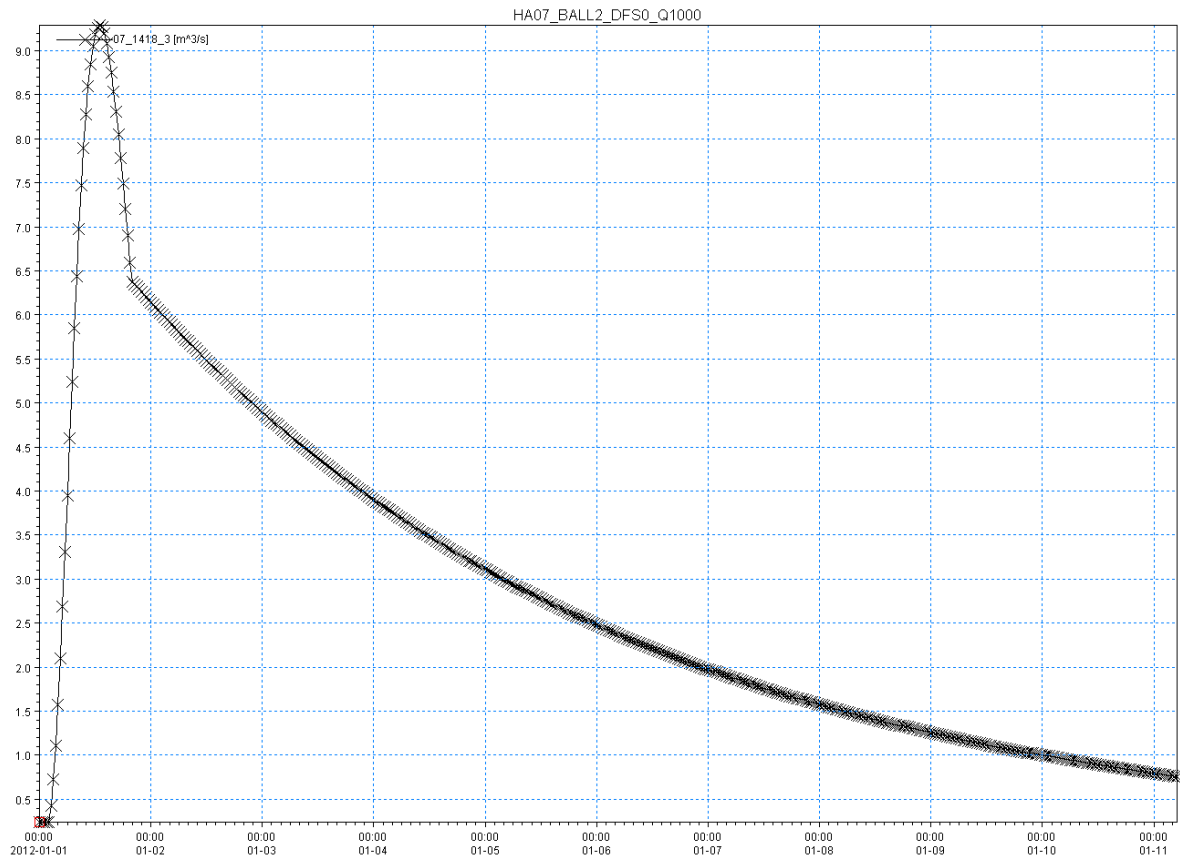


Figure 4.2.8: Inflow Hydrograph for Ballivor River during a 0.1%AEP event

**(6) Model Boundaries –
Downstream Conditions:**

The water levels generated from the downstream Trim model at the River Boyne/Ballivor River confluence and the River Boyne/Stoneyford River confluence were taken as the downstream boundary conditions on the Ballivor River and Stoneyford River respectively. The influence of the River Boyne in creating a backwater effect along the both watercourses was accounted for in this way. Figure 4.2.9 shows the downstream boundary water levels based on the River Boyne 0.1%AEP flood event.

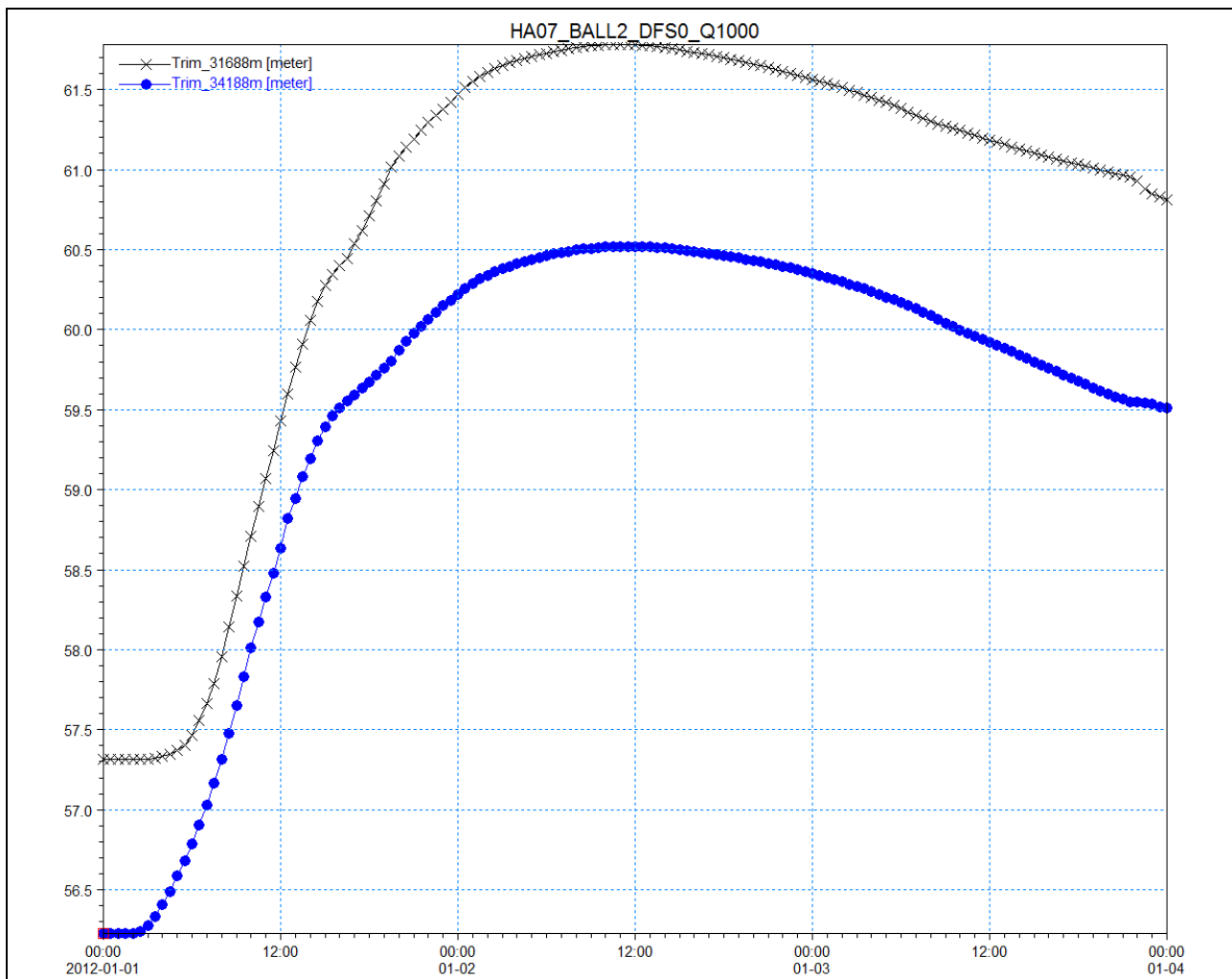


Figure 4.2.9: Downstream boundary water levels based on the River Boyne 0.1%AEP flood event

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.050
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.035
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.035 (Inverse of Manning's 'M')	Maximum 'n' value: 0.045 (Inverse of Manning's 'M')

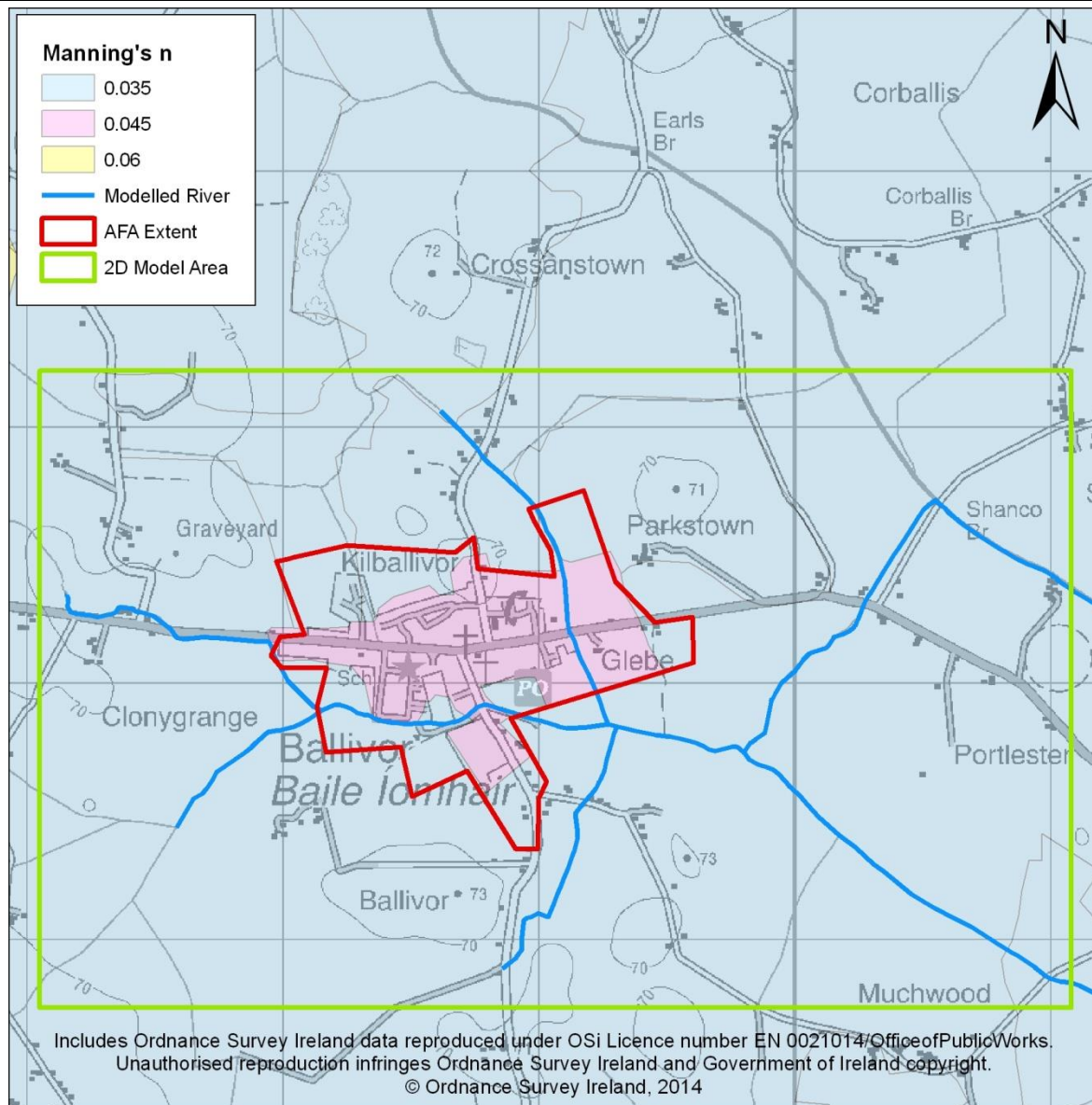


Figure 4.2.10: Map showing floodplain roughness values in 2D domain

This map illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the CORINE Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset. The surrounding rural area to Ballivor is assigned a Manning's n of 0.035. Ballivor is assigned a Manning's n value of 0.045.

(d) Examples of In-Bank Roughness Coefficients

A review of all the channels was carried out in order to estimate the roughness coefficients. The upper reaches of the Ballivor River and its associated tributaries are affected by dense vegetation such as hedgerows growing into the channel. A roughness coefficient was utilised to allow the effects of this to be accounted for.



Figure 4.2.11: Example of roughness coefficients on the Ballivor River at cross section 0730_00302

Manning's n value: 0.050

Straight stream with vegetation and stones



Figure 4.2.12: Example of roughness coefficients on Ballivor River at cross section 0730_00356

Manning's n value: 0.035

Straight stream with some vegetation

4.2.4 Sensitivity Analysis

In accordance with OPW guidance, sensitivity simulations have been conducted with the purpose to assess the sensitivity and impact the 1% AEP fluvial within the AFA boundary. Sensitivity analysis is achieved by the adjustment of various model parameters, several model simulations have been completed, they are as follows:

- a) Channel roughness and flood plain roughness increased to upper bound values - the change in channel and floodplain roughness values resulted in a high increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.2.13. This indicates that the Ballivor model demonstrates a high sensitivity to changes in roughness parameters. The increase in flood extents affects 1 additional AFA receptors, whereas no receptors have been impacted during the 1% AEP design event.

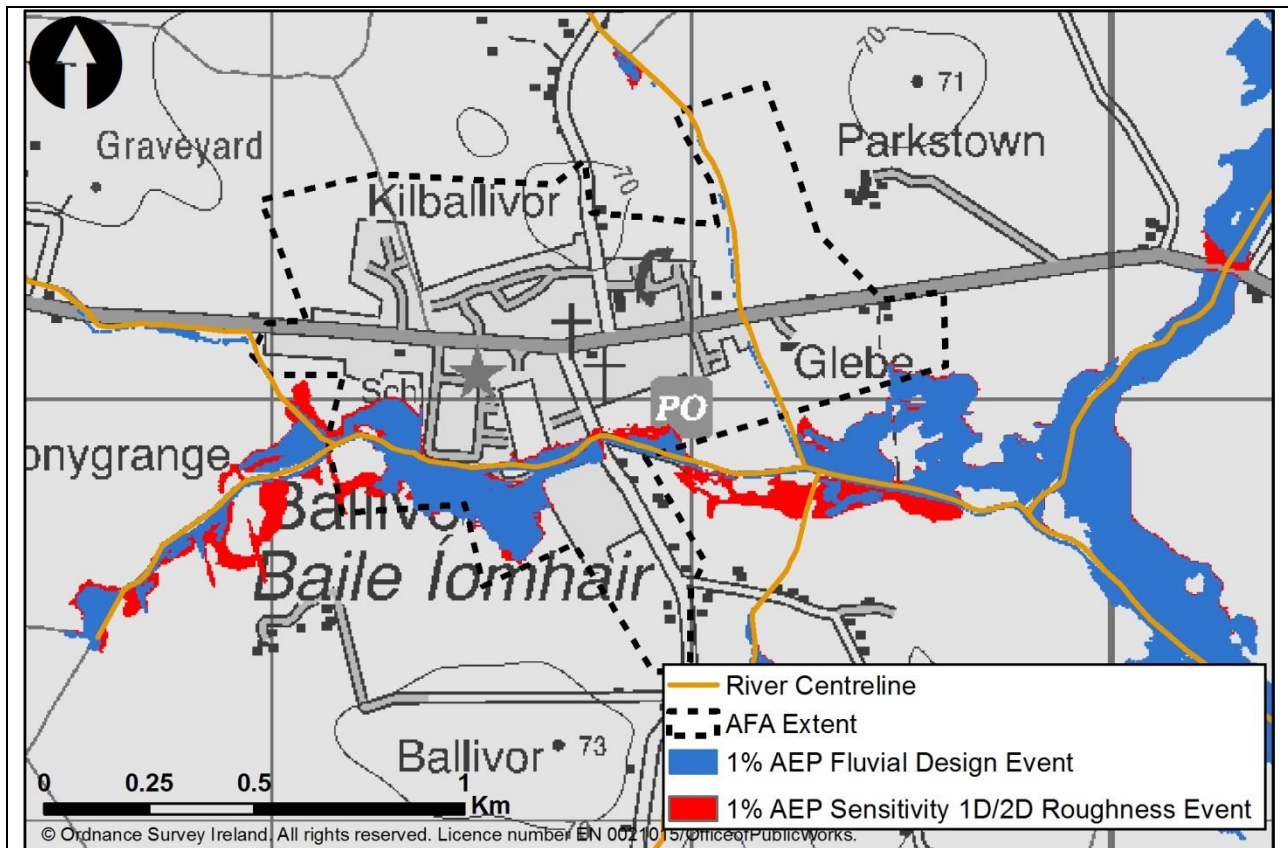


Figure 4.2.13 Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in roughness 1

- b) Downstream boundary increase – The downstream boundaries are increased to the water levels generated from the 1% AEP mid-range future design scenario. Peak water levels are increased to 61.7m OD (Ballivor watercourse) and 60.48m OD (Stoneyford watercourse). Changing the boundary condition increases the peak water levels at the downstream extent of the model. Table 4.2.2 is a comparison between the design and sensitivity water levels located at downstream extent of the modelled Ballivor and Stoneyford watercourses. Changing the downstream boundary condition increased Ballivor River water level from the downstream extent and upstream for 2.3km. Similarly, the water level increased along Stoneyford River from its downstream extent and upstream for 3.8km.

Table 4.2.2: Comparison of Design and Sensitivity Downstream Water Levels

River Name (Chainage)	Design (water-level ODM)	Sensitivity (water-level mOD)	Difference (m)
Ballivor River 0730 (7831.07)	61.25	61.70	0.455
Stoneyford River 0741 (5209)	59.83	56.68	0.65

As shown in Figure 4.2.14, changing the downstream boundary condition has a negligible influence on the flood extents of the Ballivor model. The Ballivor model therefore has a low sensitivity to downstream boundary increase with no impact to receptors within the AFA.

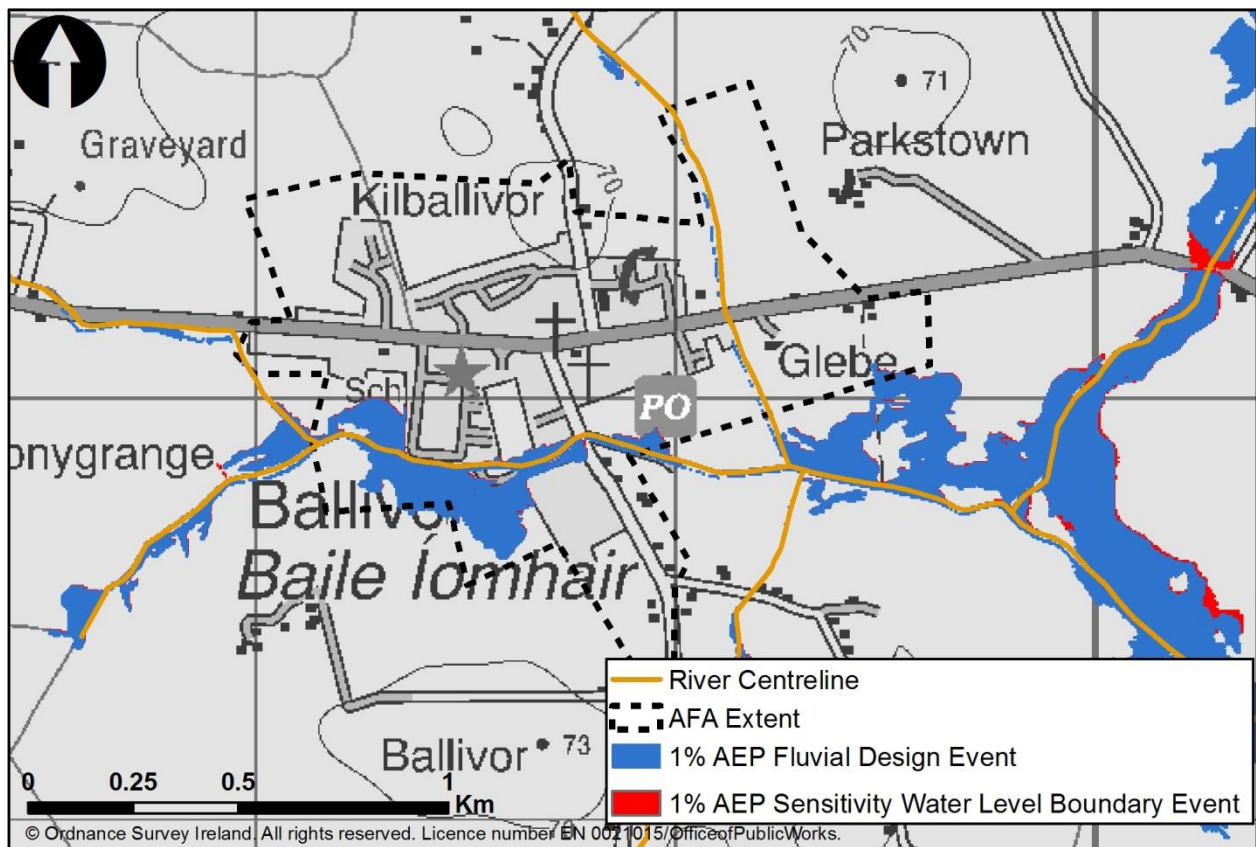


Figure 4.2.14: Comparison between 1% AEP Design Event & 1% AEP Sensitivity Water Level Boundary Event

- c) Increase in flow –The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The simulated flows for the Ballivor model have been assessed as having medium uncertainty/sensitivity (refer to the Hydrology Report IBE0600Rp0012 *for further detail*); subsequently a factor of 1.57 is applied to design flows for the sensitivity simulation. Figure 4.2.15 shows that the Ballivor model has a high sensitivity to increased inflow parameters; this is reflected by the high increase of flood extent. This increase impacts the area where the Kinnegad Road crosses over the Ballivor River. This change impacts 1 additional AFA receptors compared to no impact to properties resulting from the 1% AEP design event.

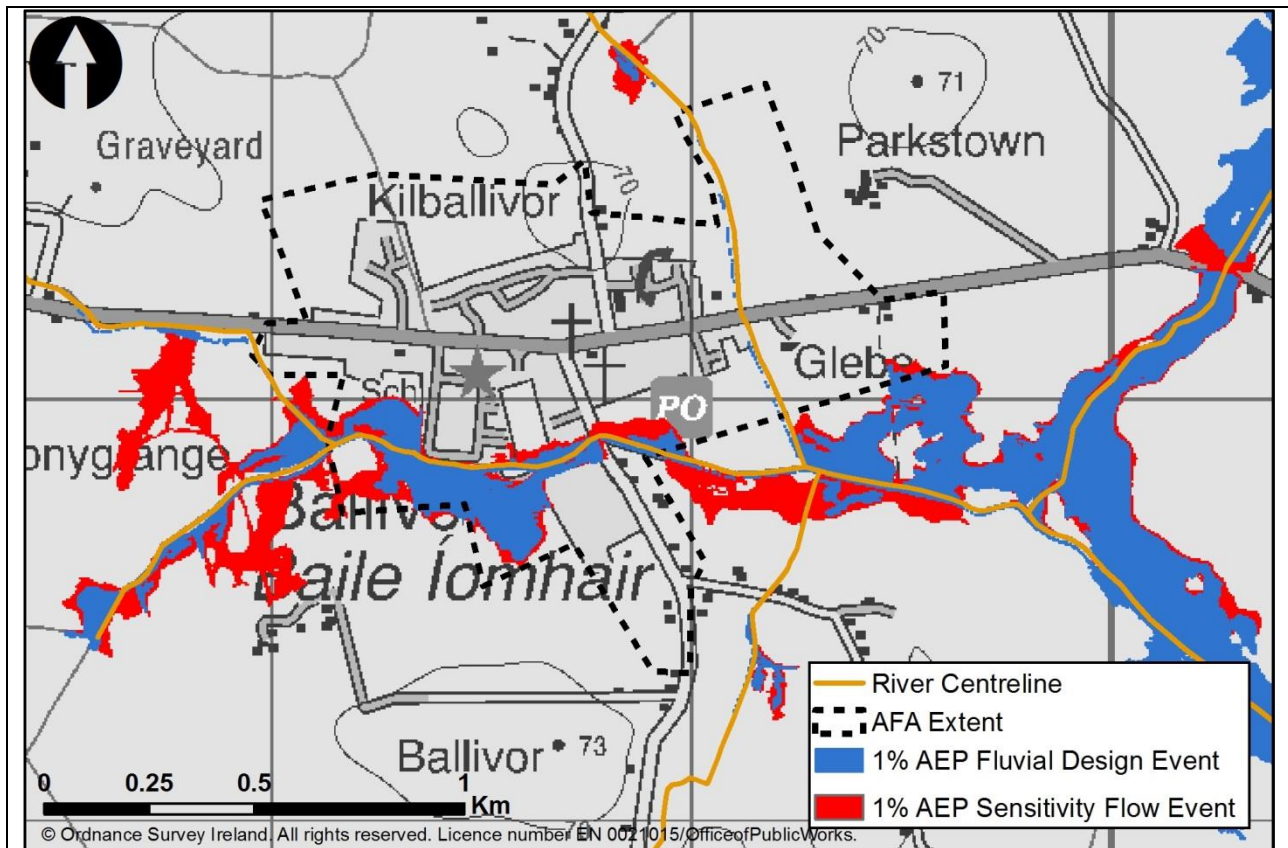
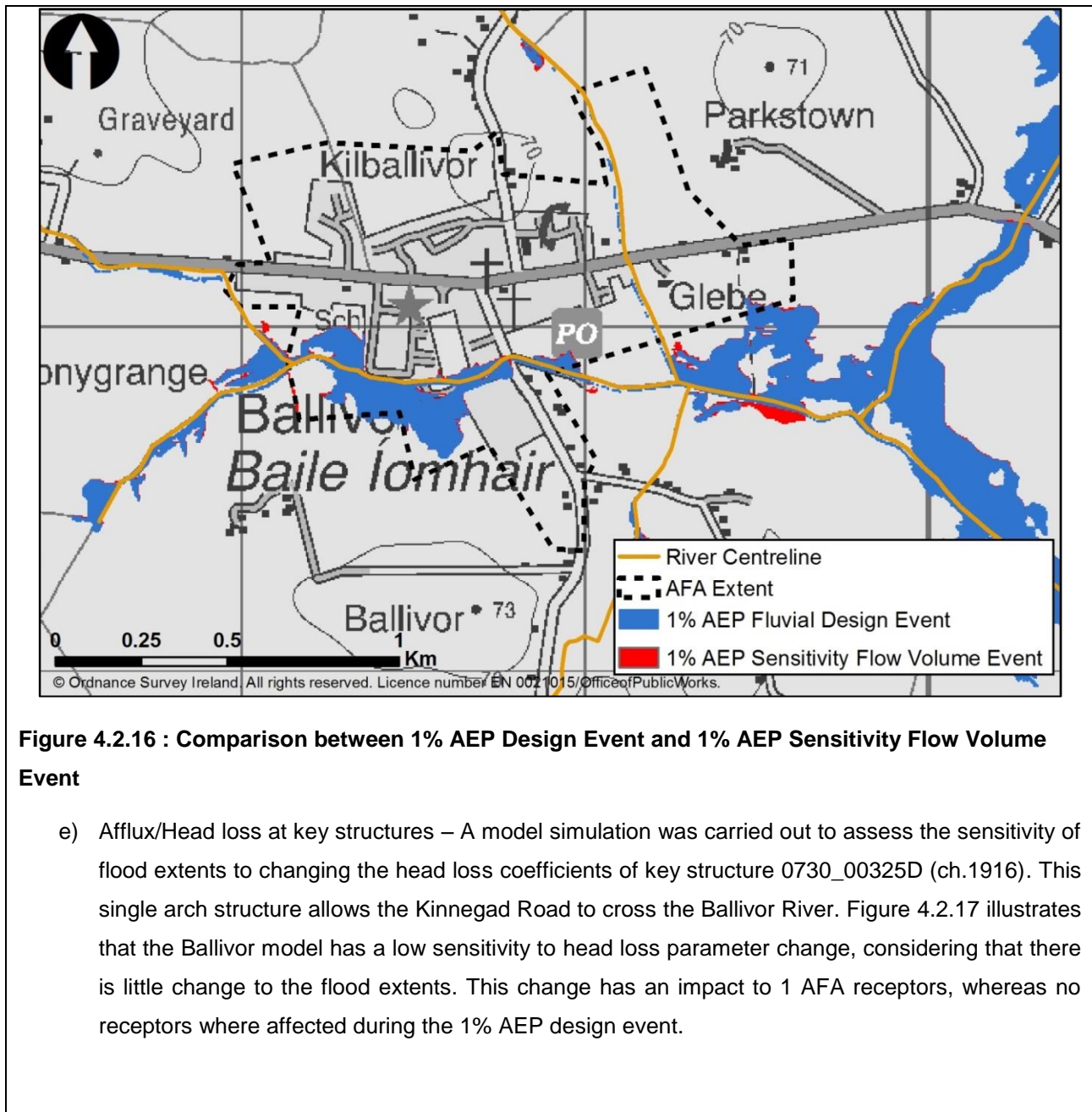


Figure 4.2.15 Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in flow 2

- d) Flow volume – A sensitivity adjustment factor of 2 has been applied to flood duration to assess the effect on the model as flood durations in this case have been derived from observed data with some uncertainty at flood flows. Figure 4.2.16 shows that the Ballivor model indicates a low sensitivity to flow volume parameters which results in a low increase to flood extents. This change has an impact to 1 AFA receptors, whereas no receptors were affected during the 1% AEP design event.



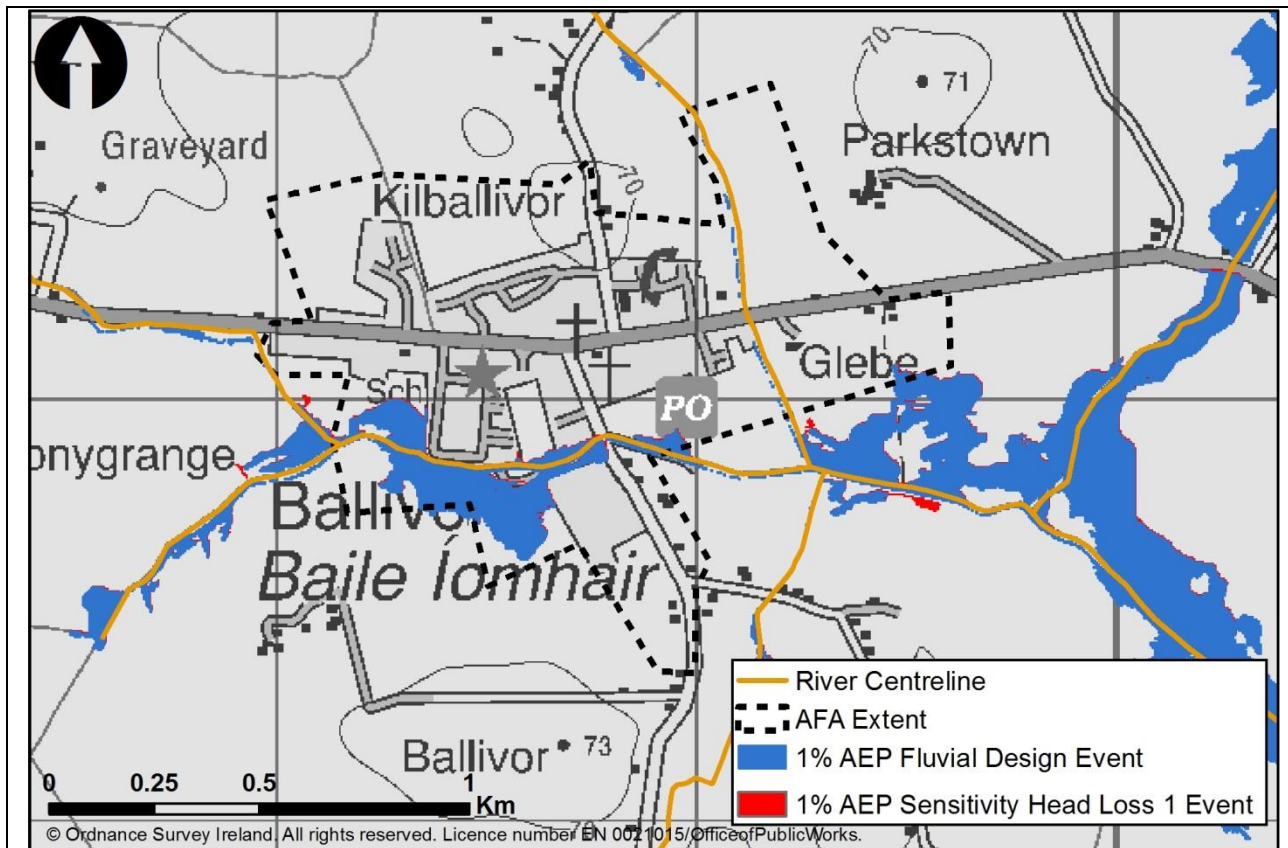


Figure 4.2.17: Comparison between 1% AEP Design Event and 1% AEP Sensitivity (Head Loss 1)

- f) Floodplain roughness decreased to lower bound values – The change in floodplain roughness values resulted in no alteration to flood extents within the AFA as shown Figure 4.2.18. The Ballivor model is considered to have a low sensitivity to 2D roughness parameters and no receptors are impacted within the AFA.

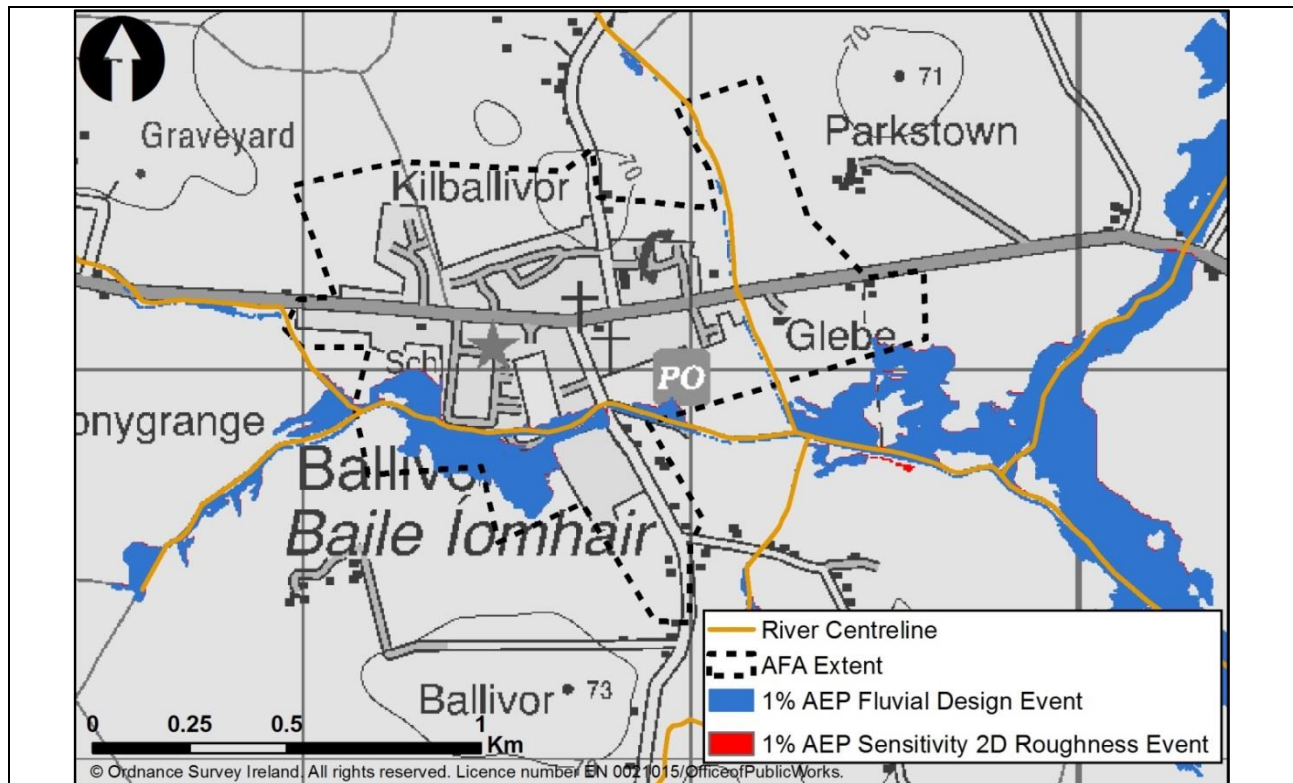


Figure 4.2.18 : Comparison between 1% AEP Design Event and 1% AEP Sensitivity 2D Roughness Event The model also indicates a high sensitivity to changing 1D/2D roughness parameters resulting in a high impact to receptors. The model is less sensitive to the other parameters, as indicated by a minor increase of the flood extent. The 1% AEP fluvial design event did not impact any receptors located within the AFA. In comparison, sensitivity impact is considered as relatively high.

Table 4.2.2 summarises the outcomes of the above sensitivity simulations or evaluations that were considered for the Ballivor model. Of these parameters, the model is most sensitive to increase in model inflows. Table 8.1 in the Hydrology Report states that there is medium uncertainty/sensitivity associated with the hydrological inputs for this model. The model also indicates a high sensitivity to changing 1D/2D roughness parameters resulting in a high impact to receptors. The model is less sensitive to the other parameters, as indicated by a minor increase of the flood extent. The 1% AEP fluvial design event did not impact any receptors located within the AFA. In comparison, sensitivity impact is considered as relatively high.

Table 4.2.2: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	High	High
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	High	High
1% AEP Sensitivity Flow Volume Event	Low	High
1% AEP Sensitivity Head Loss 1 Event	Low	High
1% AEP Sensitivity 2D Roughness Event	Low	-

4.2.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0600Rp0004_HA07 Inception Report, unless otherwise specified).

<p>(a) Aug 2008</p>	<p>Flooding occurred in Navan, Trim, Ballivor, Athboy and Edenderry in August 2008. In the Ballivor area, the Clonycavan channel overflowed resulting in adjacent lands becoming flooded. Photographs taken in the area did not show any houses within the flooded lands.</p> <p>The Clonycavan area is located upstream of the Ballivor AFA and as such was not modelled. The flood event is therefore of limited use in verifying the modelled flood extents.</p>
<p>(b) Feb 1995</p>	<p>Heavy rainfall in February of 1995 led to flooding in Trim and Ballivor. In Ballivor, land was flooded in the Parkstown and Coolronan areas. A mean daily flow for this event of over $1.5\text{m}^3/\text{s}$ was recorded at the Ballivor Hydrometric Station (07044) compared to an average flow from all available records of $0.176\text{m}^3/\text{s}$.</p> <p>The Coolronan area is located beyond the limits of the Ballivor model and flooding in this area cannot be confirmed. The Parkstown area shows flooding in the 1% and 0.1% AEP event as shown in Figure 4.2.19. No estimated size of flood event could be given for any flooding within Ballivor as no long annual max data was available. The model can however verify that the Parkstown area is susceptible to flooding during extreme flood events.</p> <div data-bbox="432 1214 1450 1904"> </div>

	<p>rainfall. No flow data is available for the river in Ballivor. However gauge data for this date is available and this indicates that the water level reached its highest level (at 1.1m on the staff gauge) in 22 years of available records.</p> <p>The staff gauge zero datum at this gauging station is 61.32mOD making the highest recorded level during the 1993 flood 62.42mOD. The model estimates the peak water level during the 1% AEP and 0.1% AEP events to be 62.36mOD and 62.79mOD respectively. This suggests that the flood in 1993 was somewhere between a 1% and 0.1% AEP event. No information on flooding locations was provided apart from stating that McLaughlin's bar was flooded. The model results show that no flooding from any event comes close to McLaughlin's Bar which is located on main street and not in proximity of the Ballivor River. The fact that McLaughlin's Bar alone is reported as being flooded, and not the buildings adjacent or opposite it suggests that flooding to the bar originated from pluvial sources.</p>
<p>Summary of Calibration</p> <p>The lack of historical flooding information at Ballivor gives little quantitative data to calibrate or even verify the model to the larger flood events. The model was calibrated to the available spot gaugings at Ballivor gauging station and the roughness values of the model adjusted accordingly. Limited verification was achieved by the areas which are shown to flood and have historically flooded.</p> <p>A comparison between estimated and modelled flows at HEP points was carried out, the details of which are presented in Section 4.2.5 (5).</p> <p>A mass balance analysis was carried out to assess the difference between the discharge into the model and the volume of water stored with the discharge out of the model. Results showed a difference of 0.19%. This is within the acceptable limits as stated in the Environment Agency's Fluvial Design Guide.</p> <p>Whilst anecdotal information and available data has been used to the best extent possible, overall there is little or poor data to calibrate the model to and observation of more events would be necessary to reduce the uncertainty in model results.</p>	
<p>(2) Post Public Consultation Updates:</p> <p>Following informal public consultation and formal S.I. public consultation periods in 2015, analysis of the results of the rating reviews and their impact on the hydrological analysis for the UoM07 watercourses was finalised. The updated inflow hydrograph at HEP 07_1418_3 is shown in Figure 4.2.20. In conjunction with these updates the downstream boundary and cross sections in the 1D zone were refined. This resulted in the hydrology and mapping outputs being updated, however there were no changes made to hydraulics. A revised set of flood hazard and risk mapping has been issued as Final to reflect this change.</p>	

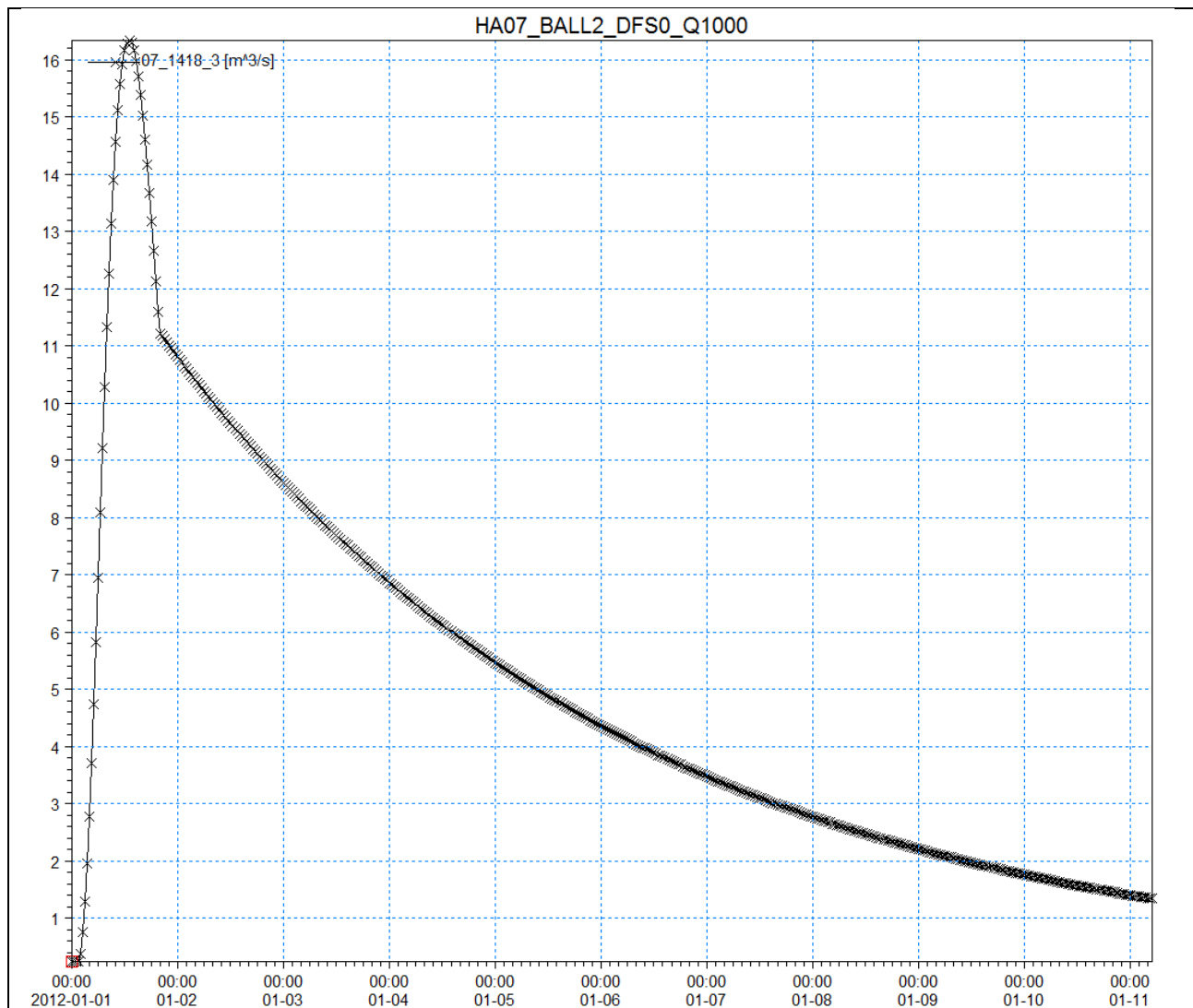


Figure 4.2.20: Inflow Hydrograph for Ballivor River during a 0.1% AEP event

(3) Standard of Protection of Existing Formal Defences:

None

(4) Gauging Stations:

There is one gauging station within the model extent

Ballivor (07044)

A gauging station is located at the downstream extent of Ballivor Town on the Ballivor River. A rating review was carried out in order to support calibration of the model. Continuous records are available for the gauging station from 1989 however the weir which was installed at the gauging station was not made water tight until 1994. The last EPA rating review which was assigned a rating standard of "fair" was carried out in 1994. This review limited the rating curve to a stage of 0.41m which equates to a flow of 1.15 m³/s. The predicted Q_{med} for the Ballivor River at the gauging station is 3.27 m³/s. Therefore, the

1.15m³/s. The predicted Q_{med} for the Ballivor River at the gauging station is 3.27 m³/s. Therefore, the existing EPA rating curve was used to calibrate the model up to 0.35 times the Q_{med} only.

A good correlation was found between the recorded spot gaugings and the model output for the lower flows. However, given the lack of high flow data there is a reduced confidence in the estimated Q_{med} . RPS recommends that additional high flows be recorded and when suitable data has been processed a revised rating curve can be produced. For details on the current rating review please refer to IBE0600Rp0012_HA07 Hydrology Report.

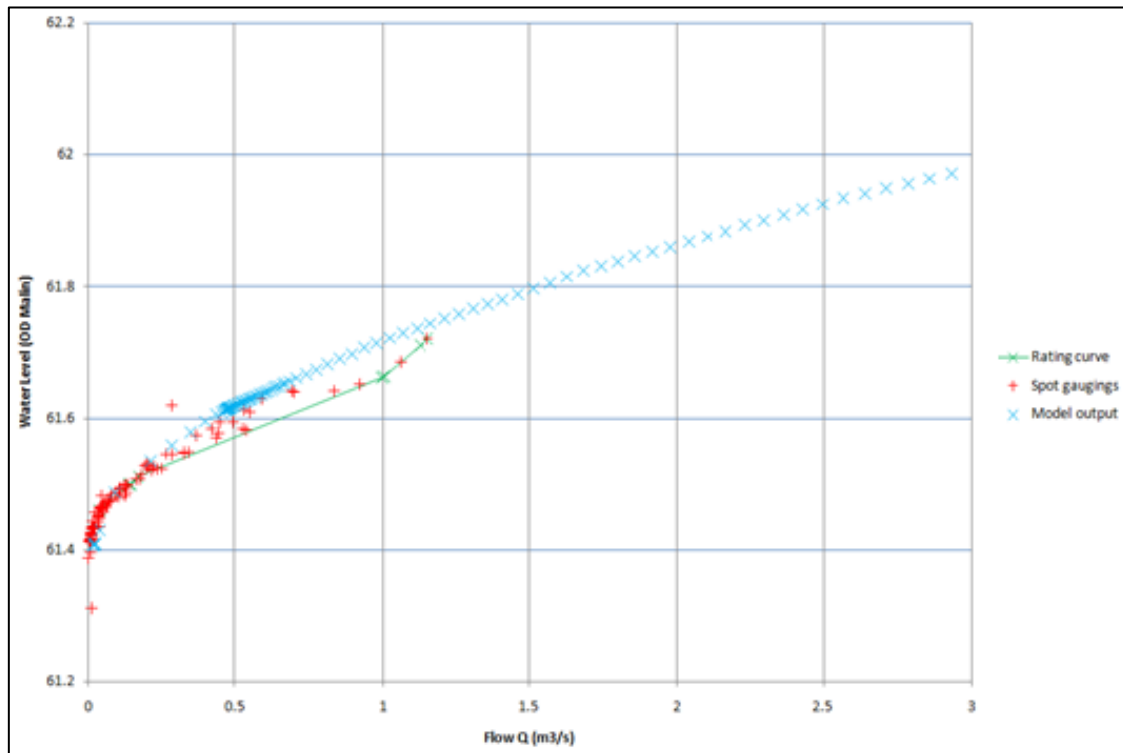


Figure 4.2.21: Qh relationship comparison between Ballivor model and EPA rating curve at Ballivor Gauging Station

(5) Validation with MIKE NAM:

It can be seen from Table 4.2.3 that comparison between the NAM model and observed data cannot be made for the event that do not have concurrent data. The 1995 flood event does show a discrepancy between the observed peak flow and the model simulated peak flow. This is due to uncertainty in the observed peak flow which is beyond the reliable limit of the rating (1.116m³/s). As the NAM simulated peak has been formulated from the ratings with its associated uncertainty there is also a discrepancy seen in the estimated water levels from peak NAM discharge.

Table 4.2.3: Comparison of Observed, Hydrologically (NAM) and Hydraulically Modelled Data for Flood Events

	Observed Peak		MIKE FLOOD Simulated Peak at Observed WL	MIKE NAM Simulated Peak	MIKE FLOOD Simulated Peak at NAM Discharge	Water Level Difference
Flood Event	Water Level (mOD)	Discharge (m ³ /s)	Discharge (m ³ /s)	Discharge (m ³ /s)	Water Level (mOD)	(m)
18/08/2008	-	-	-	2.488	61.925	-
01/02/1995	62.054	1.978	3.467	2.539	61.935	0.119
12/06/1993	-	-	-	3.79	62.089	-

(6) Comparison of Flows:

Table 4.2.4 provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

Table 4.2.4: Modelled Flows and Check Flows

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m ³ /s)	Model Flow (m ³ /s)	Diff (%)
BALLIVOR TRIB 1OF3 737.612	10%	4.82	4.82	0.00
07_1418_3	1%	9.02	8.91	-1.22
	0.1%	16.34	16.86	3.19
BALLIVOR TRIB 2OF3 1362.94	10%	0.74	0.67	-9.46
07_1704_1_RPS	1%	1.28	1.01	-21.09
	0.1%	2.16	1.63	-24.54
BALLIVOR TRIB 3OF3 1088.12	10%	0.34	0.26	-23.53
07_30000_1	1%	0.63	0.50	-20.63
	0.1%	1.14	0.92	-19.30
BALLIVOR RIVER 46.0575	10%	1.36	1.38	1.47
07_1660_2	1%	2.55	2.57	0.78
	0.1%	4.61	4.67	1.30
BALLIVOR RIVER 2101.72	10%	6.49	6.28	-3.24

07044	1%	11.81	10.98	-7.03
	0.1%	20.77	21.05	1.37
STONEYFORD RIVER 5181.5	10%	29.24	29.13	-0.38
07_248_2_RPS	1%	41.83	44.62	6.67
	0.1%	57.71	70.55	22.25

The table above provides details of flow in the model at every HEP inflow, check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

At the upstream extent of the Ballivor River the modelled flows correlate well with the check flows (HEP 07_1660_2). As the Ballivor River flows through the Ballivor AFA three tributaries join it. The modelled flows at the downstream reach of the three tributaries were compared with the check flows at HEPs 07_1418_3, 07_1704_1_RPS & 07_30000_1). Ballivor Trib 1 showed a good correlation between the modelled and check flows however Ballivor Tribs 2 and 3 showed a difference up to 24.5%. The influence of the flow from the main Ballivor River channel was assessed. The same models for the 0.1% and 1% AEP events were run with a nominal flow in the Ballivor River so as not to affect the downstream reaches of the tributaries. The results show that the percentage difference between the modelled and check flows reduce to less than 10% therefore demonstrating that the downstream flow in the tributaries are affected by a backwater effect from the Ballivor River. This also shows that the modelled flows are well anchored to the check flows at the downstream reach of the tributaries.

A comparison of the modelled and check flows at HEP 07044, the Ballivor gauging station, showed a close correlation with a percentage difference of 3.24%, 7.03% and 1.37% for the 10%, 1% and 0.1% AEP events respectively.

At the downstream reach of the model and HEP 07_248_2_RPS, where the Stoneyford River enters the River Boyne, a percentage difference 0.38%, 6.67% and 22.25% is recorded between the modelled and check flows for the 10%, 1% and 0.1% AEP events respectively. This is a relatively large difference for the 0.1% AEP event which can be accounted for in the following way. A model was run to assess the effect of the River Boyne river model. It was found that the River Boyne was having an impact in reducing the flow in the Stoneyford River. And when the influence of the River Boyne was not included the percentage difference for the 0.1% AEP event was reduced to 16.5%. While this reduces the percentage difference, it is still relatively large. A review of the hydrology was therefore carried out. The flow estimations had to account for the Ballivor River splitting, with a channel carrying on to the River Boyne and the other to the Stoneyford River. The assumption was made that the majority of the flow is directed to the Stoneyford River so that $57.7\text{m}^3/\text{s}$ is reached at the downstream reach of the Stoneyford River compared the $0.13\text{m}^3/\text{s}$ at the downstream reach of the Ballivor River. The hydraulic model however was allowed to determine the conveyance of flow along both channels in question and showed that a peak flow of $5.78\text{m}^3/\text{s}$ is reached at the downstream extent of the Ballivor River. To accurately compare the modelled flow with the check flow the combined flows at the downstream extent of both the Ballivor River and the Stoneyford River is required. The combined modelled flow being $54.0\text{m}^3/\text{s}$ compared to a

combined check flow of $57.8 \text{ m}^3/\text{s}$ giving a percentage difference of 6.5%. The difference is deemed acceptable given the uncertainty of flow estimation in an ungauged catchment and the model is considered well anchored at the downstream extent.

(7) Other Information:

The watercourses in the Ballivor model are part of the arterial drainage scheme. As such they were assessed and modified to provide improved drainage to land. Part of this scheme involved delineating the benefiting land. This involved a walkover survey by land valuers to identify flood prone and marshy land that would benefit by the improved drainage. An indication can therefore be obtained from the ADS benefiting lands as to the extent of a river's floodplain and therefore be used as verification on the Ballivor model. A review was carried out for the Ballivor River, its tributaries and the Stoneyford River.

The upstream extent of the Ballivor model containing the upper part of the Ballivor River and Trib 1 shows a significantly smaller flood extent than the ADS benefiting lands as shown in Figure 4.2.22. This could be partly due to the presence on numerous smaller streams which have not been modelled but would have been included in the ADS and the resulting benefitting lands mapped. The differences between the ADS benefiting lands and the river's floodplain are more pronounced in this location as can be seen where raised developed land benefits from the ADS but is quite definitely outside the river's floodplain, for example see the highlighted area in Figure 4.2.22.

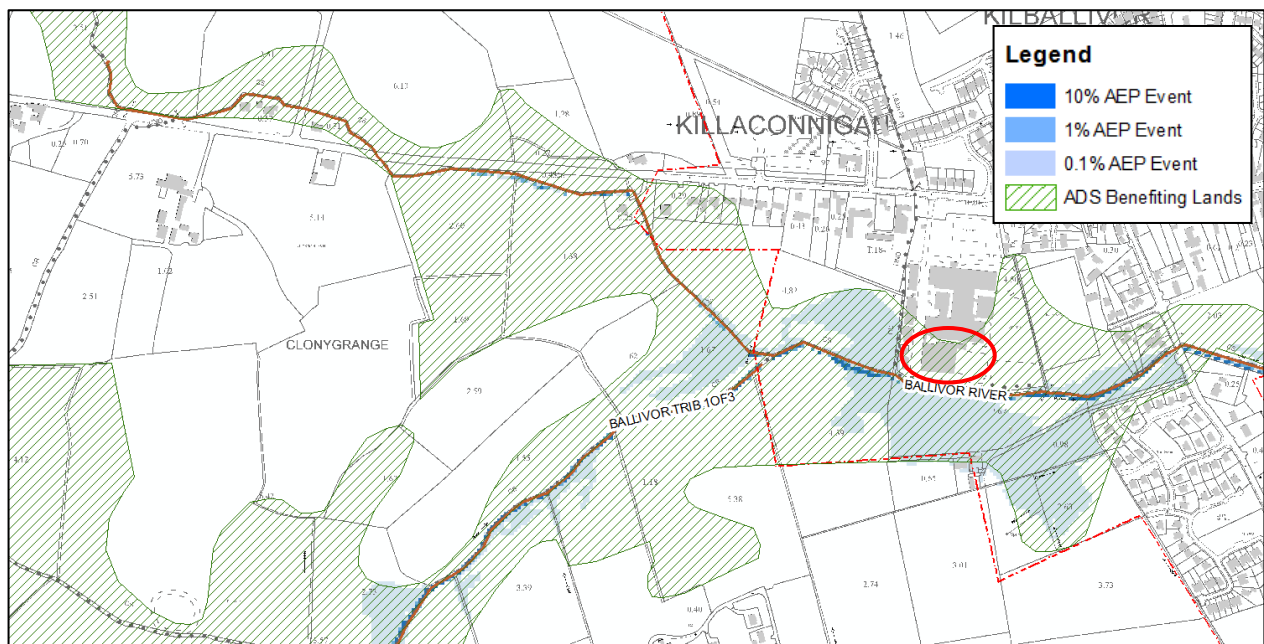


Figure 4.2.22: Comparison between ADS benefitting lands and modelled flood extents at the upper reach of the Ballivor model

In contrast, the downstream extent of the Ballivor AFA shows good correlation with the ADS. It is noted that there are less un-modelled streams in this area therefore reducing the opportunities for the ADS and modelled flood extents to differ.

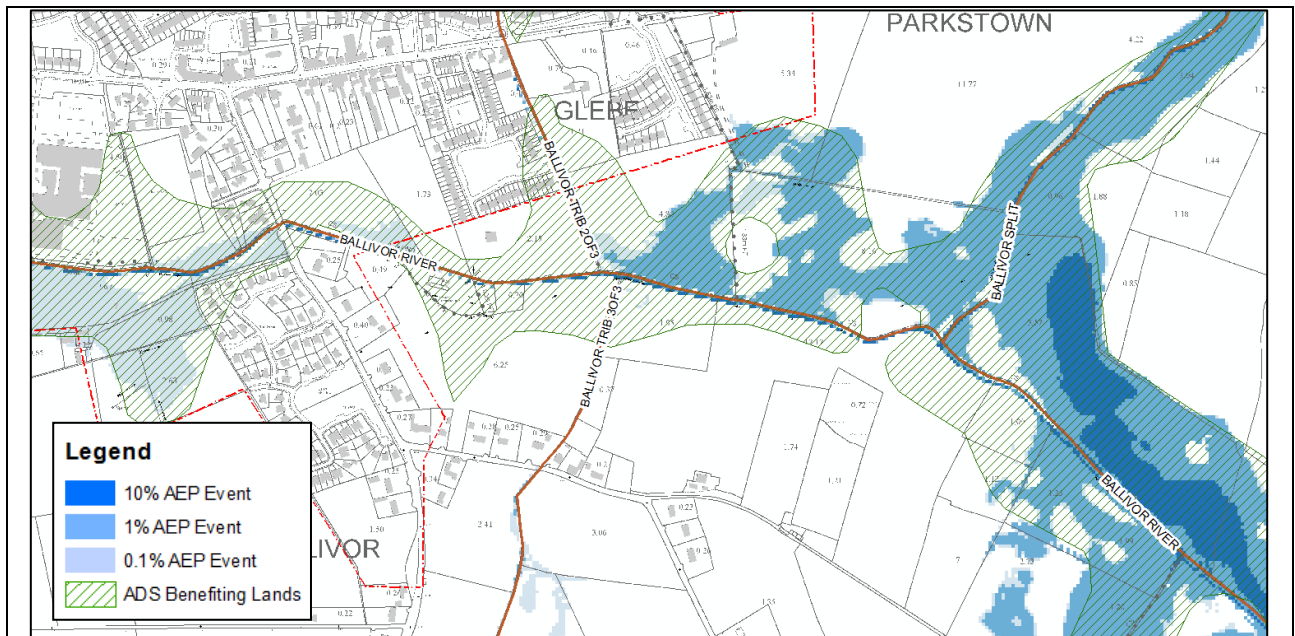


Figure 4.2.23: Comparison between ADS benefiting lands and modelled flood extents at the lower reach of the Ballivor AFA

As the model moves into the MPWs the ADS benefiting lands and the modelled flood extents correlate well as shown in Figure 4.2.24. This gives confidence in the flows being distributed along the lower reach of the Ballivor River, the Ballivor Split and the Stoneyford River.

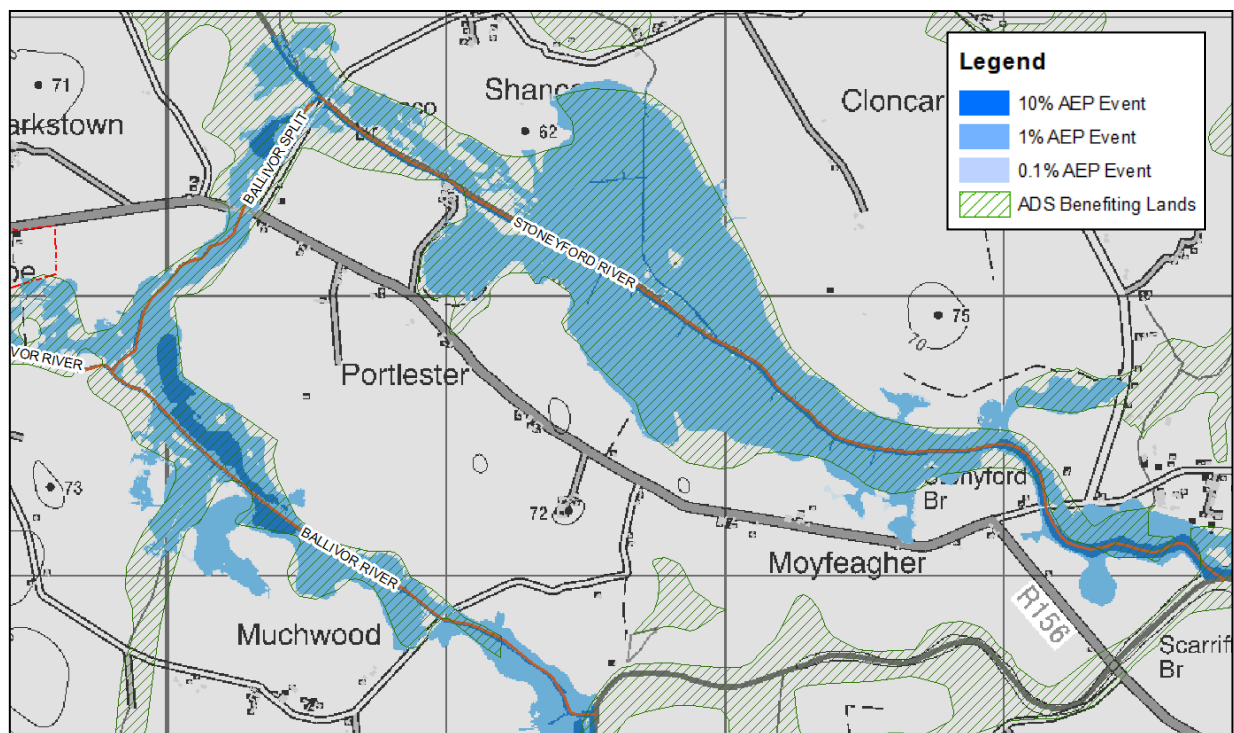


Figure 4.2.24: Comparison between ADS benefiting lands and modelled flood extents at the lower reach of the Ballivor model

4.2.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:	
<p>(a) As the Ballivor River progresses downstream of Ballivor it bifurcates at chainage 2989m. One channel proceeds to the River Boyne and the other to the Stoneyford River. There was no clear dominating channel either from maps, photographs or surveys. The model was therefore constructed so that the channel continuing to the River Boyne remained part of the main Ballivor River branch. The channel connecting to the Stoneyford River was made a separate branch called the Ballivor Split. The hydraulic model was allowed to determine the flow distribution between the two channels for each flood event.</p>	
(2) Hydraulic Model Limitations and Parameters:	
<p>(a) A 2 second time-step for both the MIKE 11 and MIKE 21 models has been selected in order to achieve a successful model simulation for all return periods.</p> <p>(b) The cross sections on the MPW (Ch858m – Ch5,209m Stoneyford River & Ch4,096m – Ch5164m Ballivor River) were extended beyond the limit of the topographical survey using the NDHM. A review was carried out post cross section extension to compare the topographical survey level with the NDHM. A good agreement was found between the two survey datasets and no adjustments were required to the extended cross sections.</p>	
Hydraulic Model Parameters:	
MIKE 11	
Timestep (seconds)	2
Wave Approximation	High Order Fully Dynamic
Delta	0.85
MIKE 21	
Timestep (seconds)	2
Drying / Flooding depths (metres)	0.02 / 0.03
Eddy Viscosity (and type)	0.25 (Velocity Based)
MIKE FLOOD	
Link Exponential Smoothing Factor (where non-default value used)	N/A
Lateral Length Depth Tolerance (m) (where non-default value used)	N/A
(3) Design Event Runs & Hydraulic Model Handover Notes:	
<p>The Ballivor River is part of the arterial drainage scheme and as such has relatively deep channels. Developments adjacent to the river are elevated relative to the channel bed. It therefore takes a large</p>	

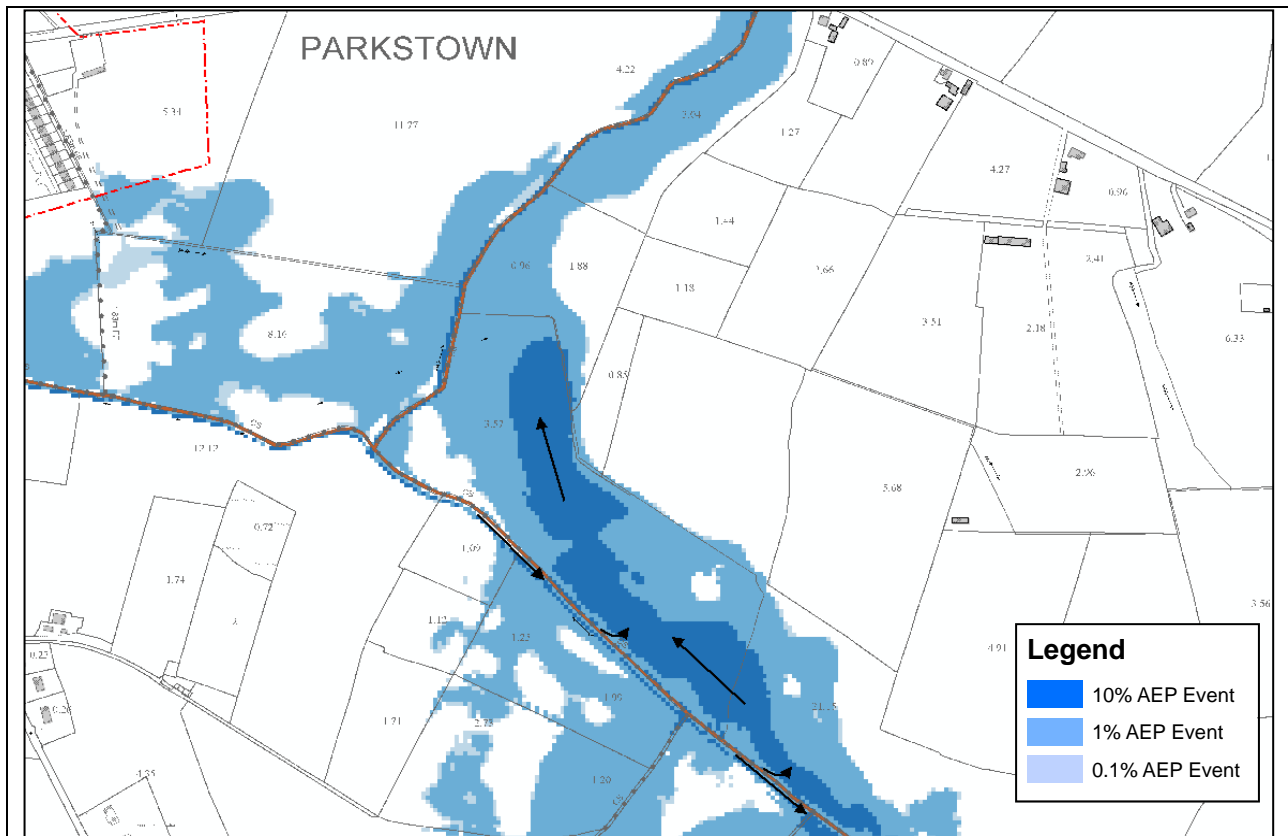


Figure 4.2.26: Flood mechanism in the area of the Ballivor River bifurcation

No properties are shown to be at flood risk from any of the modelled watercourse during any flood event. While there is significant flooding to the agricultural fields around Ballivor during the 0.1% AEP flood event no developed area is shown at risk. This is due to the elevated nature of developed land. Figure 4.2.27 shows the topography around Ballivor and it can be seen how the residential estates and commercial properties are raised in relation to the surrounding floodplain.

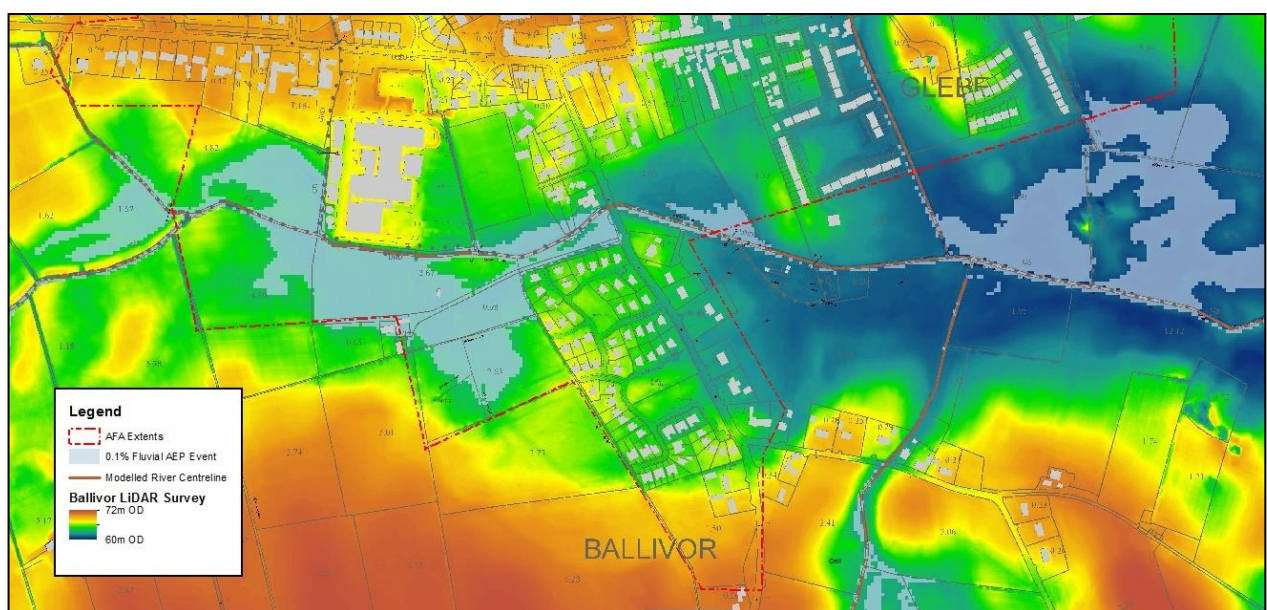


Figure 4.2.27: Topography of rural and developed areas in relation to the 0.1% AEP flood extent

(4) Hydraulic Model Deliverables:

Please see Appendix B for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Mark Wilson
Model Reviewed by:	Stephen Patterson
Model Approved by:	Malcolm Brian

4 HYDRAULIC MODEL DETAILS

4.3 DROGHEDA AND BALTRAY

Name	Local Authority	Unique ID	SSA	Status	Date
Drogheda	Louth / Meath	70033	AFA	Final	08/05/2017

4.3.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600Rp0025_Flood Risk Review) highlighted the Drogheda and Baltray areas of the Boyne catchment as AFAs for fluvial and coastal flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

The Flood Risk Review indicated that there are a number of areas which suffer from recurring flooding in the Drogheda AFA from the River Boyne itself as well as a number of tributaries which flow through the urban and suburban locales.

Baltray was taken forward as an AFA due to the flood risk emanating from coastal water levels within the Boyne Estuary. Furthermore a significant flood risk emanating from fluvial flooding in the Boyne and also from the tributary which flows through the AFA known as the Baltray Stream was identified within the Flood Risk Review.

The HA07 Inception Report (IBE0600Rp0004_HA07 Inception Report) provides specific details of the recurring flood issues which will also be discussed later in this section.

The model of the lower Boyne system incorporates the AFAs of Drogheda and Baltray and also Mornington AFA (reported in Section 4.7 of the HA07 Hydrology Report (IBE0600Rp0025_HA07_Hydrology Report)). This model extends from downstream of Navan to the mouth of the River Boyne where it enters the Irish Sea. The reach of the River Boyne affecting Drogheda and Baltray AFAs along with all tributaries flowing through these AFAs, including the Baltray Stream, have been identified as HPWs.

The entire HPW reach of the River Boyne is influenced by sea levels at the mouth of the river. The remainder (upstream portion) of the River Boyne included in the model has been designated as MPW.

Within the model extents there are two gauging stations both located on the MPW reach of the River Boyne upstream of the tidal influence.

The upstream gauging station is at Slane Castle (07012 – OPW) and has an FSU classification of its rating of A1 for the entirety of its record length (1940 – 2010) meaning there is a high level of confidence in the Q_{med} value of 191.40 m³/s. There were however arterial drainage works carried out within the Boyne Catchment upstream of this point over the period 1969 to 1986 and as such the gauge record captures a

period where there has been a significant shift in the catchment behaviour in terms of the Q_{med} which has increased significantly since arterial drainage works were implemented. Consequently it is considered prudent to regard only flow data derived post arterial drainage works as reflective of current catchment behaviour.

The other gauging station is located approximately 6km downstream of Slane and is called Roughgrange (07059 – EPA). This station has a short record length (2006 – 2011) with gaps in the record. The station was not given a classification of its rating under FSU and no information was provided by EPA on the rating of the station. Following a hydrological data review it was concluded that there is little confidence in the rating. More detail on the use of these stations and the hydrology derived from them can be found in Section 4.7 of the HA07 Hydrology Report (IBE0600Rp0025_HA07_Hydrology Report).

The MPW reach of the River Boyne has been modelled using 1D techniques. This reach the river is contained within a well-defined valley adequately represented by 1D simulation. The HPW reaches have all been modelled using hydrodynamically linked 1D/2D modelling techniques using the ICM modelling software from Innovyze. Some of the tributaries contain large reaches of steep, culverted watercourses, these reaches have been included in the model using 1D conduit and manhole units to a detail which the available information allowed. A storm and foul sewer network schematic was supplied for a large percentage of the Drogheda urban area, this information was used to incorporate detailed route and profile culvert information where available.

(2) Model Reference:		HA07_DROG7_F02	
(3) AFAs included in the model and referred to in this section:		DROGHEDA and BALTRAY	
(4) Primary Watercourses / Water Bodies (including local names):			
<u>Reach ID</u>		<u>Name</u>	
0701		BOYNE RIVER	
0702		BALTRAY STREAM	
0703		DROGHEDA RIVER BOYNE UNKNOWN TRIB 2	
0704		NEWTOWN STALABAN STREAM	
0705		BEAULLIEU STREAM	
0706		DRY BRIDGE STREAM	
0707		DRY BRIDGE STREAM TRIBUTARY	
0708		DROGHEDA RIVER BOYNE UNKNOWN TRIBUTARY 1	
0743		BEAULLIEU STREAM TRIBUTARY	
(5) Software Type (and version):			
(a) 1D Domain: Infoworks ICM (Version 5)		(b) 2D Domain: Infoworks ICM Flexible Mesh	(c) Other model elements: NA

4.3.2 Hydraulic Model Schematisation

(1) Map of Model Extents:

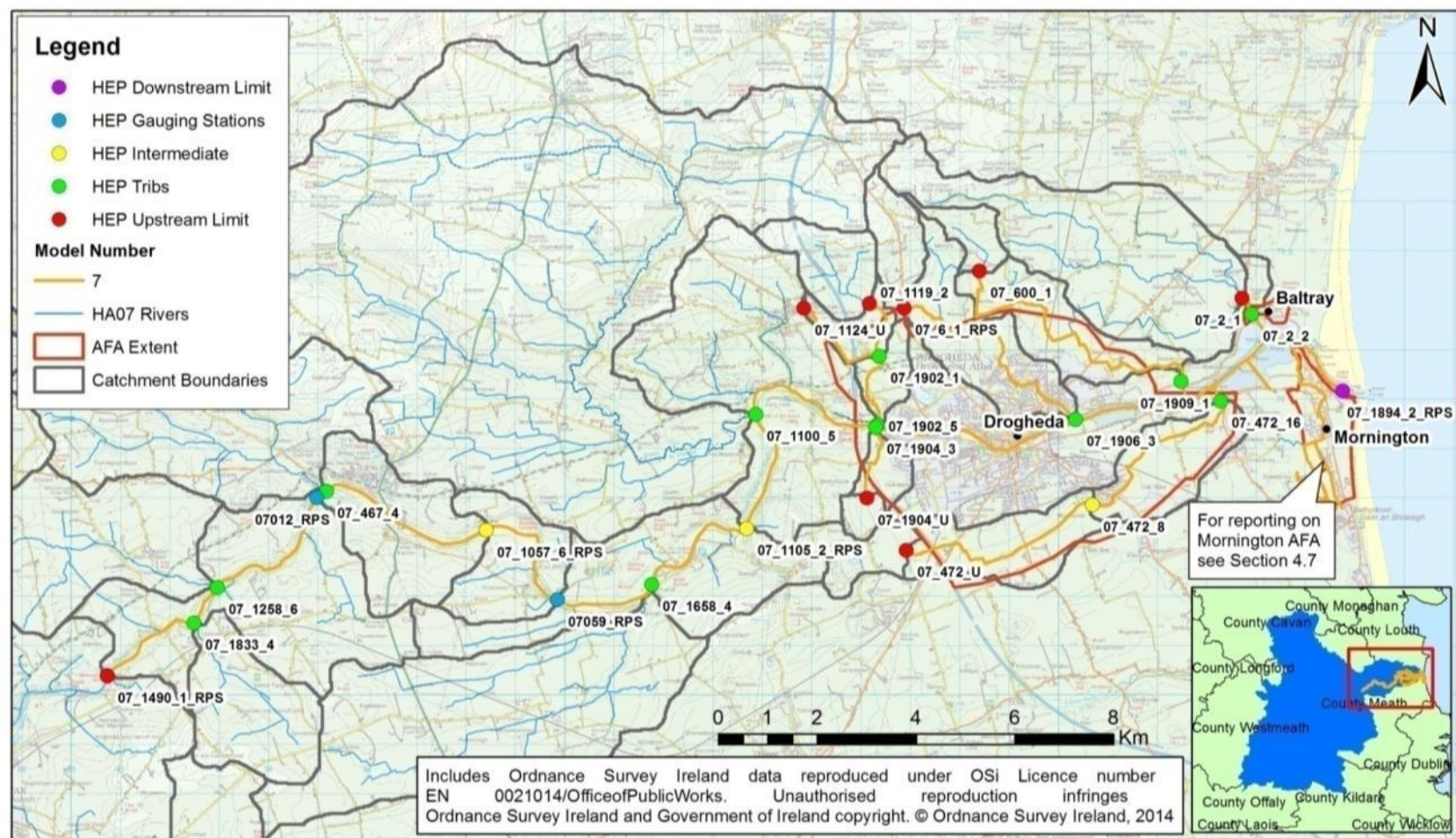


Figure 4.3.1: Map of Model Extents

Figure 4.3.1 illustrates the extent of the lower Boyne (Drogheda / Mornington / Baltray) model and Figure 4.3.2 shows the extent of the model applicable to the Drogheda and Mornington AFAs.

The extents, modelled river centre lines and HEPs are shown. The Drogheda and Baltray portions of the model contain 8 Upstream Limit HEP, 1 Downstream Limit HEP, 4 Intermediate HEPs, 10 Tributary HEPs and 2 Gauging Station HEPs. Details on the portion of the model relating to Mornington AFA are included in Section 4.7.

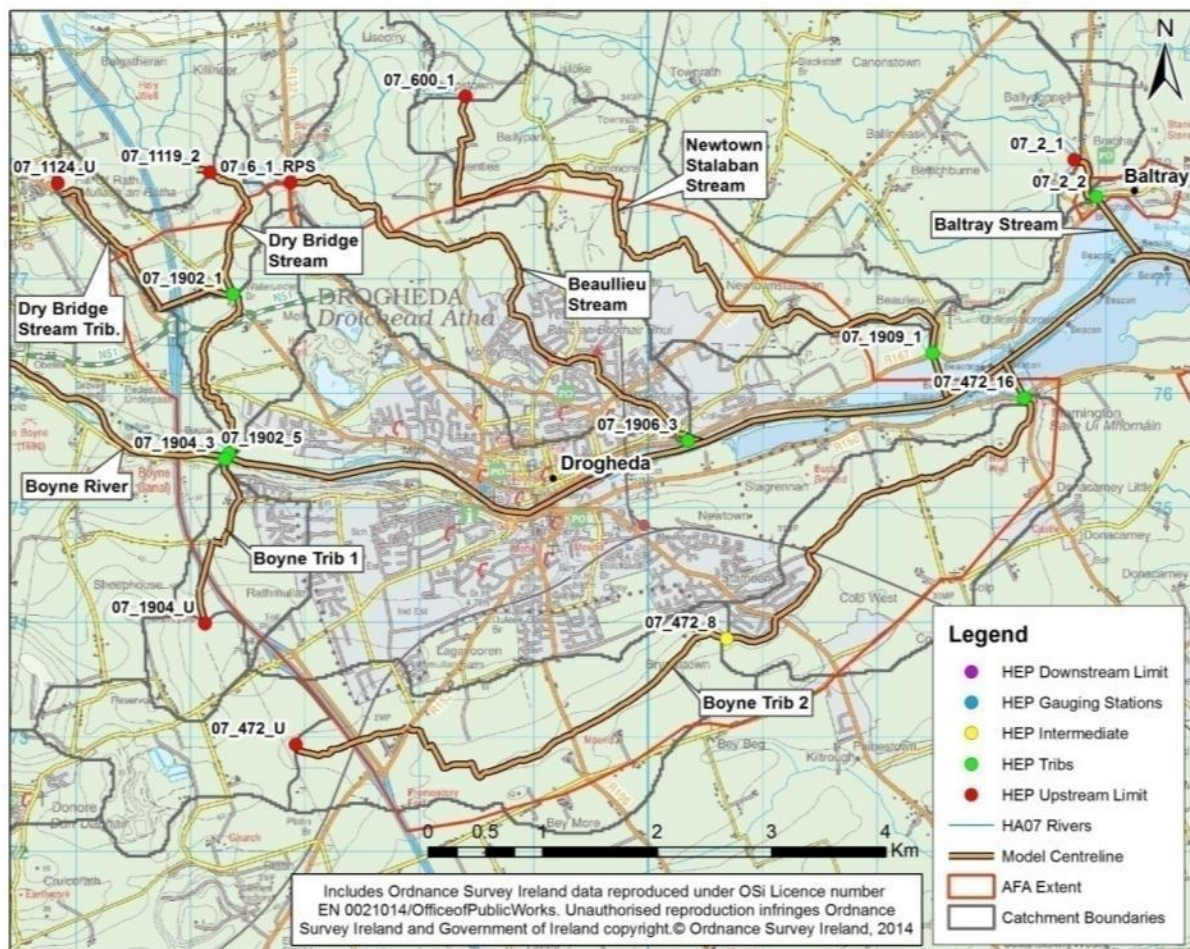


Figure 4.3.2: Map of Model Extents within AFA

(2) x-y Coordinates of River (Upstream extent):

River Name		x	y
0701	BOYNE RIVER	291773	271246
0702	BALTRAY STREAM	313723	278043
0703	DROGHEDA RIVER BOYNE UNKNOWN TRIBUTARY 2	306916	272943
0704	NEWTOWN STALABAN STREAM	308407	278601
0705	BEAULLIEU STREAM	306877	277886
0706	DRY BRIDGE STREAM	306058	277958
0707	DRY BRIDGE STREAM TRIBUTARY	304878	277741
0708	DROGHEDA RIVER BOYNE TRIB 1	306119	274016
0743	BEAULLIEU STREAM TRIB	307545	278033

(3) Total Modelled Watercourse Length:

63km

(4) 1D Domain only Watercourse Length:

19km

**(5) 1D-2D Domain
Watercourse Length:**

44km

(6) 2D Domain Mesh Type / Resolution / Area:Infoworks ICM Flexible Mesh / 1 - 25m²
/ 34.7km²**(7) 2D Domain Model Extent:**

The 2D domain of the model extends to over 2km upstream of the M1 Boyne Bridge at Drogheda (see Figure 4.3.3) to the coast. As shown in Figure 4.3.4 the 2D domain of the model covers the entire modelled reaches of the tributaries with the exception of a small section (400m) at the upstream extents of the Dry Arch Tributary which is outside the AFA. The MPW reaches of the Boyne upstream of the AFA, from the model upstream extents (5km upstream of Slane) to the bridge at Townleyhall Road west of the M1 is modelled as 1D.

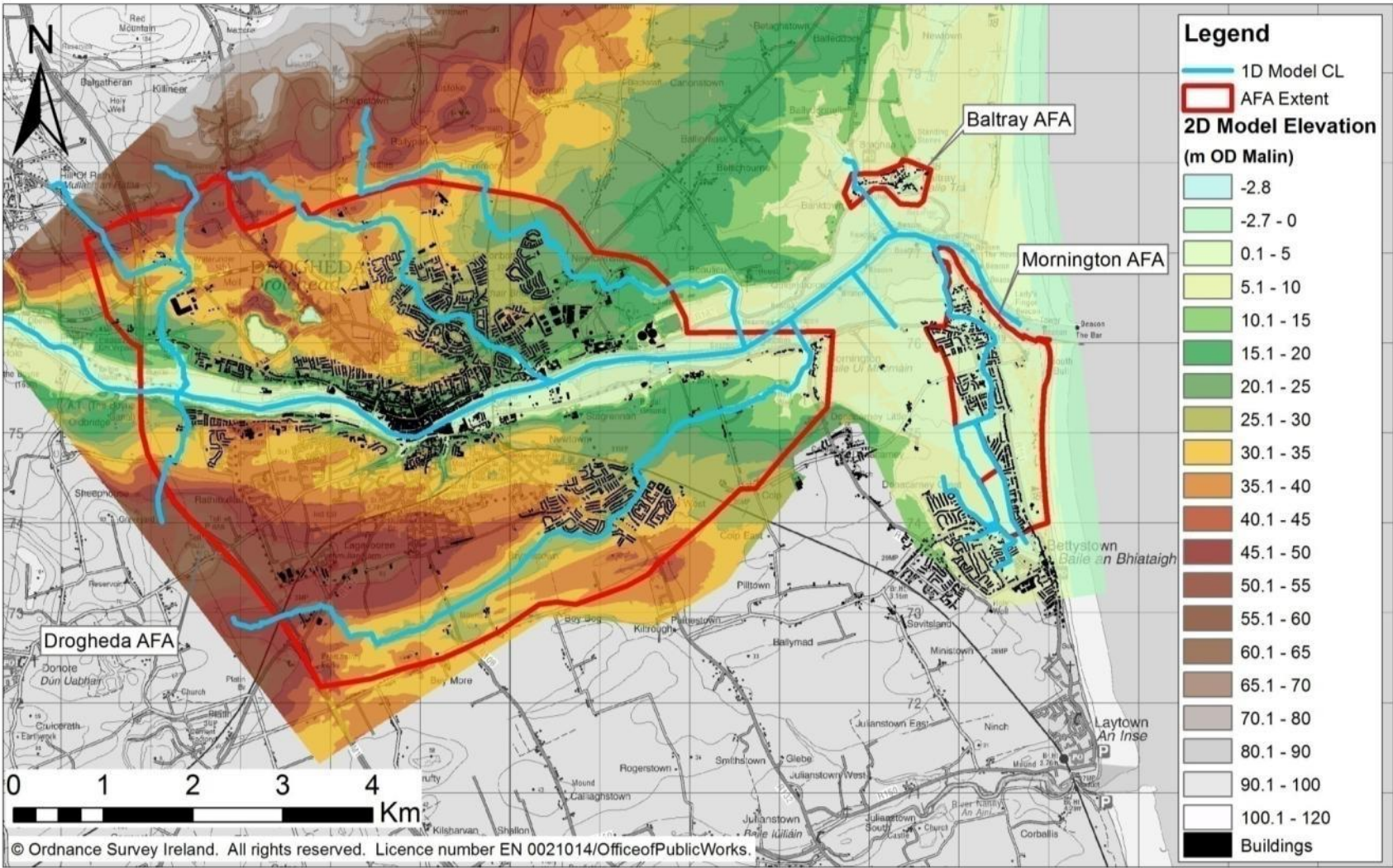


Figure 4.3.3: Model 2D Domain

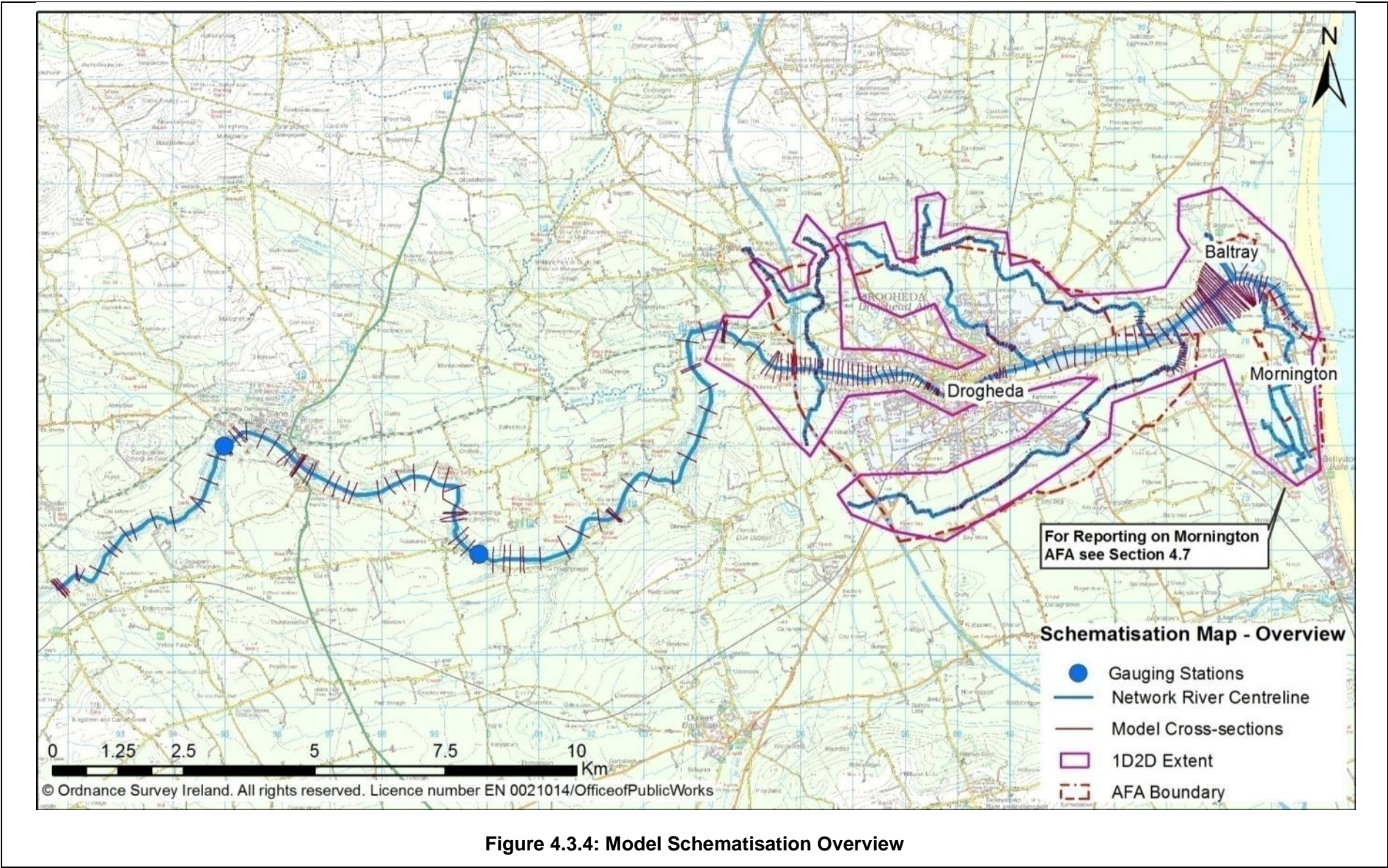


Figure 4.3.4: Model Schematisation Overview

The 2D domain was constructed from LiDAR data surveyed as part of the Study which is shown in Figure 4.3.5. Visual inspection found that there was an error in the Drogheda LiDAR data between the Drogheda and Mornington AFAs. This was corrected using LiDAR data from the extended LiDAR data survey covering the southern portion of the Mornington AFA.

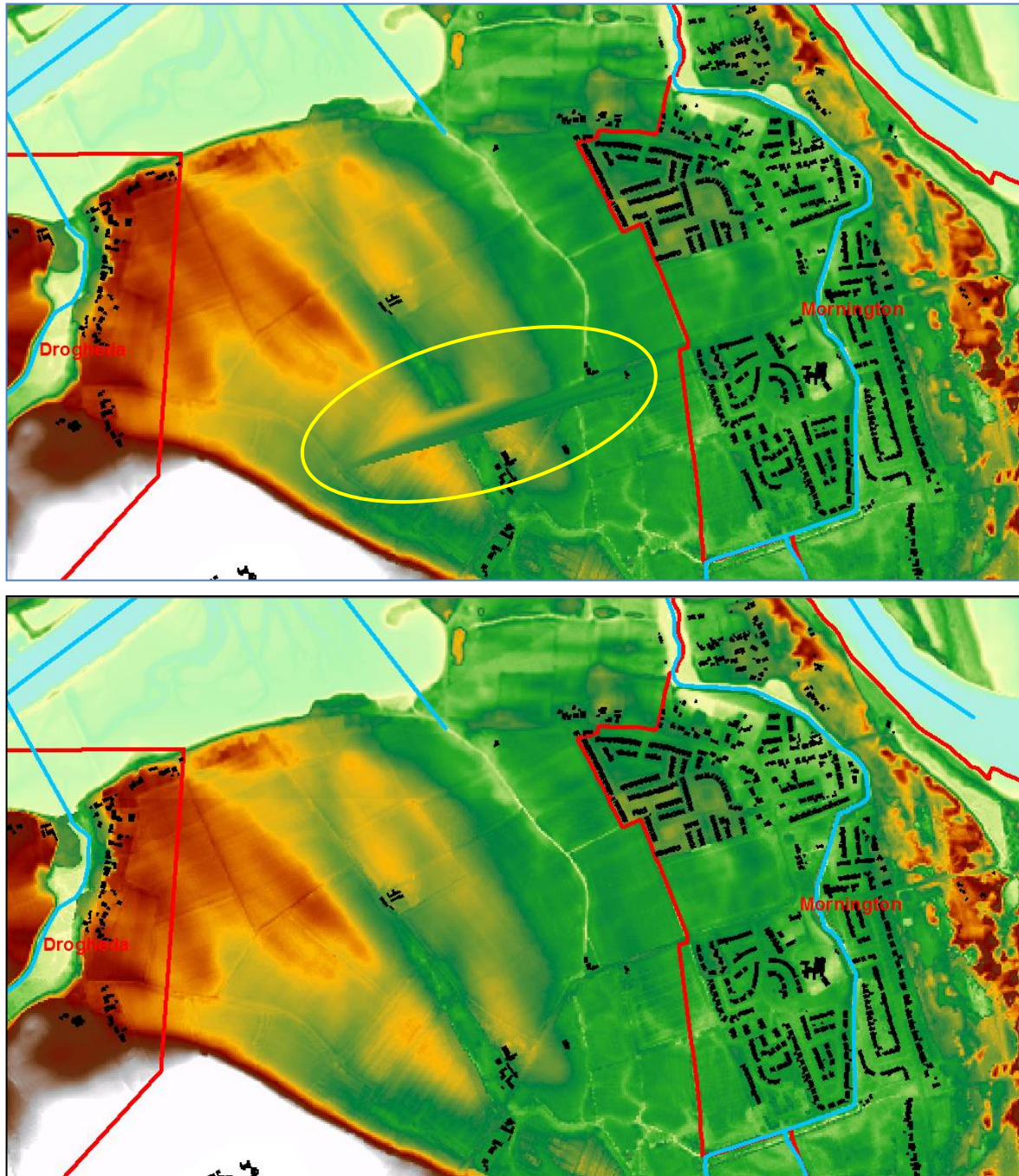


Figure 4.3.5: Erroneous Values in Original 2D Domain (top) and Correct (bottom)

Figure 4.3.6 to Figure 4.3.10 show detailed views of the model schematisation where there are critical structures and areas where there is the most significant risk of flooding. These diagrams include the surveyed cross-section locations, AFA boundary and river centre. They also show the location of the critical structures as discussed in Section 4.3.3(1), along with the location and extent of the links

between the 1D and 2D models.

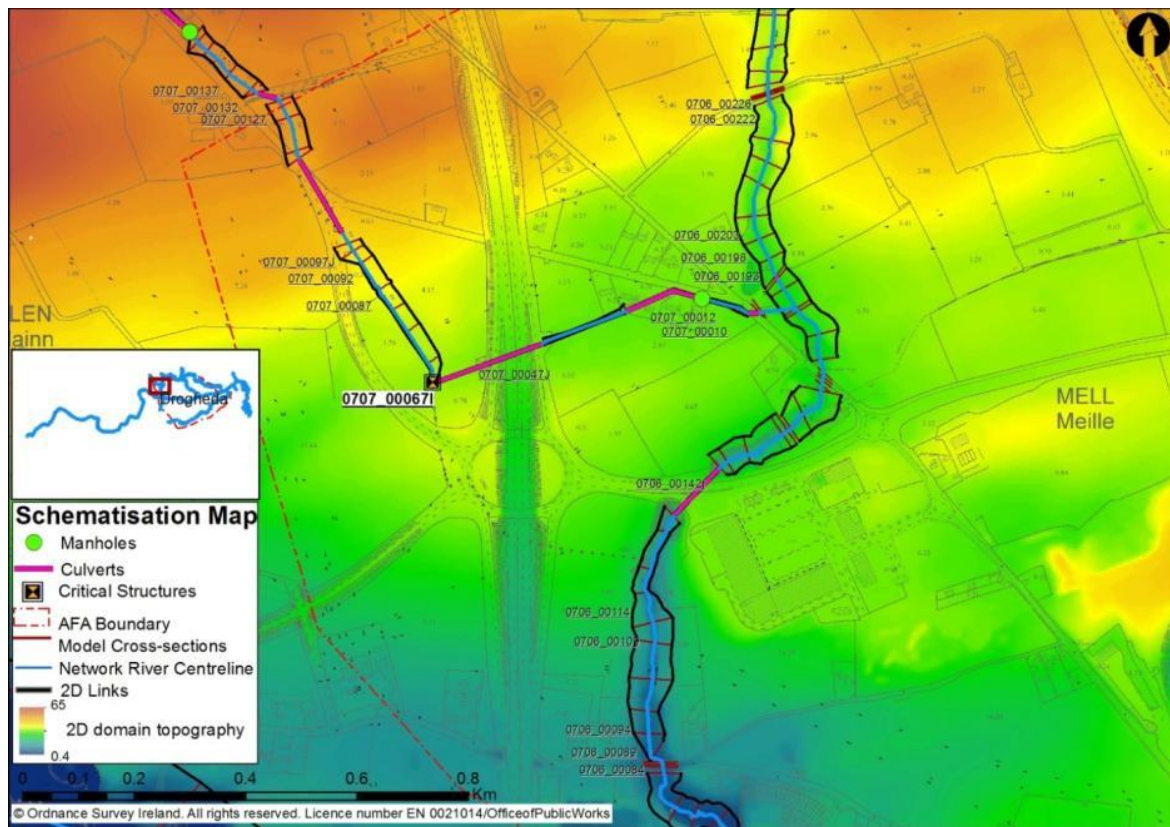


Figure 4.3.6: Detailed Schematisation of Model on Dry Bridge Stream inc. Critical Structures

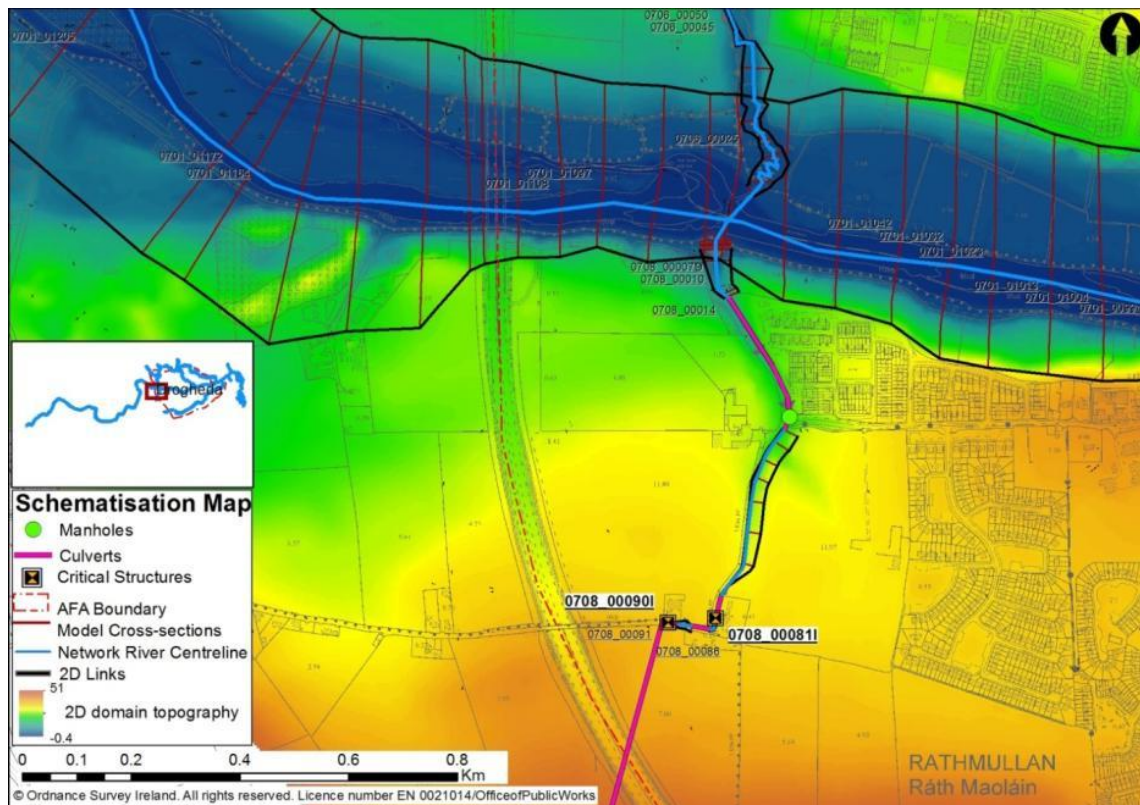


Figure 4.3.7: Detailed Schematisation of Model on Boyne Trib 1 including Critical Structures

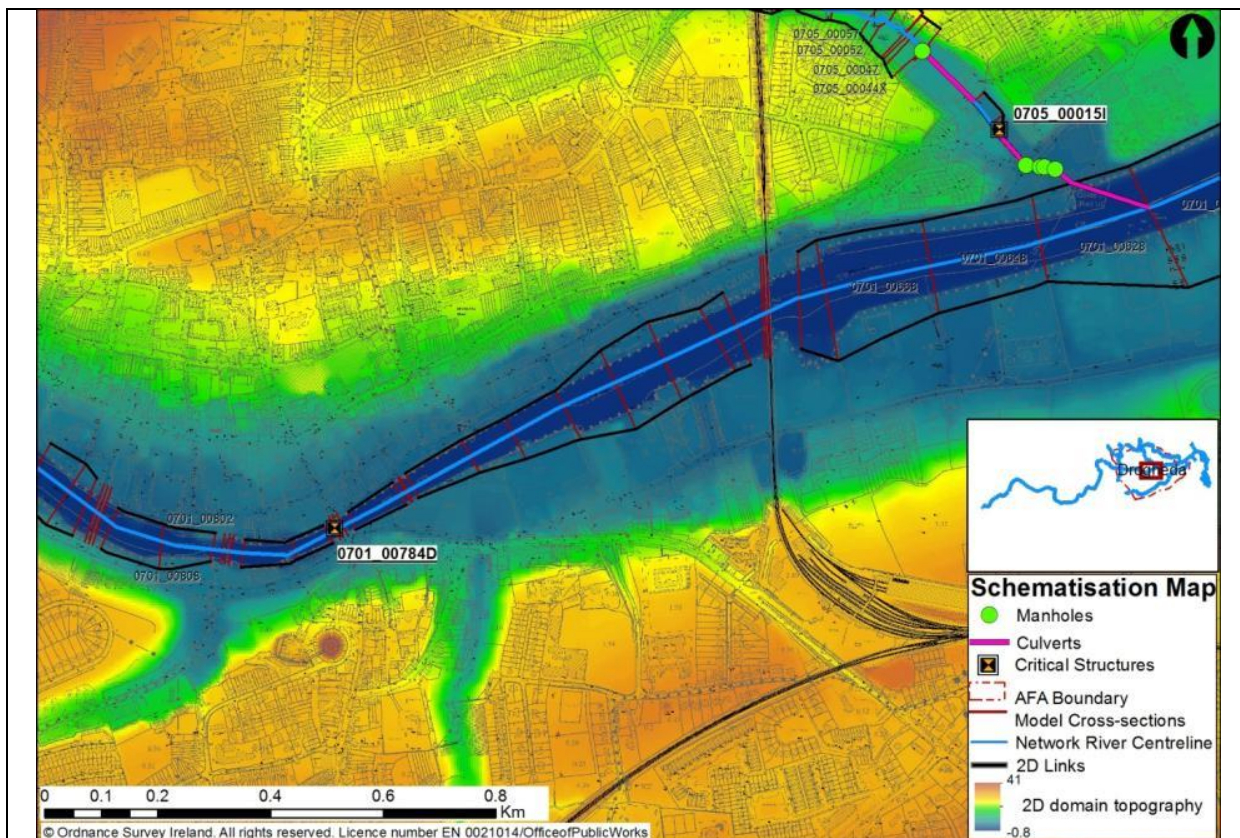


Figure 4.3.8: Detailed Schematisation of Model on River Boyne including Critical Structures

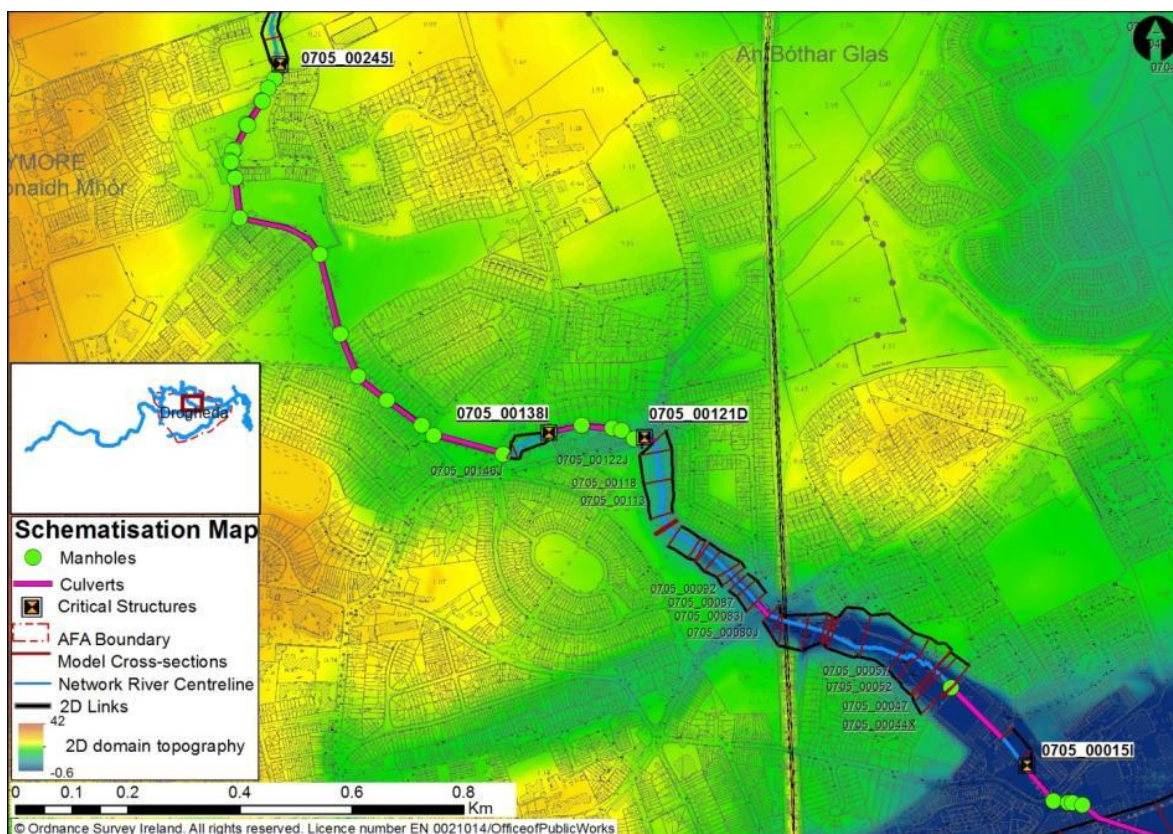


Figure 4.3.9: Detailed Schematisation of Model on Beaulieu Stream including Critical Structures

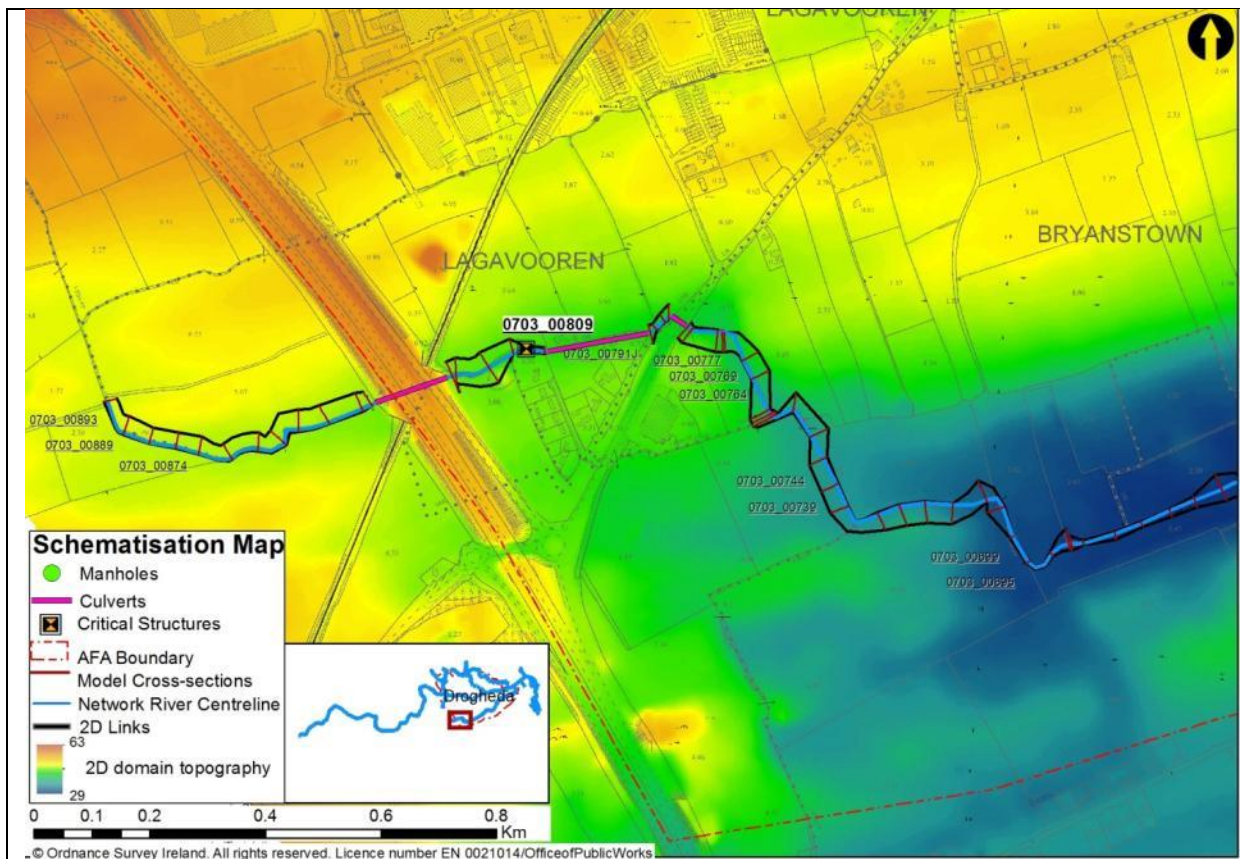


Figure 4.3.10: Detailed Schematisation of Model on Boyne Trib. 2 including Critical Structures

(8) Survey Information

(a) Survey Folder Structure:

First Level Folder	Second Level Folder	Third Level Folder
Murphy_E07_M07_WP4_0701A_120702 Drogheda Murphy: Surveyor Name E07: Eastern CFRAM Study Area, Hydrometric Area 7 M07: Model Number 7 0701A: River Reference WP4: Work Package4 Version: Most up to date 120702: Date Issued (02 nd JUN 2012)	V0_0701_A_Ascii	
	V0_0701_A_Photos	0701_00014_DN
	V0_0701_A_GIS and Floodplain Photos	Flood_Defence_Register
	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	

(b) Survey Folder References:		
Reach ID	Name	File Reference
0701	BOYNE RIVER	<i>Murphy_E07_M07_WP4_0701A_120702</i>
0701	BOYNE RIVER	<i>Murphy_E07_M07_WP4_0701B_120706</i>
0702	MORNINGTON TRIBUTARY	<i>Murphy_E07_M07_WP4_0702_120702</i>
0703	DROGHEDA RIVE BOYNE TRIBUTARY 2	<i>Murphy_E07_M07_WP4_0703_120702</i>
0704	NEWTOWN STALABAN STREAM	<i>Murphy_E07_M07_WP4_0704_120702</i>
0705	BEAULLIEU STREAM	<i>Murphy_E07_M07_WP4_0705_120702</i>
0706	DRY BRIDGE STREAM	<i>Murphy_E07_M07_WP4_0706_120702</i>
0707	DRY BRIDGE STREAM TRIBUTARY	<i>Murphy_E07_M07_WP4_0707_120702</i>
0708	DROGHEDA RIVER BOYNE TRIBUTARY 1	<i>Murphy_E07_M07_WP4_0708_120702</i>
0743	BEAULLIEU STREAM TRIBUTARY	<i>Murphy_E07_M07_WP4_0743_120702</i>
(9) Survey Issues:		
<p>The surveyed cross sections along the MPW reach of the River Boyne did not extend a sufficient distance beyond top of bank to cover the entire floodplain. All surveyed cross sections along this reach were augmented using the NDHM dataset to enable full coverage of the MPW floodplain to be included in the 1D domain of the model. The transition between the surveyed cross section dataset and the NDHM was manually edited to ensure a smooth transition between the datasets within the ICM software. No additional edits were made to the data extracted from the NDHM.</p> <p>A minor survey query was raised for the Drogheda model. The query was resolved, as outlined below.</p> <p>On the Beaulieu Stream (0705), at the Old Mill site on the north side of the river Boyne in Drogheda, just east of the Belfast Railway line, a culvert was surveyed however it lacked some critical information to represent it accurately within the model.</p>		

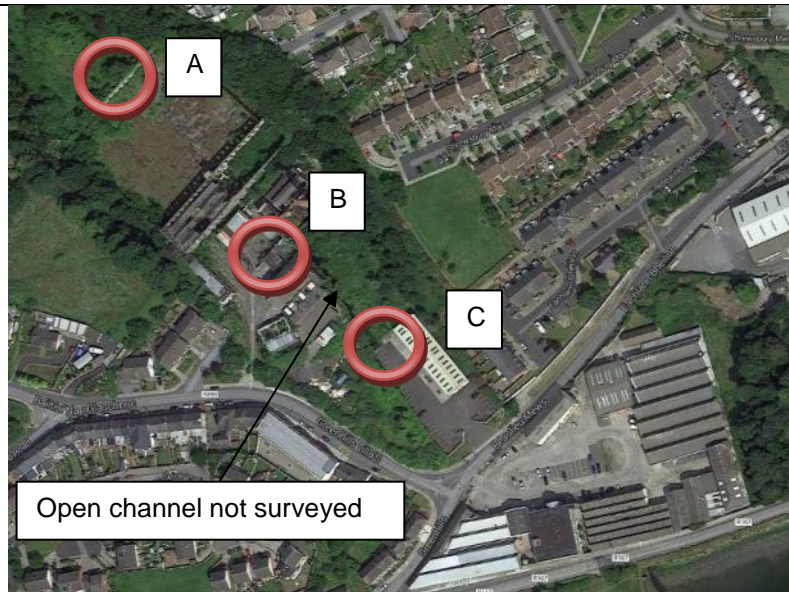


Figure 4.3.11: Location of Survey Query

At Location A, see Figure 4.3.11, where the watercourse enters a culvert there were no inlet details provided. At Location B the watercourse exits a culvert outlet details were not provided and at subsequent cross section of the reach of open water course downstream of the mill before it enters the outfall pipe to the Boyne. Instruction was given to obtain the following information:

- invert level of culvert inlet at location A and culvert outlet B
- Cross sections of open channel reach between location B
- Invert level of culvert inlet to the outfall pipe at location C.

This information was obtained and incorporated into the model.

4.3.3 Hydraulic Model Construction


(1) 1D Structures (in-channel along modelled watercourses):	See Appendix C Number of Bridges and Culverts: 85 Number of Weirs: 24
<p>Critical and Hydraulically Significant Structures:</p> <p>The structures along the modelled reaches have been defined based on the cross sectional survey information relating to the modelled reaches, photographs and site walkover survey. Details of culvert inlet and outlet headwall structures have been defined based on the surveyed cross sections and photographs however details of the culverts between inlet and outlet have been defined based on the drainage network drawings provided by Louth County Council. This information included culvert dimensions, invert levels, manhole locations, invert and cover levels. Discussion on modelling approaches for structures is included in Chapter 3.3.3 and maps showing the location of critical structures within this model are shown in Figure 4.3.6 to Figure 4.3.10.</p>	
<p>0701_00784 St Mary's Bridge is one of six bridges across the River Boyne in Drogheda. The river channel width reduces at this location leading to increased in-channel water levels and flooding on the left bank upstream in the 10%, 1% and 0.1% AEP events. Bridge deck levels have been defined through the surveyed cross section information as a cross section within the bridge component of the 1D model. The deck is not overtopped during any of the simulated events as left bank levels upstream of the bridge are lower such that flood waters spill before in channel water levels reach the deck of the bridge. The bridge location within the model is shown in Figure 4.3.8.</p>	
	
<p>Figure 4.3.12: View of St. Mary's Bridge from Upstream (0701_00784)</p>	
<p>0703_00809 The culvert located just to the east of the M1 on the tributary (unknown Boyne Trib 2) which runs to the south of the AFA surcharges in the 0.1%, 1% and 10% AEP and causes flooding to at least one property in the 0.1% AEP event.</p>	



Figure 4.3.13: View of Culvert Inlet at 0703_00809

This 350mm diameter culvert location is shown in Figure 4.3.10 and is represented as a 178m conduit within the model.

0703_00377 This culvert on the watercourse surveyed as Unknown Boyne Tributary 2 causes backing up and attenuation of flows at all design events, the flooding upstream is confined to agricultural land. There are no receptors affected and as such the structure is not considered critical.

0705_00393 Flooding of agricultural land occurs upstream of this access bridge. The flooded area acts as a storage area during flood events. There are no receptors affected and as such the structure is not considered critical but is hydraulically significant.



Figure 4.3.14: View of Culvert Inlet at 0705_00393

0705_000245 – 0705_00146 The culverted length of the Beaulieu Stream which passes under the Lia

Bhrega, Moneymore and Brookville housing areas does not flood at the inlet structure shown in Figure 4.3.15. However during the modelled 0.1% AEP event the capacity of these twin culverts is exceeded and flooding from manholes along the culvert length is predicted by the model.



Figure 4.3.15: View of Culvert Inlet at 0705_002454

The 1350mm diameter twin culvert location and route is shown in Figure 4.3.9 and is represented within the model as a series of twin conduits with dimensions, manhole locations and levels from drainage network record drawings provided by Louth County Council. Inlet and outlet headloss coefficients have been applied based on the arrangements of the structure as shown in the survey and photographs.

0705_000138 The culvert inlet for the Beaulieu Stream just upstream, of Flaxmill Court is modelled as surcharging in the 10%, 1% and 0.1% AEP design simulations. Only in the 0.1% AEP event however is this affecting properties at Flaxmill Court.

The screen is also represented in the model. The twin 1200mm culverts downstream of the inlet are represented in the model based on the information provided on the drainage record drawings.



Figure 4.3.16: View of Culvert Inlet at 0705_00138

0705_000121 The culvert inlet for the Beaulieu Stream crossing Flaxmill Lane (Figure 4.3.9) is modelled as surcharging in the 10%, 1% and 0.1% AEP design simulations. During the 1% and 0.1% AEP fluvial events Flaxmill Lane is simulated as inundated from the surcharged culvert inlet.



Figure 4.3.17: View of Culvert Outlet at 0705_00121

The screen is also represented in the model. The culvert dimensions are not included in drainage record drawings and are assumed to be consistent with the arched inlet and outlet structures. This is

considered appropriate given that the culvert is short in length (15m) and the conduit between inlet and outlet can be assumed to be consistent, as there no manholes present.

0705_00015 This culvert is located at the downstream end of the derelict mill on the Beaulieu Stream and passes under the Newtown Road as shown in Figure 4.3.9. Flooding occurs in this area affecting the Greenhills Industrial Estate. The culvert has been modelled as free flowing, however as can be seen from Figure 4.3.18 it is susceptible to blockage.



Figure 4.3.18: View of Culvert Inlet at 0705_00015

This culvert was initially not picked up in the survey and was subject to additional survey to ascertain the culvert and open channel arrangements from the old mill building to the Boyne (see Section 4.3.2(9)). The culvert dimensions, manhole locations and inverts downstream of the inlet are represented in the model based on the information provided on the drainage record drawings.

0707_00067 The Dry Bridge Stream Tributary culvert which crosses under the M1 to the north west of the AFA surcharges in the 0.1% leading to flooding of the M1 carriageway and slip road.



Figure 4.3.19: View of Culvert Inlet at 0705_00015

The 600mm diameter culvert location is shown in Figure 4.3.6 and is represented within the model as a 206m conduit within the model. Inlet and outlet headloss coefficients have been applied based on the arrangements of the structure as shown in the survey and photographs.

0708_00090 and 0708_00081 Flooding is evident in the modelled events from two culverts on a rural section of watercourse surveyed as Drogheda Boyne unknown tributary 1. The culverts are located just inside the M1 on the western edge of the Drogheda AFA. The first culvert is a 700mm diameter circular pipe approximately 18m in length providing access to farm buildings. The second (65m further downstream) is a 600mm diameter circular pipe approximately 44m in length providing access to properties. The capacity of the first culvert and the channel upstream is exceeded during the 0.1% and 1% AEP events whereas flooding in the model is predicted at the face of the downstream culvert during the 1% AEP event.

The location of both critical structures is shown in Figure 4.3.7. Both culverts are located within a heavily vegetated watercourse to the side of the road.

(2) 1D Structures in the 2D domain (beyond the modelled watercourses):	None
(3) 2D Model structures:	None
(4) Defences:	
Following discussions with the OPW and Louth County Council, two defences were identified as being formal, effective flood defences such that they would warrant inclusion in the model. However both defences are dependent on adjacent flood defences which are considered ineffective to provide the	

minimum standard of protection. An assessment of the defences identified as effective found that their effectiveness is dependent on the contiguous ineffective defences and as such their representation within the model in isolation does not provide the minimum standard of protection. As such no defences have been included in the model. The locations of all defences identified are shown in Figure 4.3.45 and Figure 4.3.46 and further discussion on their conditions and effectiveness provided in 0 (3).

Type	Watercourse	Bank	Model Start Chainage (approx.)	Model End Chainage (approx.)
Wall, Formal Effective	Boyne	Left	7580	7280
Wall, Formal Effective	Baltray	Left	120	60

(5) Model Boundaries - Inflows:

Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0025_HA07 Hydrology Report - Section 4.7 and Appendix D). The boundary conditions implemented in the model are shown below.

Node ID	Boundary Type	Description	Branch Name	Boundary Co-ordinates
0701_Inflow	Point Inflow	US Boundary	River Boyne	291773, 271246
0702_Inflow	Point Inflow	US Boundary	Baltray River	313723, 278043
0703_Inflow	Point Inflow	US Boundary	Unknown Trib 2	306916, 272943
0704_0731	Point Inflow	US Boundary	Newtown Stalaban Stream	308407, 278601
0705_Inflow	Point Inflow	US Boundary	Beaulieu Stream	306877, 277886
0706_Inflow	Point Inflow	US Boundary	Dry Bridge Stream	306058, 277958
0707_Inflow	Point Inflow	US Boundary	Dry Bridge Stream Trib	304878, 277741
0708_Inflow	Point Inflow	US Boundary	Unknown Trib 1	306119, 274016
0701_03217X	Point Inflow	Tributary	Unknown	292180, 271523
0701_03114	Point Inflow	Tributary	Unknown	292883, 272078
0701_02754	Point Inflow	Tributary	Unknown	295269, 274206
0701_01831Br_US	Point Inflow	Tributary	Unknown	302421, 272656

The lateral inflows as defined in the hydrology report were disaggregated for specific reaches of the watercourse between hydrology nodes and distributed pro-rata, based on length and applied to each link (river reach and conduit) along the length of the watercourses.

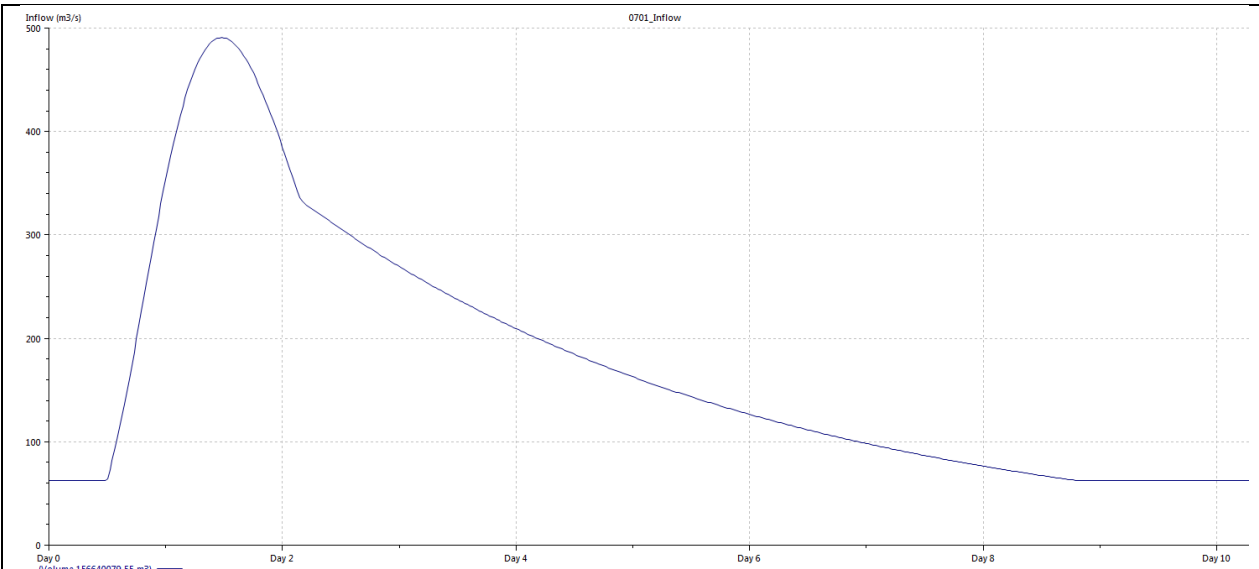


Figure 4.3.20: Boyne Upstream Inflow Hydrograph for a 1% AEP Fluvial Flood Event

(6) Model Boundaries Downstream Conditions:	<p>The modelled downstream boundary condition is taken from the ICPSS node NE_09 at the mouth of the Boyne Estuary where it meets the Irish Sea such that the effects of extreme coastal water levels in the Boyne Estuary can be considered within the model. A range of extreme coastal water level boundaries from 50% AEP to 0.1% AEP have been considered.</p> <p>The extreme coastal water level boundary has been developed using a tidal cycle halfway between an astronomical mean high water and a mean high water spring tide at Dublin Port. A typical 48 hour surge profile has then been applied to achieve a peak water level at the appropriate ICPSS extreme water level for NE_09 node. The fluvial design hydrographs have been shifted in time such that the peak occurs simultaneously with peak water level.</p> <p>The likelihood of joint occurrence of extreme fluvial events and extreme coastal events is discussed in the Hydrology Report and it is not considered that there is evidence of significant joint occurrence. However boundary conditions for the non dominant event to be considered have been kept at 50% AEP for fluvial and coastal dominated model runs in line with a precautionary approach.</p>
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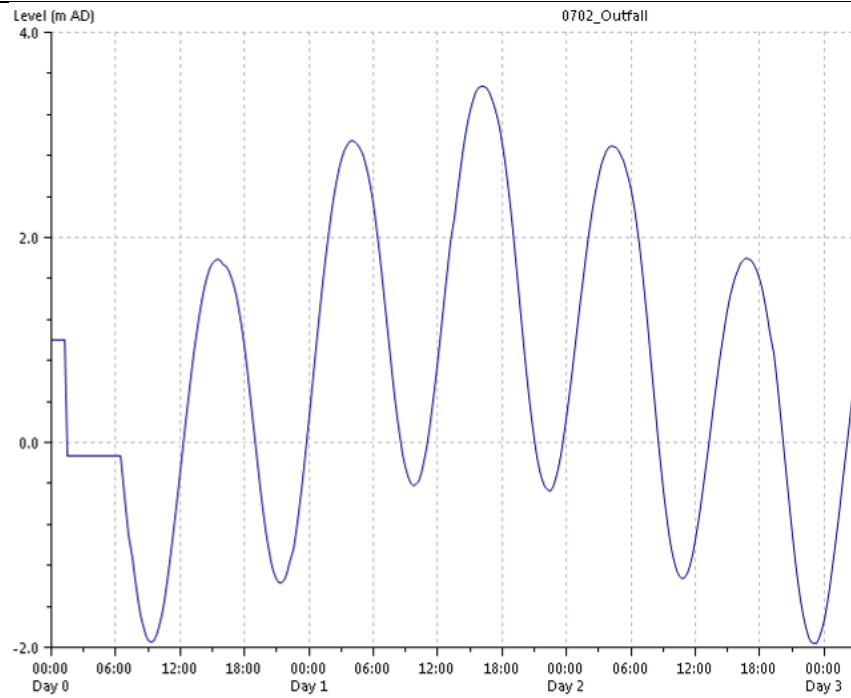


Figure 4.3.21: 0.5% AEP Coastal Water Level Boundary applied at 0701_Outflow

The coastal boundary has been applied at the model node '0701_Outflow' representing a 1D boundary at the mouth of the Boyne Estuary where it meets the Irish Sea. It is not considered that the boundary needs to be applied to a 2D boundary at this location as the channel is well defined with high ground to either side.

It is also considered that there is no flood risk to Baltray directly from the open coastline as a constant high ground level (above 4m) exists between the beach and the AFA.

(7) Model Roughness:

Roughness values specified for the 1D portions of the model have been specified for each reach for in-channel and banks where appropriate based on the values laid out in Section 3.3.5.

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.030	Maximum 'n' value: 0.070
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.040	Maximum 'n' value: 0.060
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.013	Maximum 'n' value: 0.070

Figure 4.3.22 and Figure 4.3.23 illustrate the roughness values applied within the 2D domain of the River Boyne computational model. Roughness in the 2D domain was applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset as discussed in Section 2.5.

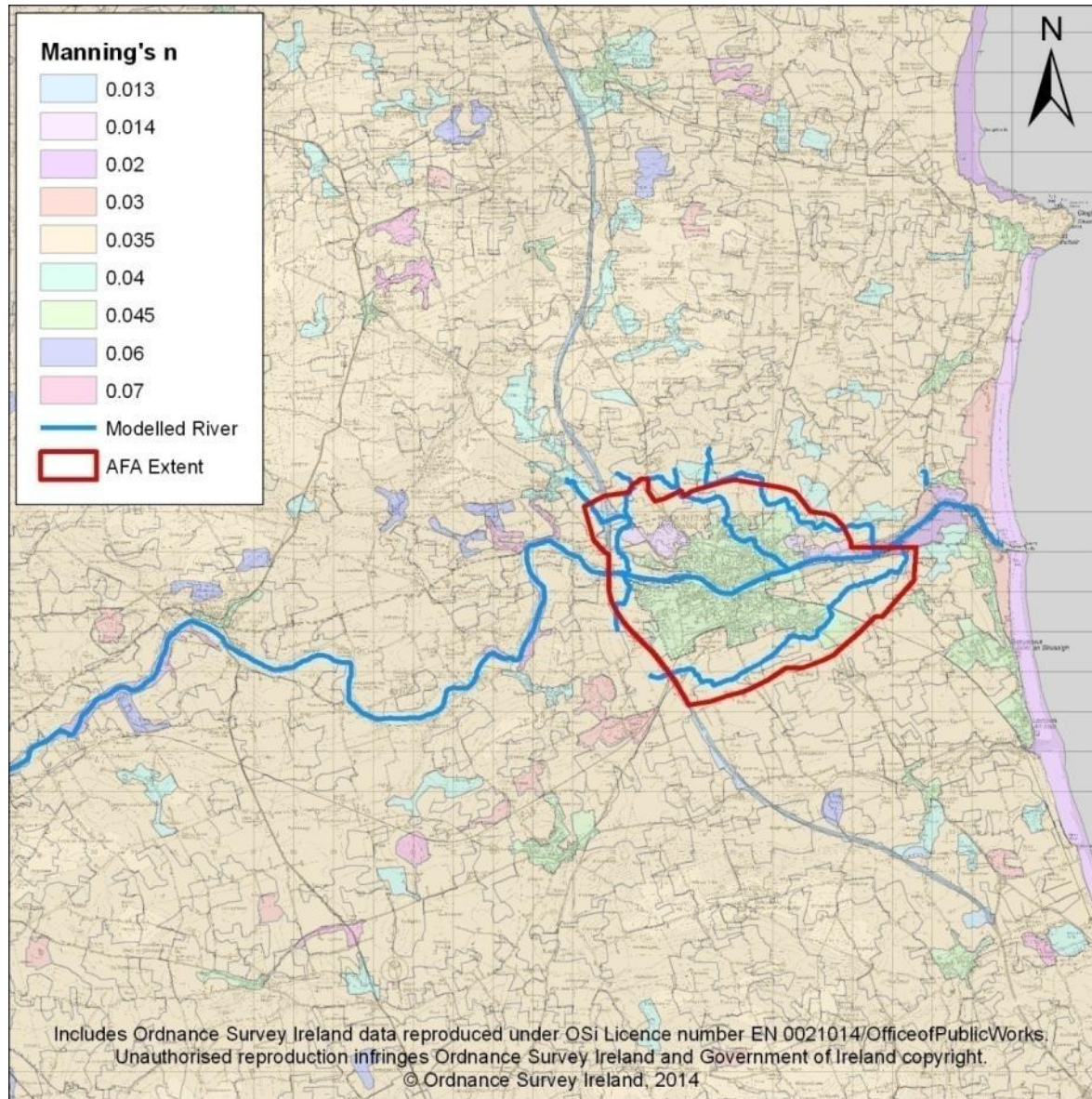


Figure 4.3.22: Map of 2D Roughness used for Drogheda AFA (Manning's n)

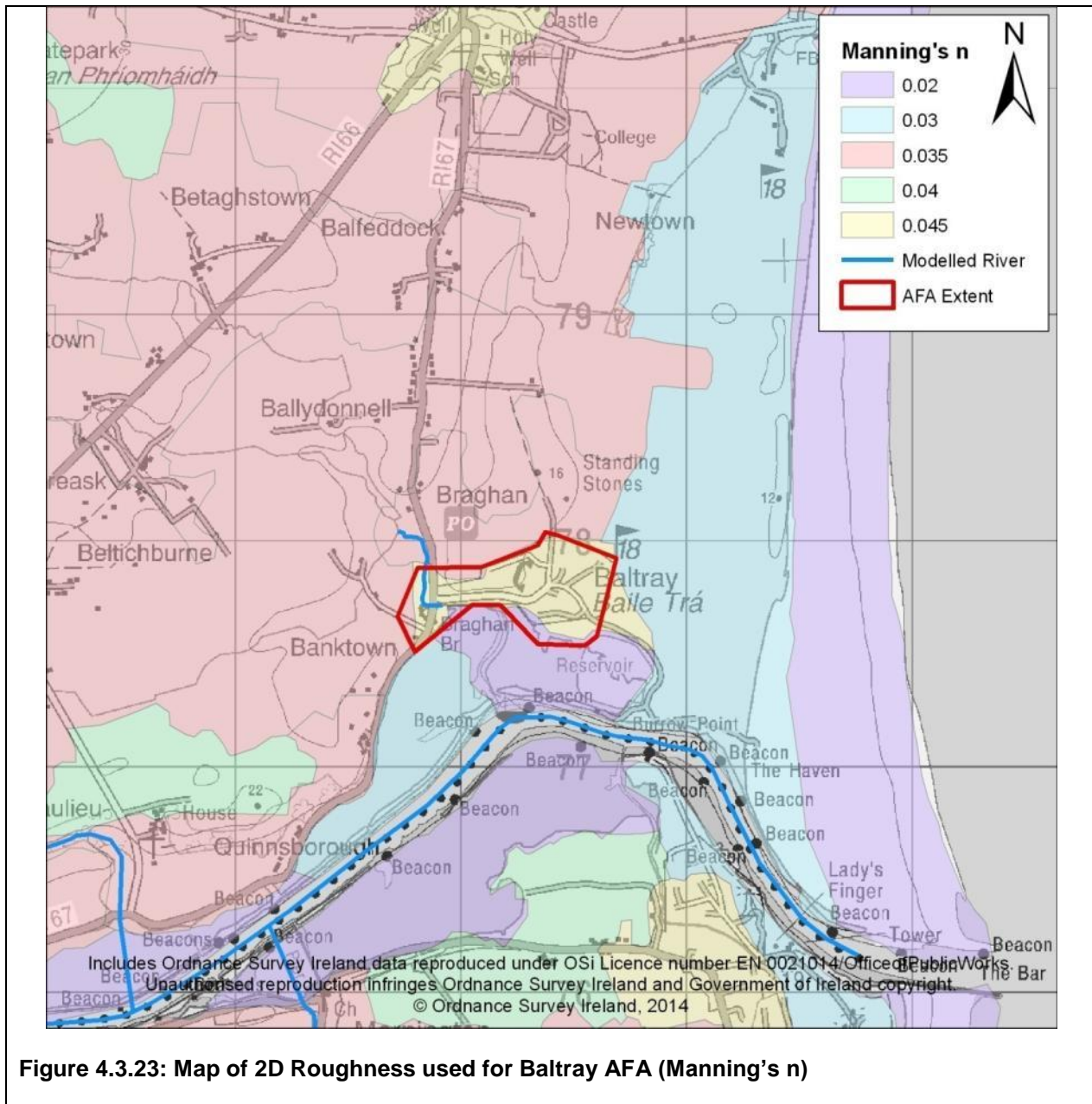


Figure 4.3.23: Map of 2D Roughness used for Baltray AFA (Manning's n)

(d) Examples of In-Bank Roughness Coefficients

Figure 4.3.24: Example of in-bank roughness for Beaulieu Stream

0705_00113 $n = 0.030$

Clean winding concrete channel



Figure 4.3.25: Example of in-bank roughness for Boyne River

0701_00851 $n = 0.040$

Large clean slightly meandering channel



Figure 4.3.26: Example of in-bank roughness for Dry Bridge Stream

0706_00089 $n = 0.055$

Stream with some stones and cobbles with some over hanging vegetation



Figure 4.3.27: Example of in-bank roughness for Newtown Stalaban Stream

0704_00552 $n = 0.060$

Stream with stones and debris and intrusive vegetation



Figure 4.3.28: Example of in-bank roughness for Boyne Unknown Trib 2

0703_00112 $n = 0.065$

Overgrown sluggish channel



Figure 4.3.29: Example of in-bank roughness for Boyne Unknown Trib 2 (upper reaches)

0703_00764 $n = 0.070$

Heavily overgrown channel

4.3.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact 1% AEP hydraulic model within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a moderate increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.3.30. This outcome indicates that the Drogheda model demonstrates a moderate sensitivity to changes in roughness parameters. This change to the flood extents has had a high impact upon properties located within the Drogheda AFA, since an additional 43 properties are affected. This is a 28% increase when compared to the design event. Conversely, there is no further impact to properties located within Baltray AFA and when considered separately Baltray can be considered as having low model sensitivity.

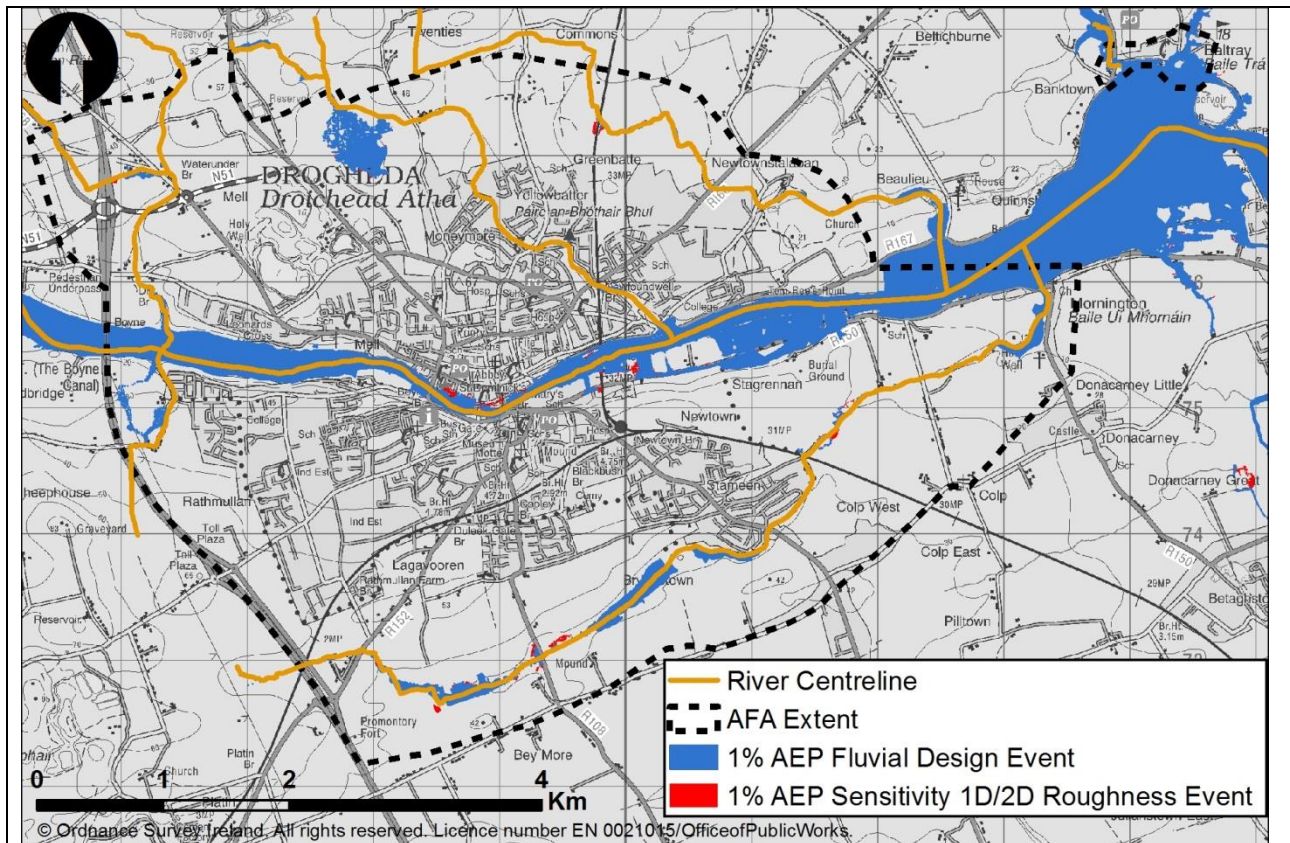


Figure 4.3.30 Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in 1D/2D Roughness

- b) Downstream boundary increase – The downstream coastal boundary located at the Boyne Estuary was increased to the water level generated from the 0.5% AEP mid-range future design scenario (peak water level 4 m OD). Changing the boundary condition increases the peak water levels at the downstream extent of the model. Figure 4.3.31 shows that the Drogheda AFA section of the model demonstrates a moderate sensitivity to downstream boundary increase with an extensive spatial extension of flooding, particularly riverside properties, including the Mall and Merchants Quay. This increase has a high impact upon receptors as 73 additional properties are affected, accounting as a 47% increase when compared to the design event. The model also indicates that the Baltray AFA shows a high sensitivity since 12 additional properties affected. Properties affected within Baltray AFA are mainly located along Rabbitt Street, particularly those within closest proximity to the estuary. This high impact is a 43% increase when compared to the design event.

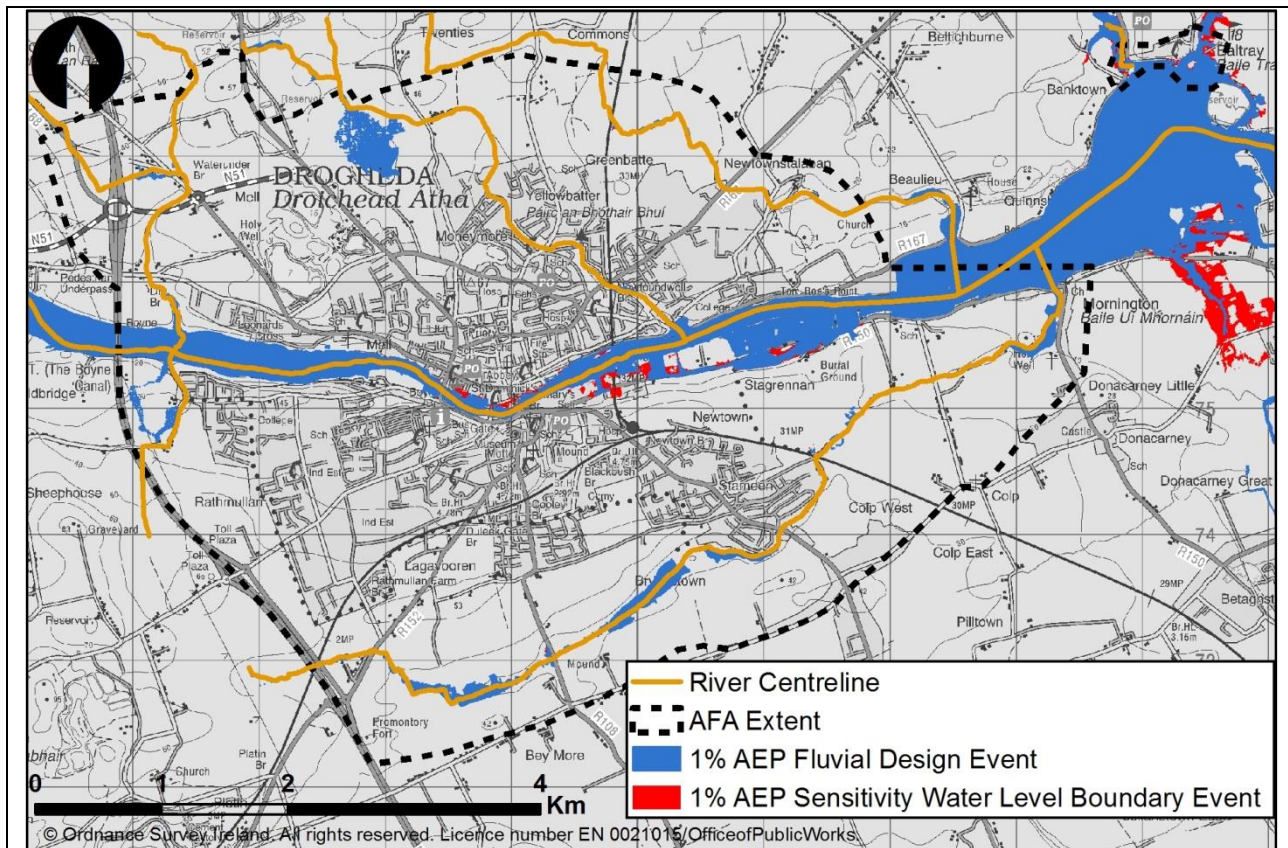


Figure 4.3.31: Comparison between 1% AEP Design Event & 1% AEP Sensitivity Water Level Boundary Event

- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Drogheda model is assessed as having low uncertainty/sensitivity (refer to the Hydrology Report IBE0600Rp0012 for further detail); factors of 1.25 and 1.68 are applied to design flows for the sensitivity simulation. Figure 4.3.32 shows that the Drogheda model has a moderate sensitivity to increased inflow parameters; this is reflected by the significant increase of flood extent. Significant increase in flood extents is located at the Stockwell Lane, Dominick Street and Patrickswell Lane, particularly those properties located within a close proximity to the River Boyne within the centre of Drogheda. These changes have a high impact on receptors as approximately 77 additional buildings are affected. This is a relative 50% increase when compared to the 1% AEP design. Within the Baltray AFA, 1 additional property is affected along Rabbitt Street; this low impact is 4% increase when compared to the design event.

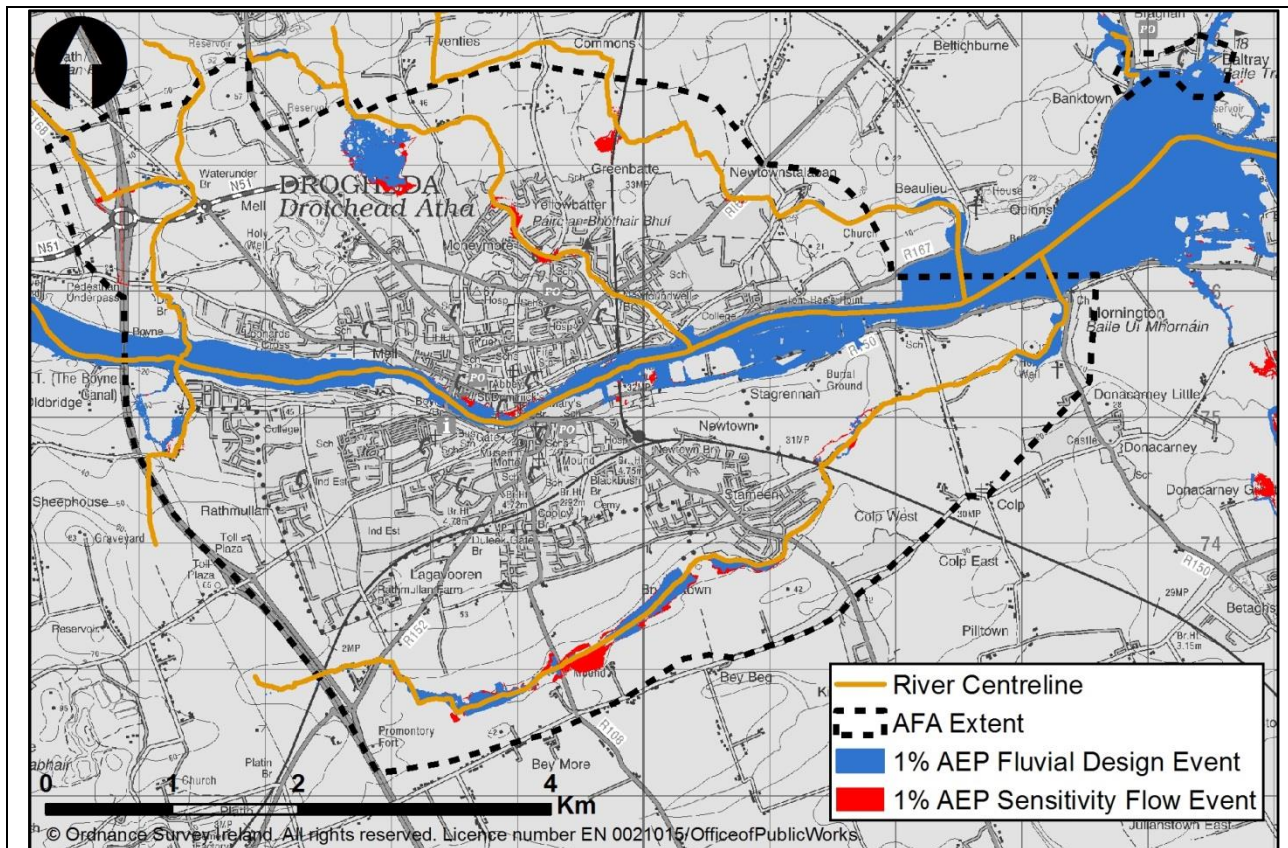


Figure 4.3.32: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event

- d) Floodplain roughness decreased to lower bound values – The change in floodplain roughness values resulted in low change of the flood extents within the AFA and therefore indicating a low model sensitivity see Figure 4.3.33 Both the Drogheda and Baltray AFA sections of the model can be considered to have a low sensitivity to 2D roughness parameters. Furthermore, when compared to the design event no additional receptors are impacted within these AFAs.

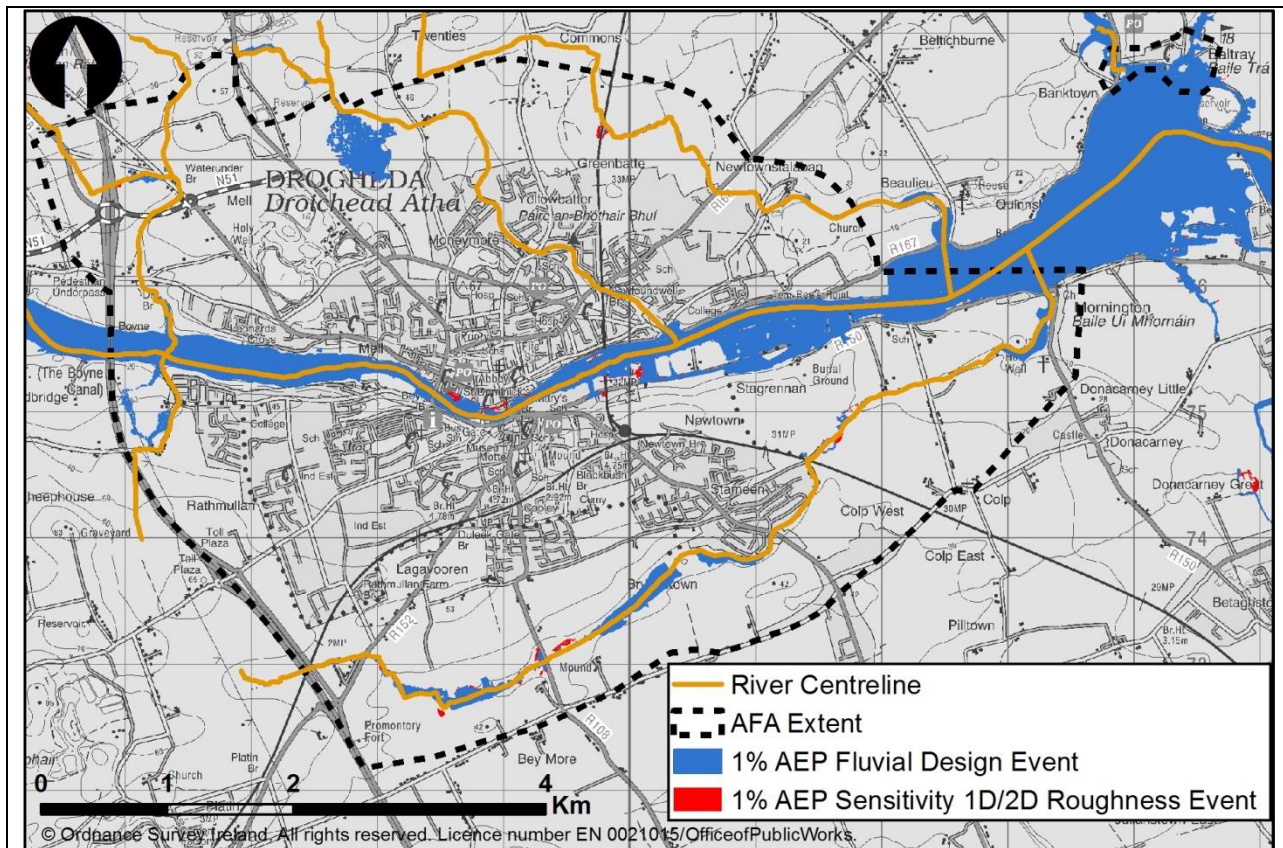


Figure 4.3.33: Comparison between 1% AEP Design Event and 1% AEP Sensitivity 2D Roughness Event

Table 4.3.1 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Drogheda and Baltray model. Overall the parameter assessed, including increasing inflows, downstream water boundary and changing the roughness parameters demonstrates moderate model sensitivity. This moderate expansion of the flood extents has a high impact upon receptors within the Drogheda AFA. When considered separately, the Baltray AFA demonstrated high model sensitivity to increasing the downstream water level boundary, resulting in a high impact to AFA receptors. This is an expected outcome considering that Baltray AFA is within a close proximity to the coast. Baltray AFA demonstrated a low sensitivity to changing the channel and floodplain roughness and increasing model flow.

Table 4.3.1: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event (Drogheda)	Moderate	High
1% AEP Sensitivity 1D/2D Roughness Event (Mornington)	Moderate	Low
1% AEP Sensitivity Water Level Boundary Event (Drogheda)	Moderate	High
1% AEP Sensitivity Water Level Boundary Event (Mornington)	High	High
1% AEP Sensitivity Flow Event (Drogheda)	Moderate	High
1% AEP Sensitivity Flow Event (Mornington)	Low	Low
1% AEP Sensitivity 2D Roughness Event (Drogheda)	Low	Low
1% AEP Sensitivity 2D Roughness Event (Mornington)	Low	Low

4.3.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0600Rp0004_HA07 Inception Report unless otherwise specified):

OCT 2011.	<p>The results of the internet search indicated that the flooding which occurred in Drogheda on 24th October followed a day of heavy rainfall. Both the Boyne and Curly Hole rivers overflowed and floodwaters became contaminated with sewage. Press articles reported that worst hit areas were the Donore Road, Greenhills and John Street, while at one stage one lane of the dual carriageway was completely submerged. The same articles report how the water level was up to the waist level of fire fighters at Drogheda bus station, where the ground level is approximately 6mOD Poolbeg.</p> <p>RPS staff visited Drogheda to collect flood event data on 25th October 2011 and met Drogheda Borough Council staff. It was confirmed that no properties were flooded. Road flooding did occur as outlined above; and surcharged combined sewer manholes at Drogheda Bus Station caused car park flooding in that location.</p> <p>A review of the hydrometric data at the Slane gauging station found that the maximum flow recorded at the gauging station during the event was not a significant flood flow on the Boyne River (flow significantly less than Q_{med}).</p> <p>There are no hourly rain gauges within the catchment; the nearest rain gauge for which data is available located at Dublin Airport, approximately 30km to the south. A review of the hourly data at Dublin Airport found that 67mm fell over a nine hour period on the afternoon and evening of the 24th October. When analysed against the FSU DDF model this can be shown</p>
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to be a rainfall frequency of 1% AEP. The next closest hourly gauge is at Killowen 42km to the north. At this station 40mm was found to have fallen over a 14 hour period which equates to a 50% AEP event. Although rainfall frequency does not exactly reflect fluvial flood frequency, given that some of the tributary catchments are relatively small catchment it is considered the rainfall event may have been of the critical duration for flooding in the tributary watercourses flowing into the Boyne. As such it is estimated that the fluvial return period may have been somewhere between a 50% and a 1% AEP for the smaller, flashier tributary catchments flowing through Drogheda. Assigning a more precise AEP within this range is difficult given the lack of data specific to the small tributary catchments which may have been affected. Whilst there were a number of reports of flooding in and around Drogheda, the event centre is considered to be in the vicinity of Dublin, based on meteorological mapping and the severity of recorded events.

Tidal gauge data recorded at the nearest tidal gauge at Dublin did not indicate that the event was a significant coastal event (less than 50% AEP).

In light of the available hydrometric and meteorological data it is not considered that the event was a significant flood event on the River Boyne or within the Boyne Estuary (coastal). There are no modelled watercourses in the vicinity of the Bus Station at Donore Road / John Street and as such it is considered that the event was pluvial / drainage related at these location. However the Beaulieu Stream does flow through the Greenhills area and the catchment is of the scale at which the rainfall event is likely to have led to the critical conditions for flooding. A review of the mapped extents at Greenhills shows that there is flooding in the 10%, 1% and 0.1% AEP fluvial flood events resulting from the capacity of the Beaulieu Stream culvert being exceeded.

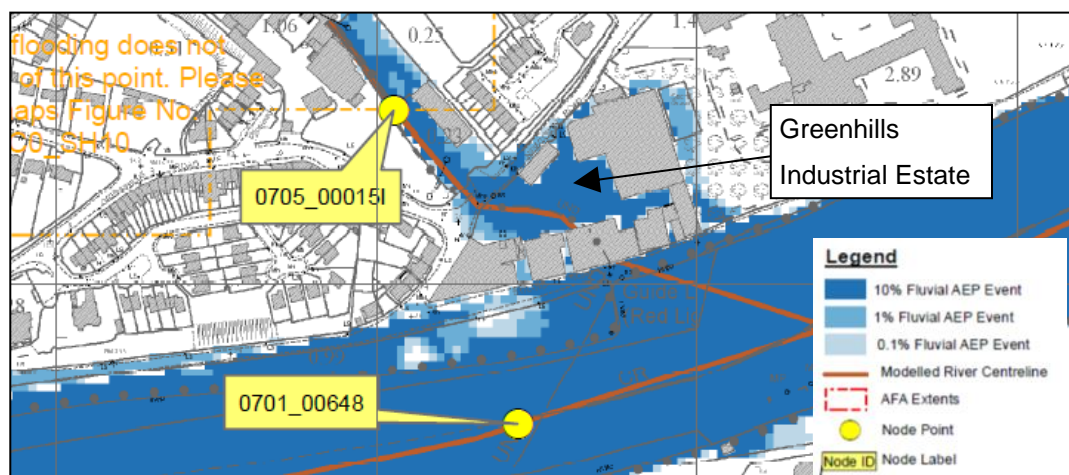


Figure 4.3.34: Modelled Flood Extents at the Greenhills Industrial Estate

The reference to the Curly Hole River is thought to represent a tributary of the Boyne which joins the river at Townley Hall / Oldbridge upstream of the AFA extents. There are no modelled watercourses other than the Boyne at this location and as such it is not considered that the model is capable of replicating this reference to flooding. The reference to rivers

	<p>overflowing may relate to this watercourse at its confluence with the Boyne but the hydrometric data available at Slane just upstream would indicate that it is highly unlikely the Boyne itself overtopped its banks.</p> <p>The flood event data in relation to flood mechanisms that can be clearly defined as fluvial is limited. Furthermore the estimated AEP of the event is of sufficient uncertainty such that the flood event data cannot be considered suitable for model calibration. However the event does partly validate the model in that the model simulates flooding at the location of the recorded fluvial flooding (Greenhills).</p>
OCT 2004.	<p>The historical data indicated that flooding occurred in Drogheda and Baltray in October 2004. This was a coastal flooding event caused by rainfall and high tides/low barometric pressure. However, no details were available on flood extents, damage caused, etc, at either location.</p> <p>Tidal gauge data recorded at Dublin indicated a significant coastal event on the 27th October 2004 with a peak water level recorded of 2.56m OD Malin. This equates to an event just in excess of a 20% AEP coastal event. The information available in relation to the event is not of sufficient detail for calibration however the modelled 10% AEP coastal event does indicate widespread flooding in Drogheda and Baltray and as such provides partial validation of the mapped extents.</p>
NOV 2002.	<p>Information was found on www.floodmaps.ie for a flood event which occurred in Drogheda, Navan, Trim and Edenderry in November 2002.</p> <p>In Drogheda, flash flooding occurred which was a result of heavy rainfall, reportedly exacerbated by blocked gullies. Roads, housing estates and an industrial estate were flooded. However, there were no reports of properties being flooded.</p> <p>A review of the hydrometric data at the Slane gauging station found that the maximum flow recorded at the gauging station during the event was 382m³/s on the 15th November 2002 which equates to a fluvial AEP of approximately 10% AEP.</p> <p>A review of processed rainfall radar data from Dublin Airport (see HA07 Hydrology Report (Appendix B) found that 41.5mm of rainfall fell over 18 hours at Baltray on the 13th and 14st November 2002. Based on the FSU DDF model it is estimated that this rainfall event had a frequency of between 10 and 20% AEP. Although this is not directly comparable to fluvial event frequency it is considered that this may have resulted in a fluvial frequency of up to 10% AEP in the small ungauged catchments in vicinity of the Drogheda and Baltray AFAs.</p> <p>The report consists of a list of flooded locations and these have been compared to the modelled / mapped flood extents in an attempt to verify flooding displayed in the model. A comparison of locations where flooding was reported are shown in Table 4.3.2.</p>

Table 4.3.2: Reported Locations of Flooding

Reported Location of Flooding	Model replicating flooding?		
	10% AEP	1% AEP	0.1% AEP
Railway Bridge at Platin Road	Remote from modelled watercourse		
Bus Depot at Donore Road	x	x	✓
Marsh Road opposite ship street	✓	✓	✓
Congress Ave. At top of watery hill steps	Remote from modelled watercourse		
Rathmullan Park	Remote from modelled watercourse		
Greenhills – Chord Road Junction	Remote from modelled watercourse		
Greenhills – Usshers Junction	✓	✓	✓
Boyne Road (at Premier)	x	✓	✓
Termonfeckin Road (at Boundary)	x	x	✓
Glenview (Pearse Park Junction)	Remote from modelled watercourse		
Brookville Park (at entrance)	x	x	✓
Boyle O'Reilly (down from hospital)	Remote from modelled watercourse		

Six of the locations noted in the report are remote from the modelled watercourses and as such it is fair to assume that the flooding can be considered pluvial / drainage related.

Of the other six locations where fluvial flooding is a potential flood mechanism two (Marsh Road and Greenhills) can be considered to validate the model in that 10% AEP modelled flood extents are consistent with the locations of flooding in November 2002 which is estimated to have been a 10% AEP fluvial event. In the case of the other four events the model indicates flooding but only in more extreme events.

In the case of the bus station at Donore Road a constant high level is provided between the Boyne and the location of the flooding which is only breached in the 0.1% AEP event. However it is likely, given the depression in the topography outside the bus station that pluvial flooding / drainage issues are likely to be a significant contributing mechanism to flooding at this location. Discussion with Louth County Council has also indicated that this is a location that regularly floods due to drainage issues.

There is a similar situation on the Boyne Road outside the premier factory where any reported flood waters reported on the road are likely to arise from the backup of the drainage system when levels in the Boyne are high.

In the case of Brookville Park the flooding was reported at the entrance. Approximately 30m back from the main road there is a depression in the access road and flood water from the Beaulieu Stream culvert inundates this area in the modelled 0.1% AEP event only. However the model does not include the development drainage systems which are likely to include gullies at the location of the flooding. During the 1% AEP fluvial event the culvert is

	<p>surcharged (although there is no out of manhole flooding). In the 1% AEP event, based on the model simulation, it is likely that there would be pluvial flooding as the drainage system would have been unable to discharge given the surcharged Beaulieu Stream culvert. Given that the event is estimated to have had a frequency of approximately 10% AEP and during the simulated 10% AEP event the Beaulieu Stream culvert is not surcharged this may indicate that modelled flooding from the Beaulieu Stream is under predicted. However given that there are other factors which may have led to flooding at this location such as performance of the drainage system or culvert blockage it is considered that the information on the event in isolation is not enough to warrant further calibration of the model and the event provides partial validation of the Beaulieu Stream portion of the model at Brookville Park in that it is replicating the conditions that may have contributed to the observed flooding.</p> <p>At Termonfeckin Road the information is limited. The flooding is noted as having occurred 'at boundary' and no further information is provided. This may refer to the boundary edge of the town where the Termonfeckin Road crosses over the Newtown Stalaban Stream but this is uncertain. At this location out of bank flooding is only evident in the modelled 0.1% AEP event. It is not known if the flooding emanated from road drainage or the river and as such the information is not detailed enough to warrant further calibration of the model or increases in flow to achieve flooding at this location in the 10% AEP model.</p> <p>Generally the information available in relation to the event is not of sufficient detail for calibration however the modelled fluvial events predict flooding at the roads / locations in question and as such the event report provides partial validation of the mapped extents.</p>
OCT 2002.	<p>In Drogheda, flash flooding occurred as a result of heavy rainfall. While the source of the flooding is not clear, it seems that a contributing factor was when sewers were overwhelmed by the rainfall, as indicated by manhole covers being lifted off pressurised systems. Furthermore, with the exception of Ship Street, Rathmullan Road, John Street and Greenhills, the remaining areas listed as being flooded (Donore Road, Platin Road, Marley's Lane, Meadowview, Castlemanor and Elmwood) are sufficiently far away from the river/coast to avoid coastal or fluvial flooding and it is likely these areas were affected due to the drainage network being unable to cope with the rainfall.</p> <p>A review of the hydrometric data for the Slane gauging station (07012) did not indicate that the event was a significant fluvial flood event (less than Q_{med}). A review of the tidal gauge data at Dublin did not indicate that the event was a significant coastal flood event (less than 50% AEP). A review of processed rainfall radar data from Dublin Airport (see HA07 Hydrology Report (Appendix B) found that 81.6mm of rainfall fell over 34 hours at Baltray on the 20th and 21st October 2002. Based on the FSU DDF model it is estimated that this rainfall event had a frequency of between 1% and 0.5% AEP. Although this is not directly comparable to fluvial event frequency it is considered that this may have resulted in a fluvial frequency of up to a 1% AEP event in the small ungauged catchments in vicinity of the</p>

	<p>Drogheda and Baltray AFAs.</p> <p>The information available in relation to the event is not of sufficient detail for calibration however the modelled 1% AEP fluvial event indicates flooding at Greenhills from the Beaulieu Stream. At Rathmullan Road and Ship Street flooding is evident but it arises from the Boyne. Flooding at John Street is only evident in the 0.1% AEP event from the Boyne although this may refer to the junction with Donore Road where flooding is largely drainage related.</p>
FEB 2002.	<p>The historical data indicated that flooding occurred in Drogheda, Baltray, Edenderry and Mornington in February 2002. No details are available of resulting damage in Drogheda.</p> <p>In Mornington, the tide level reached 3.36mOD. Analysis of tidal records in a report entitled "Mornington District Surface Water and Flood Protection Scheme" showed that the coastal flood event in Mornington had an AEP of approximately 1%.</p> <p>There is anecdotal evidence from RPS staff working in the area at the time of the event that sandbags were placed at Drogheda Port and the Quays were flooded.</p> <p>The flood event data in relation to the event is not of sufficient detail such that the event can be considered suitable for model calibration. However the event does partly validate the model in that the model simulates extensive the coastal flooding reported in the 1% AEP event.</p>
NOV 2000.	<p>Information was found on www.floodmaps.ie for a flood event which occurred in Drogheda, Navan, Trim, Baltray, Edenderry and Mornington in November 2000. Floods were caused by heavy rain and storm force winds. In Drogheda, localised flooding occurred due to drainage systems which were blocked and overwhelmed by the rainfall. A number of areas were flooded including Drogheda Quays, Moneymore Housing Estate, Port Oriel Caravan Park and the Bus Depot on Donore Road. Houses escaped flooding during this event.</p> <p>Hydrometric data available at the Slane gauging station (07012) estimated a peak flow on 6th November 2000 of 425m³/s which is estimated at a fluvial frequency of between 3 and 5% AEP. There is flooding at Drogheda Quays in the modelled 10%, 1% and 0.1% AEP fluvial flood event. Significant flooding at the Moneymore Housing Estate from the Beaulieu Stream is not apparent in the modelled scenarios until the 0.1% AEP modelled event. Given the watercourse is culverted through the estate, the culvert arrangement may have since been upgraded and blockage could have been a factor, there is not sufficient evidence in relation to the event to warrant calibration of the model to achieve flooding of the Beaulieu Stream culvert for less extreme fluvial event flood flows.</p>
DEC 1981.	<p>A review of information available on www.floodmaps.ie indicated that a flood event occurred in Drogheda in December 1981. The flood was caused by torrential rain and snow which resulted in the River Boyne overtopping its banks and flooding homes on Ship Street. Press</p>

	<p>articles indicate that dozens of homes flooded, including homes at Church Street and Bothar Brugha. However, due to the freezing temperatures, some flooding of houses was caused by water pipes freezing and bursting and it is not clear how many houses were flooded due to this.</p> <p>In addition to houses being flooded, roads were flooded with cars stranded in Wellington Quay Car Park.</p> <p>A review of the hydrometric data at the Slane gauging station found that the maximum flow recorded at the gauging station during the event was not a significant flood flow on the Boyne River (flow significantly less than Q_{med}). Tidal gauge data is not available for the event. A review of daily rainfall records at Drogheda did not indicate that the rainfall sums which fell during December 1981 were significant (AEP less than 50%).</p> <p>In light of the hydrometric and meteorological data available it is not possible to determine the source of the flooding. The reports are inconsistent with the hydrometric data available at the Slane gauging station and the event cannot be used for calibration or validation of the model.</p>
Jan 2014	<p>Flooding occurred on the 3rd January 2014 which post dates the calibration events defined in the Inception Report. For this event data with a high level of detail was captured and as such it is appropriate that it is considered for model calibration / verification.</p> <p>Significant damage to the town centre and properties in the vicinity of the quays was reported. Approximately 100 residential and 63 non-residential properties were affected. Flood extents during the Jan 2014 event included Greenhills Industrial Estate, Merchant's Quay, North Quay, Wellington Quay, Ship Street and Marsh Road.</p> <p>In Drogheda the Flood Event Report compiled by Louth County Council reported that the event was coastal in nature with a peak flood level recorded of 3.42m OD Malin. Louth County Council reported that this was as a result of a combination of storm surge and high tides. Comparison of this level with the ICPSS extreme water levels puts the frequency of the event at just above a 1% AEP coastal flood event. The flood was reported to have commenced at 11.30am and peaked at 12:30pm before ending at 4:00pm.</p> <p>A review of the hydrometric data at the Slane gauging station found that the maximum flow recorded at the gauging station during the event was not a significant fluvial flood flow on the Boyne River (flow less than Q_{med}).</p> <p>The frequency of the event can be summarised as a coastally dominant flood event of just over 1% AEP. The non dominant fluvial event is less than a 50% AEP. In light of this it would be expected that the modelled flood extents for the 10% AEP coastal event should be less than those observed and the modelled flood extents for the 0.5% AEP event should be greater than those observed. However it must also be considered that drainage issues, largely the impeded ability of pluvial flooding to drain into the Boyne while the levels in the river / estuary were elevated, may be a contributing factor to the observed flood extents.</p>

Detailed maps, together with photographs of the flooding are provided in Figures 4.3.29 - 4.3.38 comparing modelled and mapped extents and an assessment made of the comparison and whether further adjustment of the model was necessary during calibration.



Figure 4.3.35: Merchant's Quay (from the junction with Constitution Hill looking towards rail bridge)



Figure 4.3.36: Merchant's Quay (looking towards rail bridge)



Figure 4.3.37: The Mall (from the junction with Mayoralty Street looking towards rail bridge)

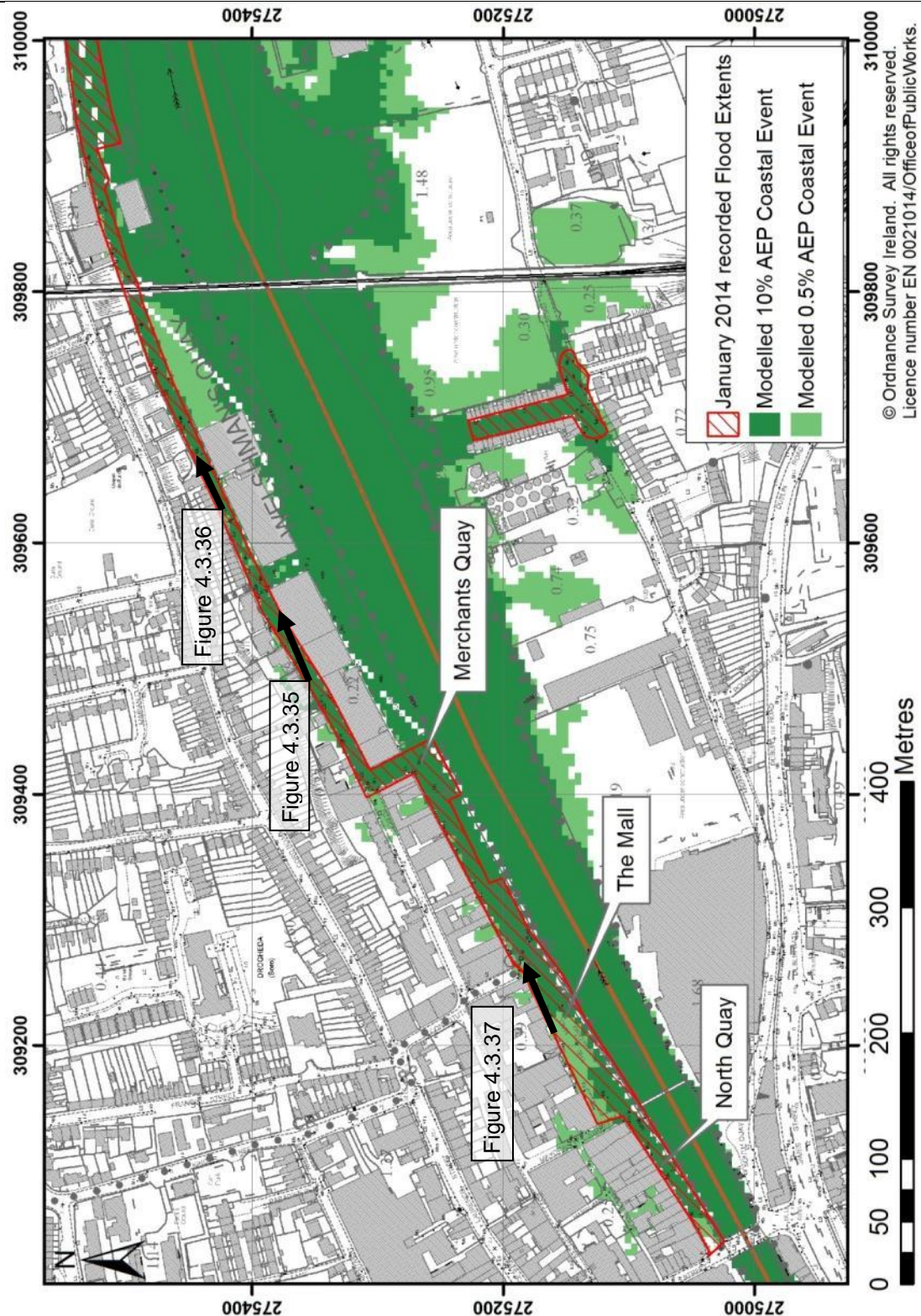


Figure 4.3.38: Mapped Flood Extents at Merchants Quay Comparison with Modelled Extents (January 2014)

A comparison of the modelled flood extents and the recorded flood extents mapped in Figure 4.3.38 demonstrates that the model is replicating the recorded extents in the vicinity of images Figure 4.3.35, Figure 4.3.36 and Figure 4.3.37. The extents generally match the 0.5%

AEP modelled extents in the vicinity of the quays however flooding in this modelled event extends out to streets leading off the quays. The 10% AEP modelled extents are smaller and as such it is considered that extensiveness of the modelled outlines along the quay is validated by the observed flood extents given the estimated event frequency conditions. On the right bank of the Boyne the observed flood extents at Ship Street are lesser than both the 0.5% AEP and 10% AEP modelled events.

This appears to indicate that the model is over predicting the flood extents on the right bank. However it was found that this flood extent in the model is totally controlled by the levels in the 2D domain and adjustment of boundary conditions in order to arrive a better agreement in this location would reduce agreement on the opposite side of the river along the quays. It is possible that the observed flood extents may have been observed at this location after the peak had passed. It was not considered therefore that this warranted further adjustment during calibration of the model.



Figure 4.3.39: Wellington Quay (looking towards St. Dominic's Bridge)



Figure 4.3.40: Wellington Quay from Leyland Place (looking towards St. Dominic's Bridge)



Figure 4.3.41: Upstream End of The Ramparts (looking upstream)



Figure 4.3.42: Downstream end of The Ramparts (looking upstream)

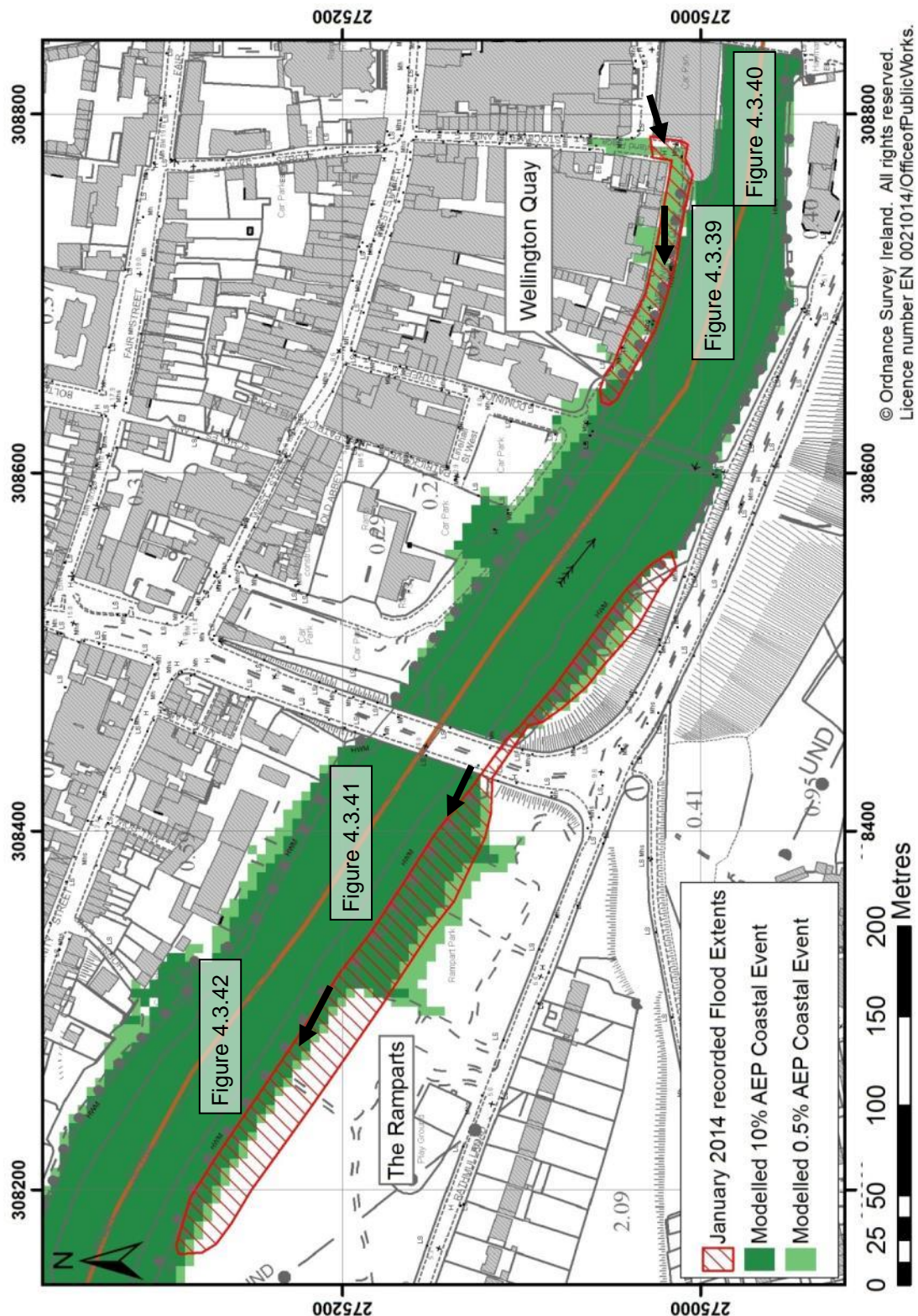


Figure 4.3.43: January 2014 mapped flood extents at Wellington Quay and The Ramparts comparison with modelled extents

A comparison of the modelled flood extents and the recorded flood extents mapped in Figure

	<p>4.3.43 demonstrates that the model is replicating the recorded extents in the vicinity of Wellington Quay. As would be expected the modelled 0.5% AEP event extents are larger and the 10% AEP extents are smaller. Upstream of the Dominic Street Bridge the model simulates flooding which is not within the observed / mapped extents. Along this reach a defence (wall) is in place with a crest height above the flooding however it was obviously breached downstream of the bridge. It may be the case that the section of wall upstream of the Dominic Street Bridge was not breached during the January 2014 event although this could not be confirmed. This wall is not represented in the model because it is considered this defence is ineffective (see 0 (3)). It is also notable that the eastern end of Wellington Quay was breached via the drainage system (see Figure 4.3.40) which may not have occurred upstream of the bridge. It is considered that the model is validated at Wellington Quay by the event information.</p> <p>Along the Ramparts (also referred to as Dominic's Park) the modelled extents along the reach adjacent to the bridge are again validated in that the 10% AEP modelled extents are smaller and the 0.5% AEP modelled extents are larger. Along the portion of the walkway upstream however the recorded extents are smaller than either of the modelled events. However examination of the photographs indicates that flooding is extending beyond the footway adjacent to the bridge and restricted to the footway further upstream. It is therefore concluded that the modelled outlines represent this more detailed reflection of the topography. To consider the observed extents the mapped outlines and photos must be taken together and at all locations along Wellington Quay and The Ramparts these validate the model when the estimated frequency of the event is taken into account.</p>
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Comparison with ICPSS

A comparison was made between the modelled 0.5% AEP extents and the ICPSS 0.5% AEP extents as shown in Figure 4.3.44.

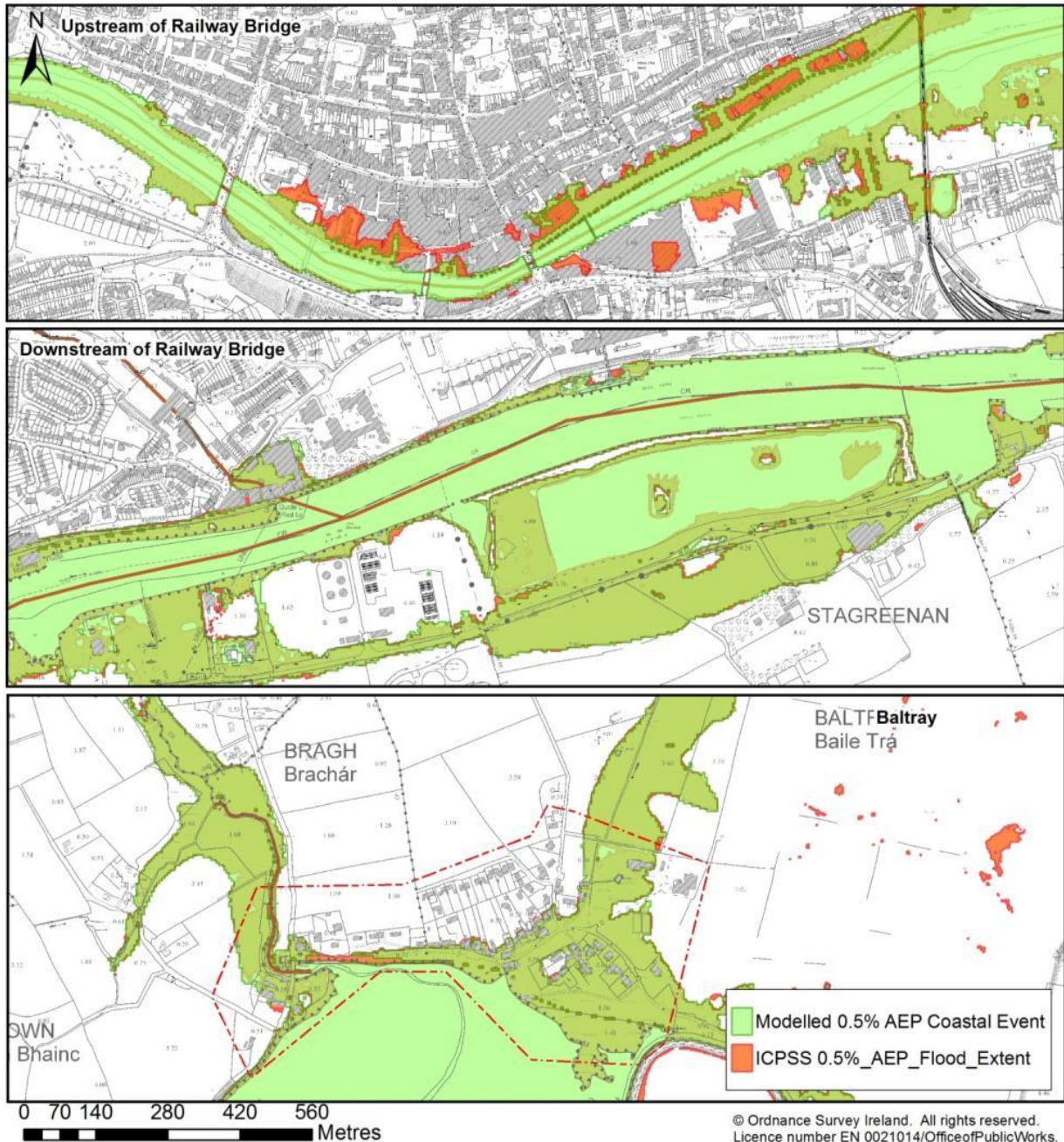


Figure 4.3.44: Comparison of ICPSS Extents with Modelled Extents for 0.5% AEP event

It can be seen that the modelled flood extents and the ICPSS mapped extents are in good agreement in the lower Boyne Estuary and at Baltray. However in the centre of Drogheda there are significant differences between both outlines with the ICPSS outlines showing significantly more flooding in the centre of Drogheda. This is considered to be arising due to how the ICPSS flood extents are mapped.

The water levels are projected onto a digital terrain model and as such the ICPSS is not capable of taking into account barriers / restriction to flow inland and temporal effects. In the case of the centre of Drogheda, propagation of coastal flood waters up the Boyne Estuary is likely to be partially restricted by the bridges which are represented in the 1D portion of the model and other raised areas represented in the 2D domain. Propagation of flow inland is not modelled in ICPSS and as such the CFRAM Study model represents a more detailed analysis of inland flooding.

Summary of Calibration

The model replicated recorded flood extents where sufficient data was available; this was primarily along the quay area of Drogheda during an extreme tidal event. Much of the additional information relating to flooding in Drogheda indicated a likely flooding mechanism of minor drainage systems being overcome during extreme rainfall events. The model does not contain information on the minor drainage infrastructure and as such much of the historic flooding cannot be replicated.

The mass balance assessment of the model is within acceptable bounds with a mass error of 0.1% during the 0.5% AEP coastal event and a mass error of 0.0% during the 1% AEP fluvial event. There are number of warnings within the model log files as is common with models of this size and complexity. The warning which appears most frequently is 'invert level above ground level'. This occurs at culvert inlets and outlets where the invert of the conduit protrudes from the side of a bank into the watercourse channel. This is realistic and as such it is not considered that the model is adversely affected. All warnings have been reviewed and it is not considered that the accuracy of the model is adversely affected.

As a result of draft mapping review workshops, Local Authorities provided information on past flood events that contributed further to the model calibration. In light of the review, condition survey and response to flooding which occurred during the January 2014 event a wall which had previously been deemed as formal effective was determined to be ineffective and removed from the model in the Baltray area.

(2) Post Public Consultation Updates::

All recorded comments were investigated following informal public consultation and formal S.I. public consultation periods in 2015 but this did not result in a requirement to update the model in relation to Drogheda or Baltray. However, further information supplied by Louth County Council in an email including photos indicated a flood event on John Street, Drogheda in November 2010. Louth County Council estimated that ten properties were affected. A review of hydrometric data for the Slane gauging station on the Boyne did not indicate that the event was a significant fluvial event with the maximum flow recorded in November 2010 significantly less than Q_{med} , the median annual peak flood flow. A review of the nearest rainfall gauge, the daily station at Bellewstown 8km to the south of John Street did not indicate an extreme rainfall event during November 2010 although significant sums were recorded. It is therefore assumed that the flooding was in some way related to the drainage system. In reviewing the event RPS looked at the potential for fluvial flooding to reach properties along John Street and in doing so it was decided that an improved representation could be made by moving bank lines along this reach from the boundary wall defining the channel edge to the higher ground on John Street. This allows flooding to propagate more easily from the river channel to John Street during extreme flood events and was considered a more accurate representation of the flooding to the rear of the properties along John Street which back onto the river.

An amendment was made to the inflows affecting one of the tributaries to the Mornington River which is included within the same hydraulic model but is reported in relation to the Mornington AFA (see Section 4.7 of the HA07 Hydrology Report (IBE0600Rp0025_HA07_Hydrology Report)). This has no bearing on any of the watercourses affecting Drogheda or Baltray however the models have all been re-run as a result. A revised set of flood hazard and risk mapping has been issued as Final to reflect these changes.

(3) Standard of Protection of Existing Formal Defences:

A flood defence data collection exercise was undertaken to locate possible flood defence structures and assess their condition. This information has been compiled in a Defence Asset Database and this has been used as the basis for assessing whether areas of the AFAs can be considered defended. Extracts from the Defence Asset Database relating to the Drogheda and Baltray AFAs are shown in Figure 4.3.45 and Figure 4.3.46.

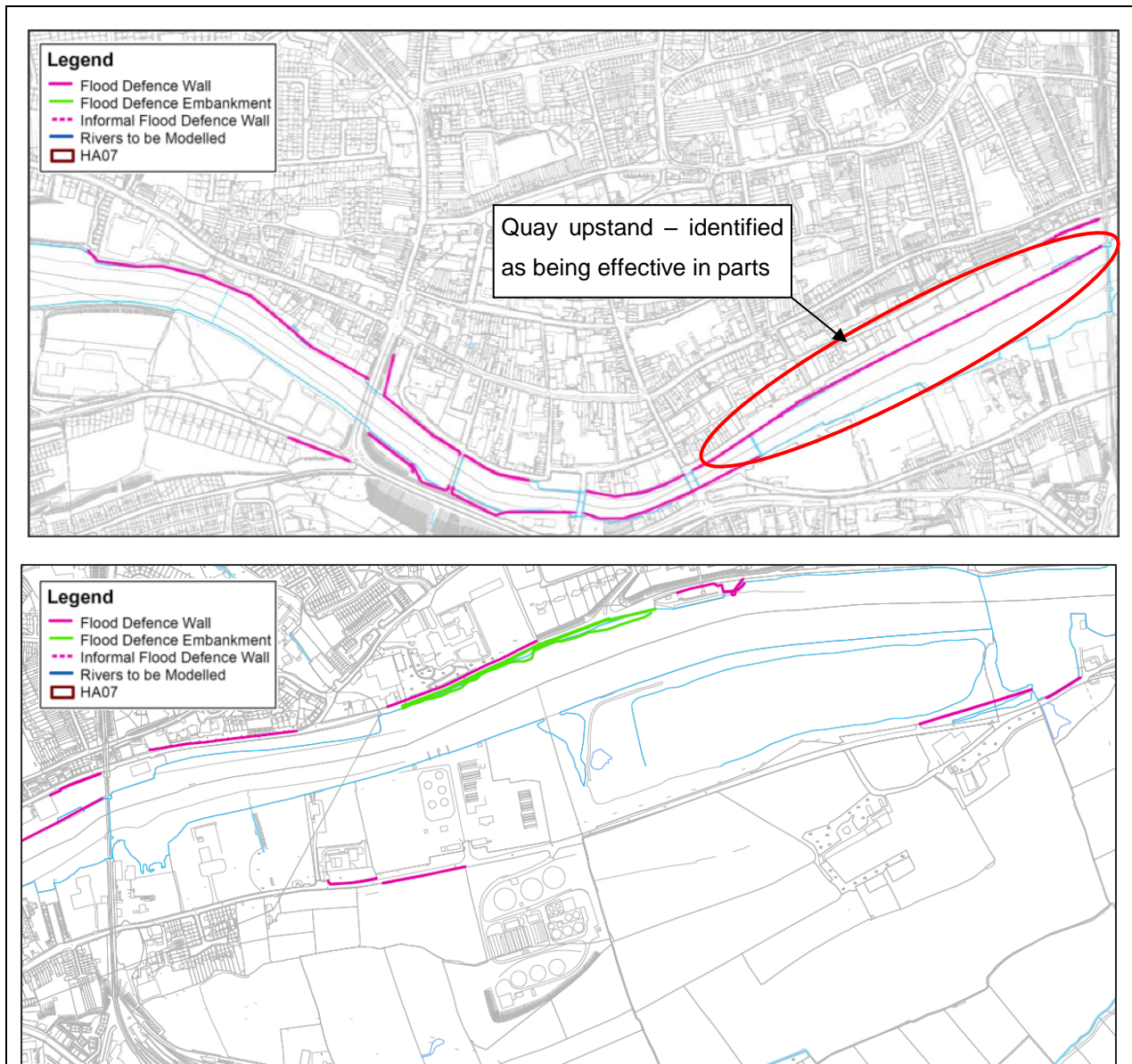


Figure 4.3.45: Potential Flood Defences identified in Drogheda

A large number of defences along the quays in Drogheda were identified as potential defences. The defences identified took the form of the quay wall, quay wall upstands, earthen embankments, property boundary walls and vertical walls.

Following discussions with Louth County Council and based on the experiences of the January 2014 flood event it was considered that all of the defences, with the exception of the low level upstand on the North Quay, could be considered ineffective. These defences were found to be either not providing continuous protection, were in a poor condition or were compromised by drainage provided through them.

In the case of the low level upstand on the edge of the quay initial modelling found that this defence does not provide continuous protection along the North Quay and is reliant on property walls to provide a continuous level of protection along the North Quay. In light of this all of the defences identified in Drogheda were considered ineffective and no defence structures were represented within the Drogheda

AFA portion of the model.

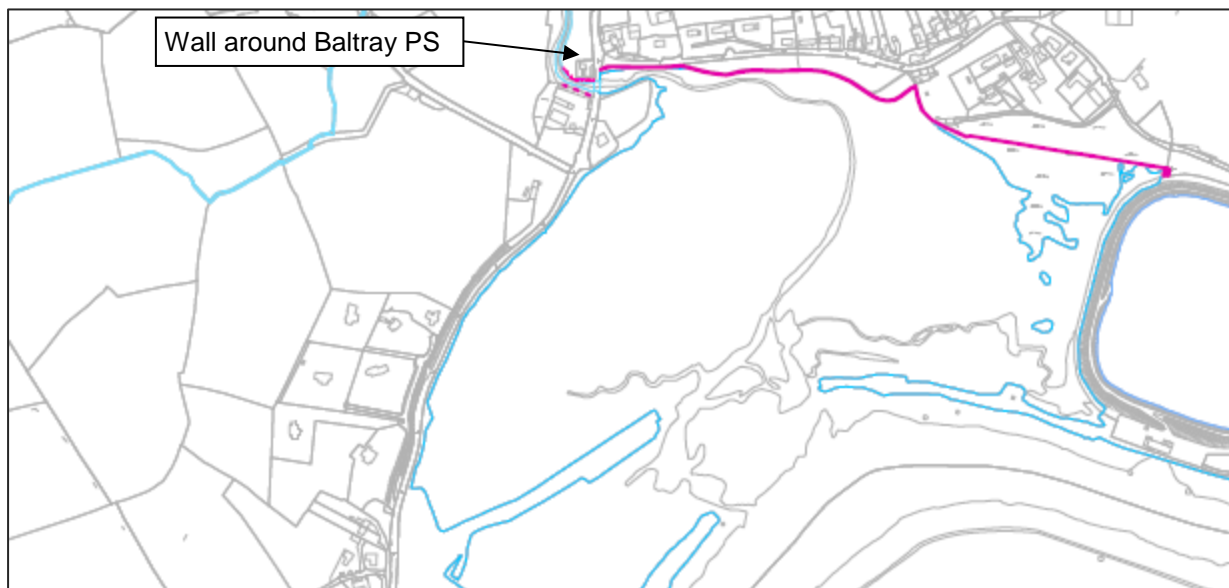


Figure 4.3.46: Potential flood defences identified in Baltray

Three defences were identified within Baltray.

The wall along the frontage at Baltray was assessed to have several issues in relation to its potential effectiveness. The masonry walls were found to be in poor condition with cracks and porous rubble embankment sections with uncertainty surrounding the tie in between wall and embankment sections. Some drainage outfalls were also found to be unflapped and there was generally considered to be uncertainty surrounding the structural integrity of the defence. As such the defence was considered to be ineffective and is not represented in the model.

A further section of flood defence wall on the left bank of the Baltray Stream surrounding the sewage pumping station upstream of Baltray Bridge was found to be in good condition and in itself could be considered effective. However initial modelling found that with the wall along the frontage at Baltray considered ineffective the sewage pumping station is not protected as a flood route develops across the road in front of the pumping station site for all of the modelled event scenarios and therefore the minimum standard of protection is not provided. As such the walls were not included in the model.

A third defence on the right bank of the stream opposite the pumping station in the form of a property boundary wall was identified as potentially providing protection to a cottage and outbuildings. The rubble masonry wall of the outbuildings forms the defence however given that the structural integrity of the wall for the purposes of flood defence is uncertain and, consistent with the approach of treating informal boundary walls as ineffective, the defence was not included in the model.

(4) Gauging Stations:

Two gauging stations are located within the modelled reach; however both are located within the MPW section of the model. One is located at Slane Castle (HEP 07012_RPS) and the other is located at

Roughgrange (HEP 07059_RPS). Neither gauging station has been subject to rating review as part of the Study.

Good correlation was achieved between the spot gaugings taken at the Slane Castle gauging station with the Q-h relationship from the model. The weir downstream of the gauging station acts as a control structure, an adjustment to the discharge coefficient of the weir from the default 1.7 to 1.8 was applied to achieve a better relationship between the observed and modelled data.

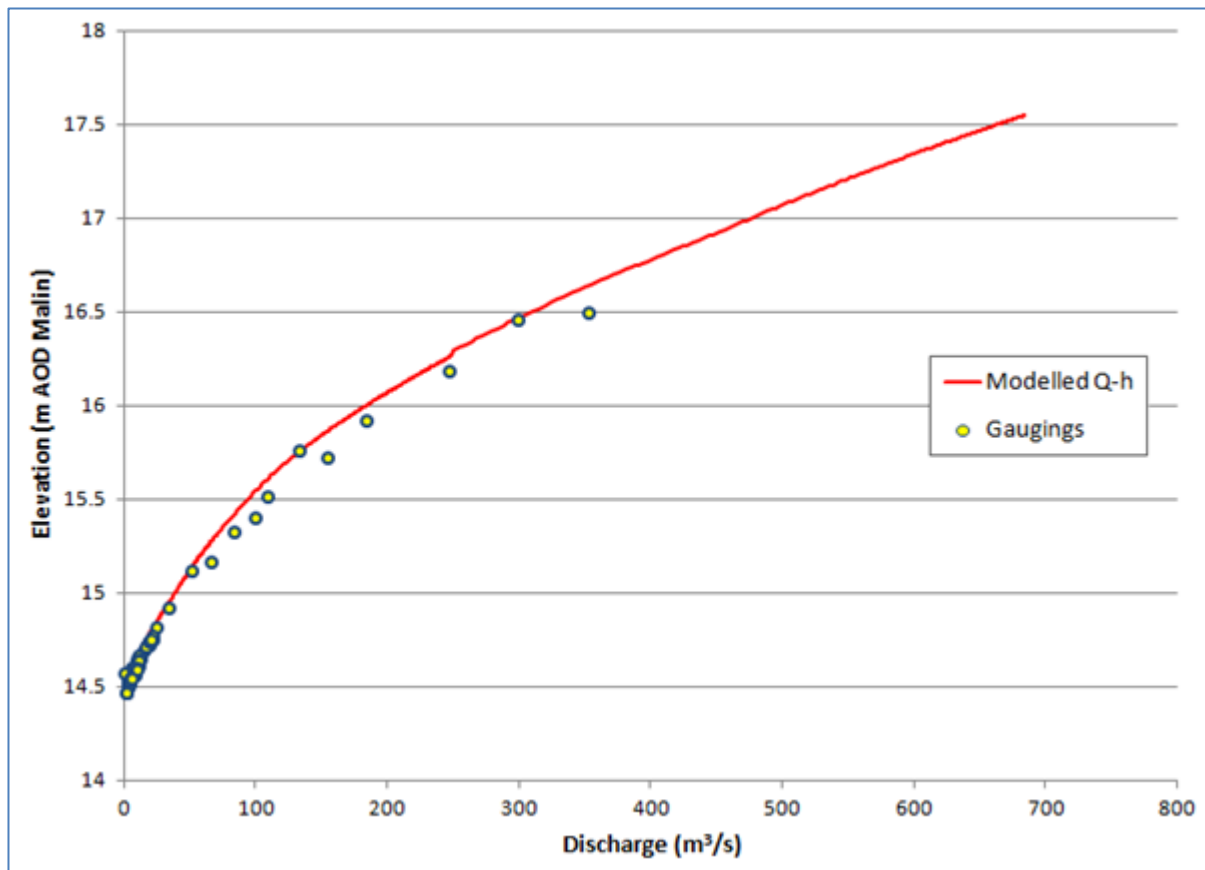


Figure 4.3.47: Comparison of spot gauging at Slane (7012) with the modelled Q-h

As can be seen from Figure 4.3.47 the modelled Q-h relationship is well matched to the spot gaugings on the upper edge of the mild scatter that can be observed within the range. It is therefore considered that this MPW reach of the model is fairly well calibrated to observed data up to flows beyond the Q_{med} flow.

The other gauging station is located approximately 6km downstream and is called Roughgrange (07059 – EPA). This station has a short record length with gaps in the record. The station was not given a classification of its rating under FSU and no information was provided by EPA on the rating of the station. As such it has been assumed that there is little confidence in the rating and the gauging station cannot therefore be used for calibration or validation of the modelled Q-h relationship.

(5) Validation with MIKE NAM:**(a) Slane (7012)**

It can be seen from Table 4.3.3, there is observed data available for all eight historical flood events. The difference between the modelled peak water level achieved at the peak discharge estimated by MIKE NAM and the observed peak water level (where available) is provided in Table 4.3.3. The difference is less than 330mm for each event. This comparison is provided for information purposes only, and has not been used during the calibration or validation of the model. Model calibration is based on observed data, which takes precedence over the data provided from the NAM hydrological model. Further details on the NAM model are provided in the Hydrology Report.

Table 4.3.3: Comparison of Observed, Hydrologically (NAM) and Hydraulically Modelled Data for Flood Events

	Observed Peak		MIKE FLOOD Simulated Peak at Observed WL	MIKE NAM Simulated Peak	MIKE FLOOD Simulated Peak at NAM Discharge	Water Level Difference
Flood Event	Water Level (mOD)	Discharge (m ³ /s)	Discharge (m ³ /s)	Discharge (m ³ /s)	Water Level (mOD)	(m)
11 th Jan 2014 (Predominantly Coastal Event)	15.31m	119.9m ³ /s	72.4m ³ /s	-	-	-
24 th Oct 2011	16.00m	205.2m ³ /s	184.2m ³ /s	-	-	-
27 th Oct 2004 (Predominantly Coastal Event)	15.13m	65.5m ³ /s	51.4m ³ /s	66.7m ³ /s	15.35m	+0.24m
15 th Nov 2002	16.60m	384.8m ³ /s	341.6m ³ /s	291.3 m ³ /s	16.44m	-0.16m
22 nd Oct 2002 (Unknown Flood Mechanism)	15.70m	136.7m ³ /s	124.8m ³ /s	190.7 m ³ /s	16.03m	+0.33m
26 th Feb 2002 (Possible Coastal Event)	16.29m	283.0m ³ /s	249.8m ³ /s	176.5 m ³ /s	15.97m	-0.32m
6 th Nov 2000	16.71m	424.9m ³ /s	378.5m ³ /s	279.8m ³ /s	16.40m	-0.31m
Dec 1981 (Unknown Flood Mechanism)	15.88m	152.9m ³ /s	158.0m ³ /s	110.4m ³ /s	15.62m	-0.26m

(6) Comparison of Flows:

Table 4.3.4 provides details of the flow in the model at every HEP intermediate check point, and modelled tributary. These flows have been compared with the hydrology flow estimation and a

percentage difference provided.

Table 4.3.4: Modelled Flows and Checked Flows

River Name & Chainage	Peak Flows			
	AEP	Check Flow (m3/s)	Model Flow (m3/s)	Diff (%)
River Boyne 0701 - 02816 07012_RPS	50%	247.9	248.1	0.1%
	10%	350.3	350.3	0.0%
	1%	497.0	496.9	0.0%
	0.10%	683.7	683.4	0.0%
River Boyne 0701 - 02405 07_1057_6_RPS	50%	252.9	249.7	-1.3%
	10%	357.4	352.6	-1.3%
	1%	507.1	500.4	-1.3%
	0.10%	697.6	688.5	-1.3%
River Boyne 0701 - 02074 07059_RPS	50%	254.4	251.1	-1.3%
	10%	359.4	354.5	-1.4%
	1%	510.0	503.1	-1.4%
	0.10%	701.6	692.3	-1.3%
River Boyne 0701 - 16650 07_1105_2_RPS	50%*	258.28	254.7	-1.4%
	10%	364.95	360.4	1.5%
	1%	517.85	513.2	-0.9%
	0.10%	712.33	709.0	1.1%
River Boyne 0701 - 140 07_1894_2_RPS (Downstream Boundary)	50%*	277.14	288.3	4.0%
	10%*	391.6	n.a.	n.a.
	1%*	555.67	586.4	5.5%
	0.1%*	764.36	n.a.	n.a.
Baltray River 0702 - 0 07_2_2	50%	4.02	4.23	5.2%
	10%	7.24	7.33	1.2%
	1%	13.43	13.61	1.3%
	0.10%	24.03	24.73	2.9%
Unknown Trib 0703 - 4340 07_472_8	50%	0.9	0.74	-17.8%
	10%	1.62	1.00	-38.3%
	1%	3.03	1.25	-58.7%
	0.10%	5.49	1.60	-70.9%
Unknown Trib 0703 - 0 07_472_16	50%	1.75	1.56	-10.9%
	10%	3.11	3.43	-10.3%
	1%	5.71	3.95	-30.8%
	0.10%	10.16	5.88	-42.1%
Newtown Stalaban Stream 0704 - 0 07_1909_1	50%	0.84	0.78	-7.1%
	10%	1.52	1.47	-3.3%
	1%	2.85	2.76	-3.2%
	0.10%	5.17	4.65	-10.1%

Beaulieu Stream 0705 - 0 07_1906_3	50%	2.58	2.39	-7.4%
	10%	4.67	5.15	-10.3%
	1%	8.74	9.66	-10.5%
	0.10%	15.85	16.76	-5.7%
Dry Bridge Stream 0706 - 0 07_1902_5	50%	1.34	1.22	-9.0%
	10%	2.42	2.19	-9.5%
	1%	4.52	4.11	-9.1%
	0.10%	8.2	7.06	-13.9%
Dry Bridge Stream Trib 0707 - 0 07_1902_1	50%	0.2	0.19	-5.0%
	10%	0.36	0.40	11.9%
	1%	0.67	0.65	-3.0%
	0.10%	1.22	0.90	-25.9%
Unknown Trib 0708 - 0 07_1904_3	50%	0.59	0.61	3.4%
	10%	1.06	1.07	0.9%
	1%	1.98	2.01	1.5%
	0.10%	3.6	3.66	1.7%

Note*: Modelled flows extracted from model simulation undertaken with a constant water level boundary of 1m such that the effect of tidal flows within the downstream reaches could be discounted and a direct comparison with the hydrological estimates of peak flow made.

07_1105_2_RPS - MPW HEP just upstream of the Drogheda AFA, the check flows and model flows match very well at this location.

07_1894_2_RPS - The downstream HEP of the River Boyne, this location is tidally dominated and as such the peak flows are significantly different to the check flows at this location.

07_2_2 - The downstream HEP of the Baltray River, the check flows and model flows match well at this location.

07_472_8 - The intermediate HEP on the un-named 0703 watercourse, there is a large difference between the check flows and the model flows at this location. This discrepancy is due to an under capacity culvert downstream of the HEP location causing a large attenuation effect along this reach with the peak inflow being greater than the peak outflow and a resultant truncated and elongated outflow hydrograph as shown in the Figure 4.3.48.

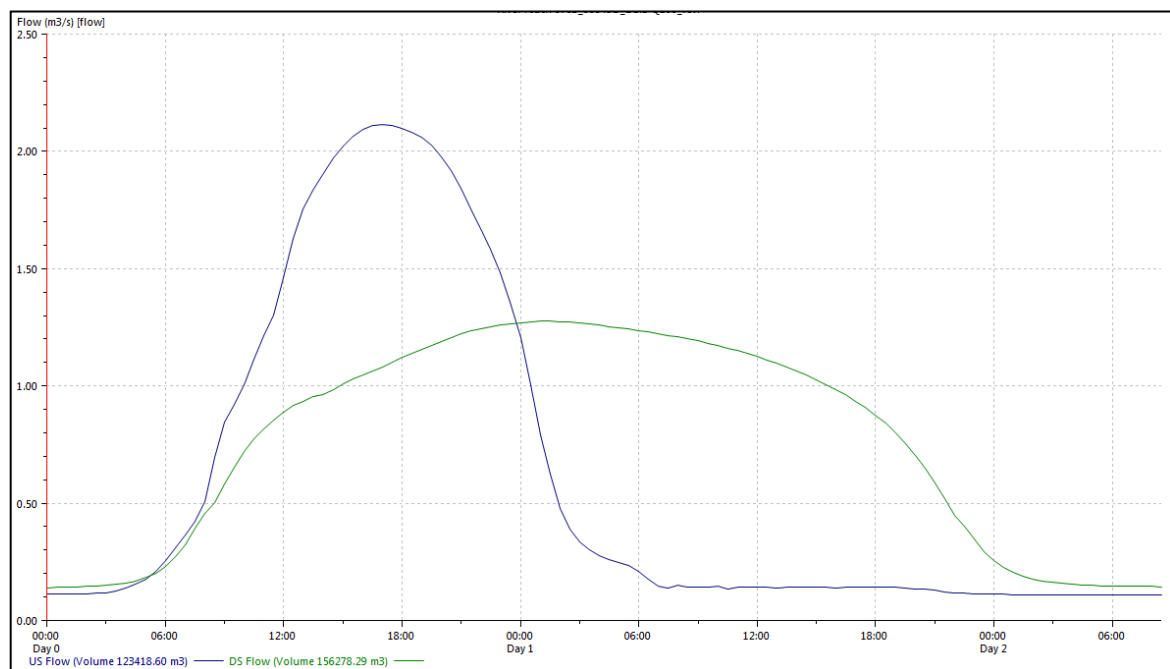


Figure 4.3.48: Comparison of flow upstream and downstream of culvert ID 0703_00377 on the Boyne Trib 2 watercourse

07_472_16 - The downstream HEP on the un-named 0703 watercourse, there is a large discrepancy between the check flows and modelled flows at this location due to the attenuation in the vicinity of HEP 07_472_8 as discussed above.

07_1909_1 - The downstream HEP on the Newtown Stalaban Stream, there is good agreement between the check flows and the modelled flows. Flooding and subsequent pondage during the 0.1% AEP event has increased the percentage difference.

07_1906_3 - The downstream HEP on the Beaulieu Stream, there is reasonable agreement between the modelled and check flows at the more frequent return periods. There are significant hydraulic effects occurring along this reach which flows through a heavily urbanised area which would not be represented within the hydrological estimates.

07_1902_5 - The downstream HEP of the Dry Bridge Stream, there is reasonable agreement between the check flows and modelled flows.

07_1902_1 - The downstream HEP of the Dry Bridge Tributary, there is reasonable agreement between the check flows and the modelled flows.

07_1904_3 - The downstream HEP of the un-named 0708 watercourse, there is good agreement between the check and modelled flows.

(7) Other Information

(a) Out of bank flooding from the River Boyne along the MPW reaches is largely confined by the steep gradients along the river banks during flood events. Out of bank flooding occurs adjacent to the Taaffe's Loch, Broadboyne Bridge and the Roughrange areas. During a 1% AEP fluvial flood event there are six properties at risk and there are two properties at risk during a 0.5% AEP coastal flood event. During a 0.1% AEP fluvial flood event there are eight properties at risk and there are two properties at risk during a 0.1% AEP coastal flood event. More detail of the approach to modelling the MPWs in Drogheda is presented in Chapter 3.

4.3.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

- (a) Please refer to Chapter 3.3.2 for general assumptions using the Infoworks ICM modelling software.
- (b) In channel roughness values have been selected based on normal bounds values which have been reviewed during the calibration / verification process.
- (c) All culverts and bridges have been assumed to run at full capacity (clear of blockages) during design events.
- (d) Draft Final mapping assumed all formal and informal defences are ineffective (see 0 (3) for further details).
- (d) No specific afflux information is available for calibration of headloss across bridges, as such all bridge coefficients have been set at default values.
- (e) Culvert data from section 0708_00044 was not available from Drogheda Sewer Network Drawings, culvert levels/gradient from manhole 0708_MH4004 to section 0708_00016 have therefore been assumed to have a linear slope.
- (f) The manhole and culvert information contained in the Drogheda sewer network drawings was assumed to be correct.
- (g) No drainage networks have been included in the model; as such flows have been introduced directly to the 1D domain as point or lateral inflows as determined in the hydrological analysis. Hydrological analysis of the drainage networks is limited to their use in delineation of the urban catchment boundaries.

(2) Hydraulic Model Limitations and Parameters:

- (a) Road and street networks have been defined by the inclusion of building polygons to restrict flow paths to between buildings, but road profiles have not been specifically embedded in the 2D mesh.
- (b) A mesh resolution of 1m^2 to 25m^2 has been applied.
- (c) There are no major instabilities within the model.
- (d) Only flooding from the channels included in the model has been considered. Flooding from backing up of minor drainage systems has not been considered.
- (e) A model co-efficient of discharge (C_d) across bank lines of 0.8 has been used throughout the model. This is considered appropriate through the urban centre of Drogheda as the bank lines represent a mixture of boundaries including quay walls with railings and fences and ineffective walls.

(3) Design Event Runs & Hydraulic Model Handover Notes:

The main source of fluvial flood hazard from the Boyne River is from the out-of-bank flooding of the built up areas of the Drogheda AFA directly adjacent to the main channel. The topography of the AFA is such

that the floodplain is set within a well-defined valley between high ground to the north and south of the AFA. As such flooding is confined to the streets adjacent to the river and flood extents do not extend to areas distant from the river channel.

The first out of bank spilling of flood waters into the town centre is evident in the 10% AEP fluvial flood simulation on the left hand bank of the reaches upstream of St. Mary's Bridge where the channel narrows. At Wellington Quay flood water enters the town but is restricted to the road adjacent to the river.

During the 1% fluvial event simulation flooding is more extensive along Wellington Quay with depths of up to 1m along the quay and extends into Stockwell Lane and car parking areas at Preston Drive where flood depths of up to 0.5m are simulated. Flooding during the 0.1% AEP fluvial event is much more extensive and extends along Dyer Street to Shop Street and a large area between Dominic Street and Stockwell Lane, see Figure 4.3.49. This area of the town to the north of the river represents a slight depression in the topography and it is considered that flooding can only be relieved through the functioning of the urban drainage system which is not included within the model. However this would be expected to occur once the fluvial peak has passed and flood duration would be expected to be in the order of 12 – 18 hours.

Further downstream along North Quay and Merchant's Quay flooding is also evident in the 1% AEP fluvial event of up to 0.3m.

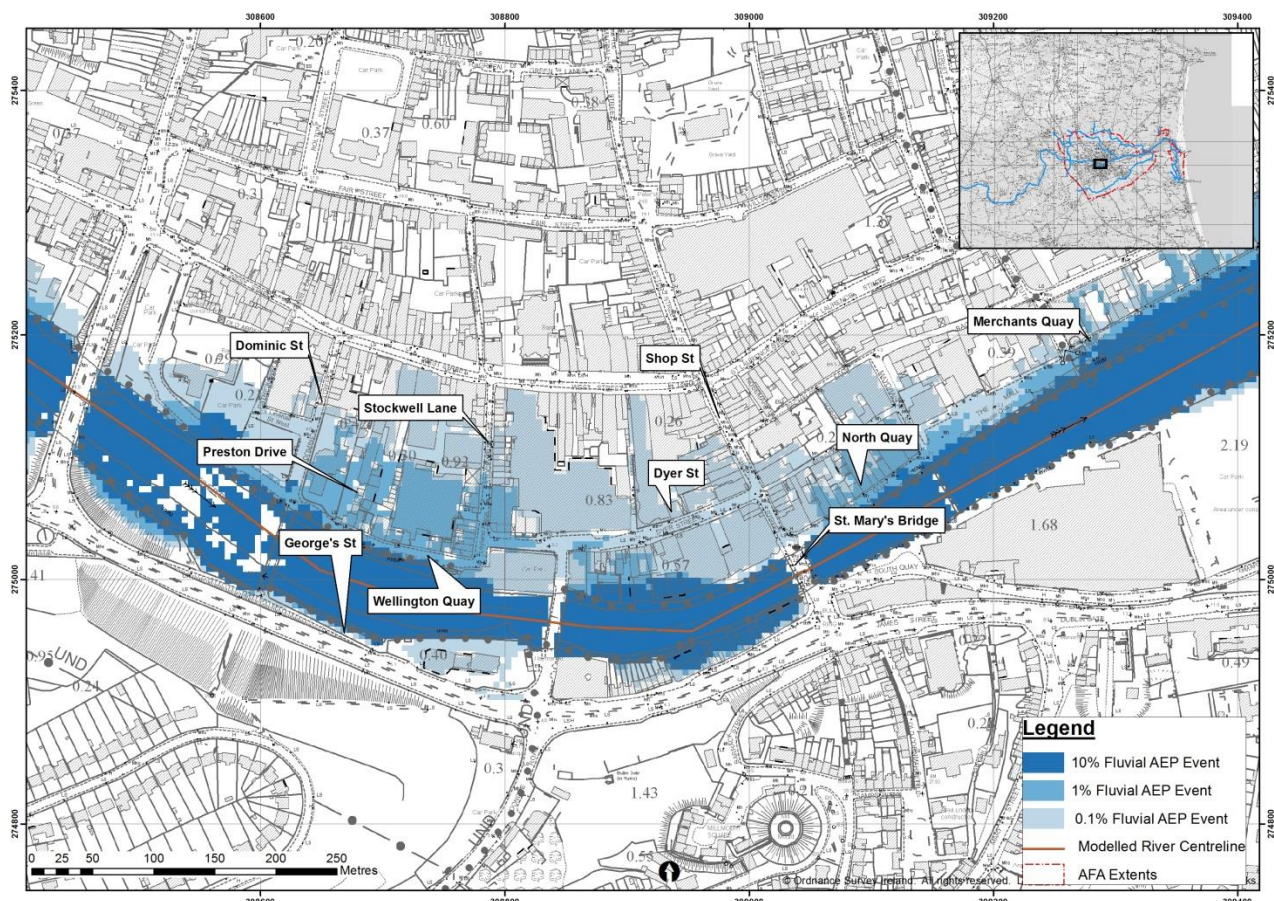


Figure 4.3.49: Modelled Fluvial Extents in the St. Mary's Area

Downstream of St Mary's Bridge coastal flooding is the dominant flood mechanism and flood water enters Merchant's Quay and North Quay during the 10% AEP simulation. Flooding of large swathes of the

commercial / industrial areas downstream of the town centre served off North Strand and Marsh Road on both banks of the river is simulated in the 10%, 1% (0.5% AEP for coastal event) and 0.1% AEP fluvial, see Figure 4.3.50, and coastal scenarios, see Figure 4.3.51. This area is much flatter as the Boyne flows towards the coastline at Mornington and Baltray. Coastally dominated flooding generally coincides with the peak extreme water level on the high tidal cycle and may persist over as many as three subsequent high tidal cycles.

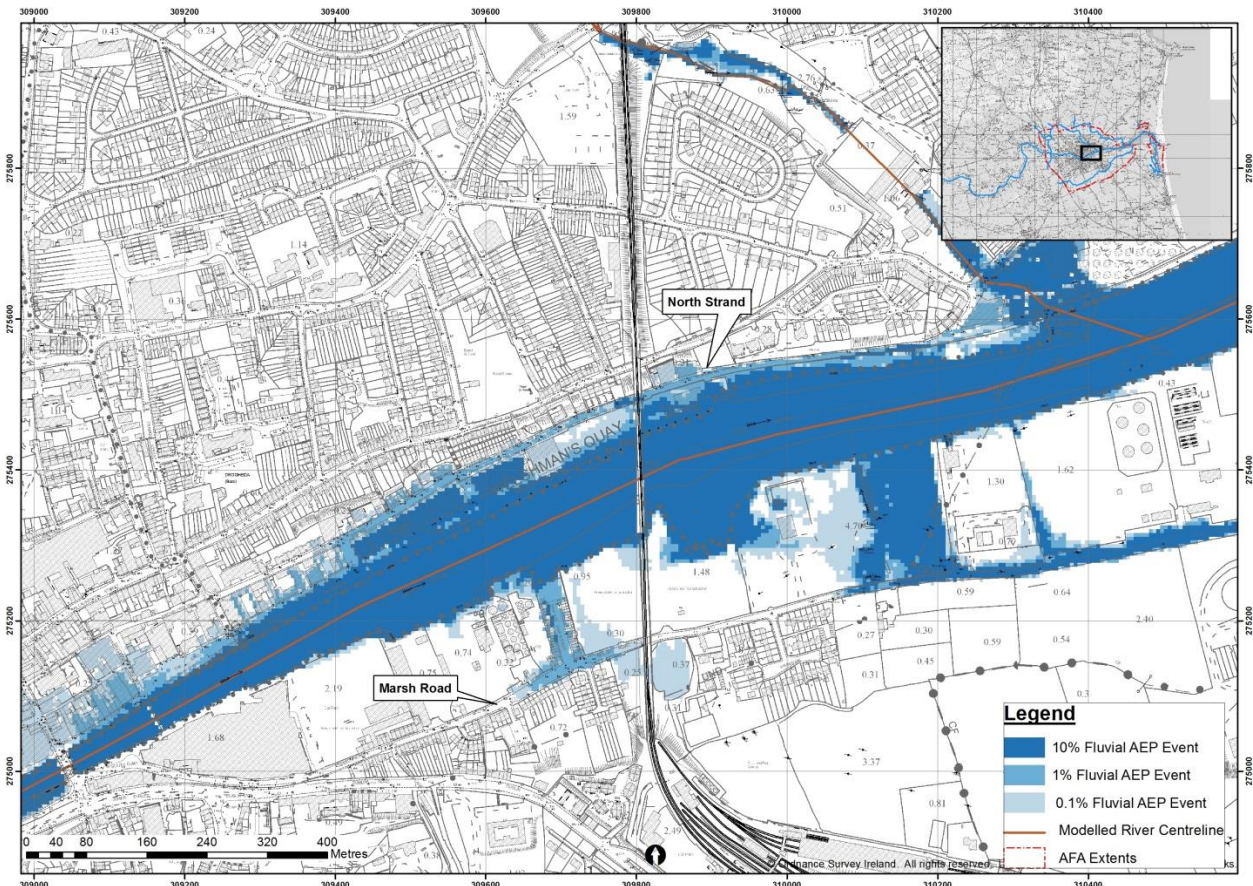


Figure 4.3.50: Modelled Fluvial Extents in the Marsh Road Area

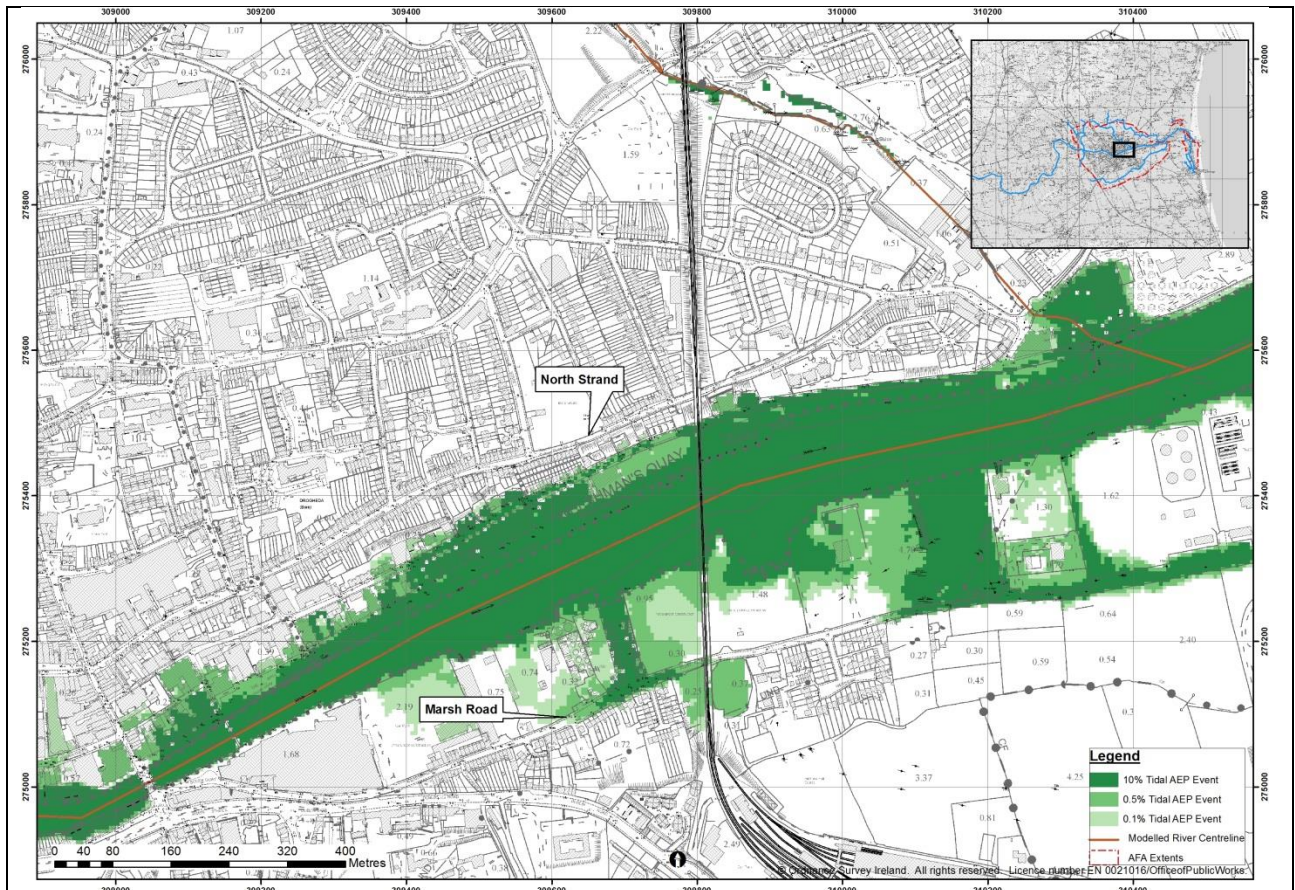


Figure 4.3.51: Modelled Coastal Extents in the Marsh Road Area

Mornington Bank initially provided the boundary between the 1D and 2D domains. This initial modelling approach resulted in instability and excessive velocities at Mornington Bank. It was also considered that the representation of Mornington Bank in the 1D by bank levels defined at cross section spacing may not be accurately representing this critical flood spill location. In light of this the 1D / 2D boundary was moved back into the estuary such that Mornington Bank could be fully defined by the LiDAR within the 2D domain. A finer mesh zone was specified as the topography here is critical to one of the main coastal flood mechanisms in relation to Mornington.

In addition to the flood hazard emanating from the Boyne in relation to the Drogheda AFA there is also significant flood hazard emanating from the Beaulieu Stream and the Drogheda Boyne Unknown Tributaries 1 & 2. This hazard generally emanates from constrictions at culvert / bridge structures and is discussed in greater detail in relation to these structures in Section 4.3.3 (1).

In relation to the Baltray AFA the main flood hazard emanates from extreme coastal water levels leading to flooding along the frontage and the lower reaches of the Baltray Stream. This flooding is simulated in all of the modelled scenarios however is greatest in the coastally dominant design simulations.

Hydraulic Model Parameters:	
1D Domain	
Timestep (seconds)	1
Min / Max space step	0.1m / 25m
Max Timestep Halvings	30
Max Iterations	30
2D Domain	
Timestep (seconds)	Dynamic
Timestep Stability Control	0.95
Maximum Velocity	10m/s
Theta	0.9
Inundation Mapping Depth Threshold	0.01m
(4) Hydraulic Model Deliverables:	
Model deliverables are supplied in an accompanying InfoWorks ICM transportable database containing all model files as required by the brief and the relevant network and event files.	
(5) Quality Assurance:	
Model Constructed by:	Andrew Sloan
Model Reviewed by:	Stephen Patterson/Brendan Quigley
Model Approved by:	Andrew Jackson

4 HYDRAULIC MODEL DETAILS

4.4 EDENDERRY MODEL

Name	Local Authority	Unique ID	SSA	Status	Date
Edenderry	Offaly	70849	AFA	Final	08/05/2017

4.4.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600 Rp0001_Flood Risk Review) highlighted Edenderry as an Area for Further Assessment (AFA) for fluvial flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

Edenderry lies at the upstream extents of the River Boyne and is affected by both the main channel of the River Boyne and a smaller watercourse which emanates from within the town itself, called Weavers Drain. This drain flows northwards passing through a number of culverts along the way before discharging to the Boyne to the north of Edenderry. Both the River Boyne and Weavers Drain are included in the Arterial Drainage Scheme (ADS) and may be subject to periodic dredging.

The Edenderry model (Model 1 within UoM 7) is one of four hydraulic models along the River Boyne. Edenderry is the most upstream of these models, the three other models which represent three AFAs (namely Trim, Navan and Drogheda) are located downstream on the River Boyne, with the Trim model being immediately downstream of the Edenderry one.

The extents of the Edenderry model encompass identified watercourses to be modelled. Reaches of watercourses with the potential to affect Edenderry directly were assigned a high priority; this encapsulated all of the Weavers Drain and its tributary and the reach of the River Boyne starting approximately 1km upstream of the AFA boundary and 1.5km downstream. The remainder of the River Boyne was given a medium priority the downstream limit of which is located at the Boyne Aqueduct gauging station. This location is also the start of the Trim model and is an HEP allowing the two models to be checked against the hydrological assessment and ensure continuity between the two models.

The HPWs, ie the uppermost 8km of the River Boyne and Weavers Drain including its tributary were modelled as 1D-2D using the MIKE suite of software and LiDAR was used along these extents to model the 2D out of bank flow. The remaining reach of the River Boyne, MPW, was modelled as 1D using the MIKE suite of software. Extended cross sections and the national DTM were used to map the out of bank floodplain flood extents.

Downstream of the AFA boundary a number of tributaries join the Boyne main channel including major tributaries, the Yellow River (180.6km²) and the Kilwarden River (75.4km²). The total catchment area at the downstream extent (at the Boyne Aqueduct gauging station) is 431.9km².

Gauge flow records are available for the portion of the River Boyne in the vicinity of Edenderry with one gauge located at the upstream end of the model, just east of Edenderry called Kishawanny Weir (07109 – EPA). This gauging station was not given a classification under FSU and as such cannot be considered to have a high confidence at flood flows. No AMAX data has been extracted for this station.

The OPW gauging station called Boyne Aqueduct (07007) is located at the downstream boundary of Edenderry Model just west of Longwood and on the Boyne main channel where the Royal Canal traverses the river. This gauging station has three classification periods under FSU, A1 pre 1962, A1 from 1962 – 1973 and B from 1979 to date. This would suggest that post arterial drainage scheme there is less confidence in the rating but for all three periods the station should be reliable up to Q_{med} . The values for the three periods of Q_{med} are 37.15, 31.04 and 35.70 m³/s, respectively.

Although the main channel of the River Boyne affects the eastern extents of the AFA the main fluvial flood risk is due to the smaller watercourse system called Weavers Drain which emanates from the centre of Edenderry and flows northwards, discharging to the Boyne to the north of Edenderry. The Weavers Drain system is ungauged and as such estimates of Q_{med} have been derived from catchment descriptor methods. The total catchment area of weavers drain is 2.8km².

While all the watercourses being modelled are part of the ADS and may be subject to dredging the Edenderry model represents the current status as of June 2012 when the watercourses were surveyed.

(2) Model Reference:		HA07_EDEN1	
(3) AFAs included in the model:		Edenderry	
(4) Primary Watercourses / Water Bodies (including local names):			
<u>Reach ID</u>		<u>Name</u>	
0701		BOYNE RIVER	
0738		WEAVERS DRAIN	
0739		WEAVERS DRAIN TRIB	
(5) Software Type (and version):			
(a) 1D Domain: MIKE 11		(b) 2D Domain: MIKE 21	(c) Other model elements: MIKE FLOOD

4.4.2 Hydraulic Model Schematisation

(1) Map of Model Extents:

Figure 4.4.1 and Figure 4.4.2 illustrate the extent of the modelled catchment, river centre line and priority, HEP locations and AFA extents as applicable. The Boyne catchment contains 3 Upstream Limit HEPs, 1 Downstream Limit HEP, 10 Tributary HEPs and 2 Gauging Station HEPs.

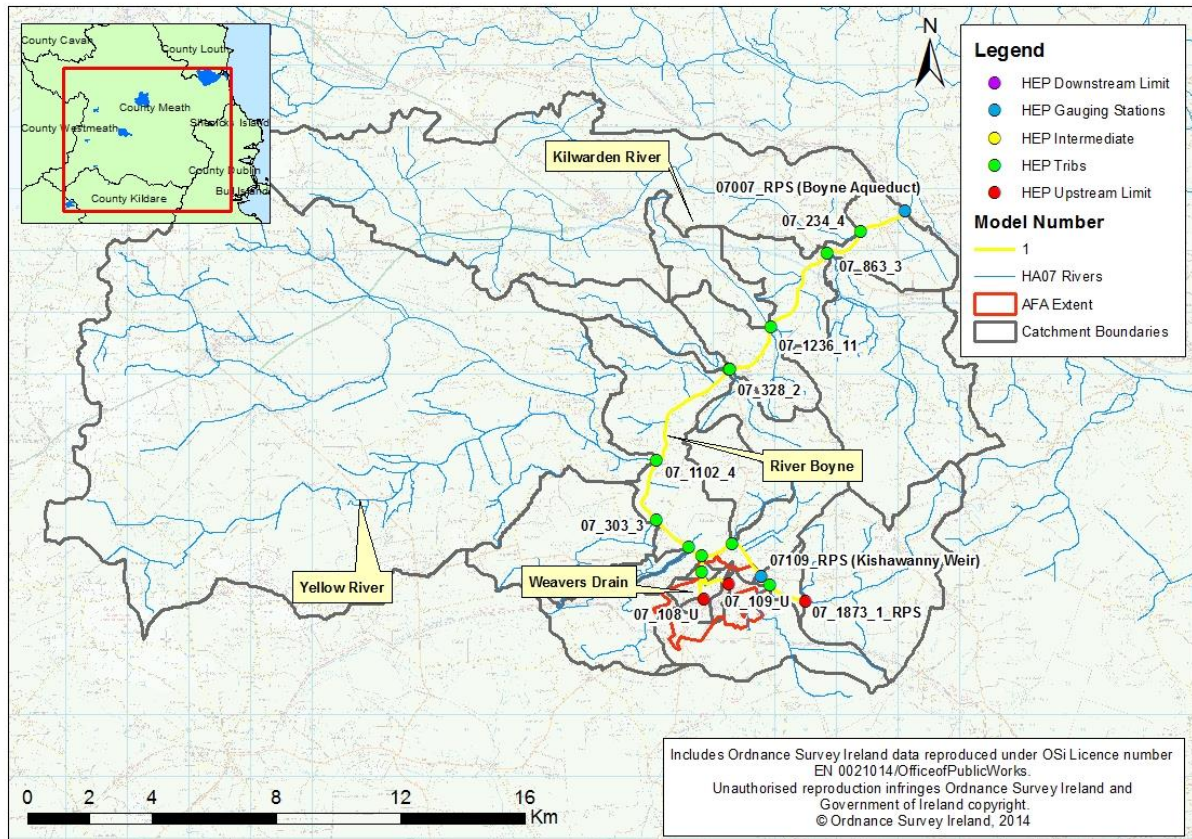
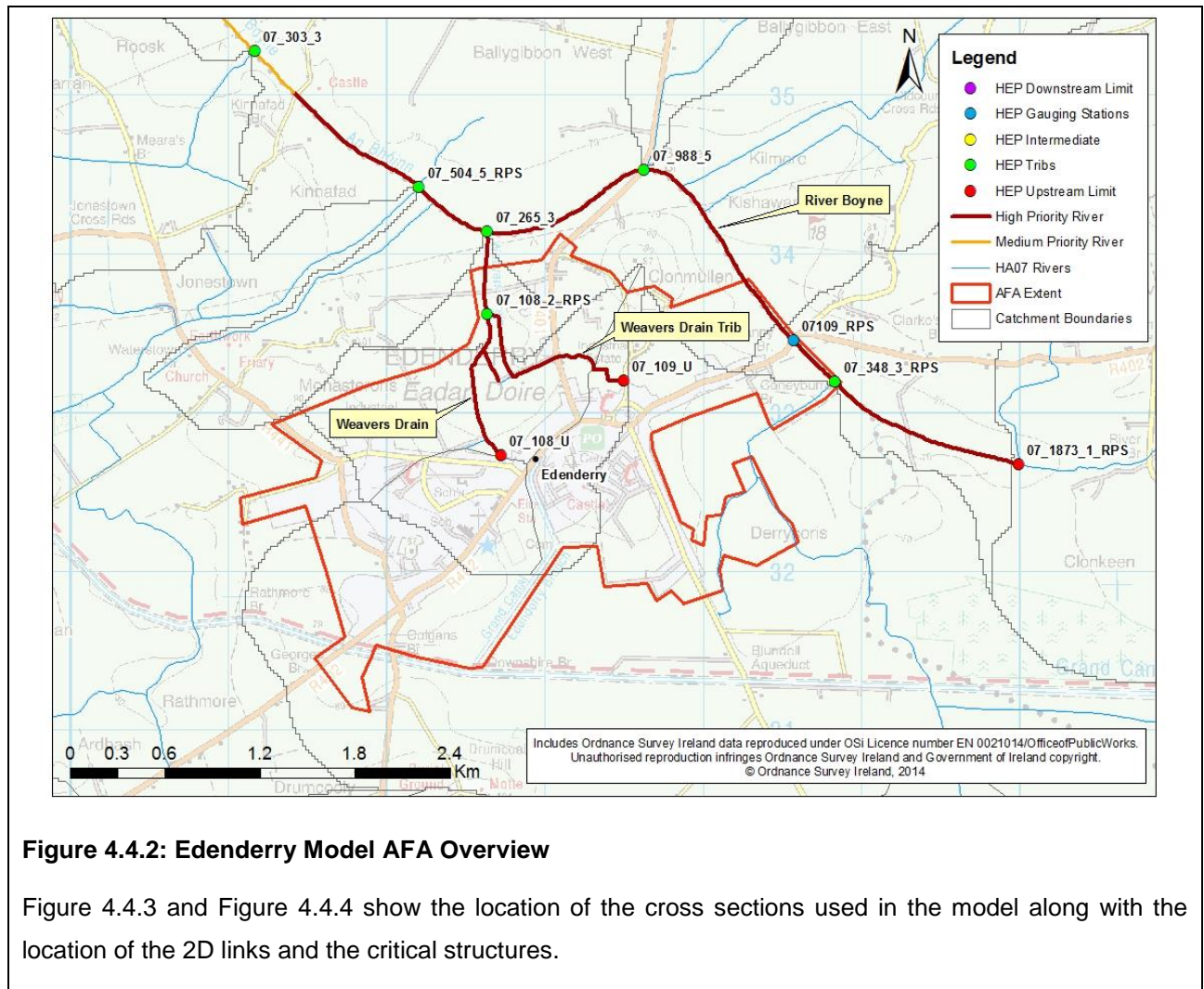


Figure 4.4.1: Edenderry Model Overview



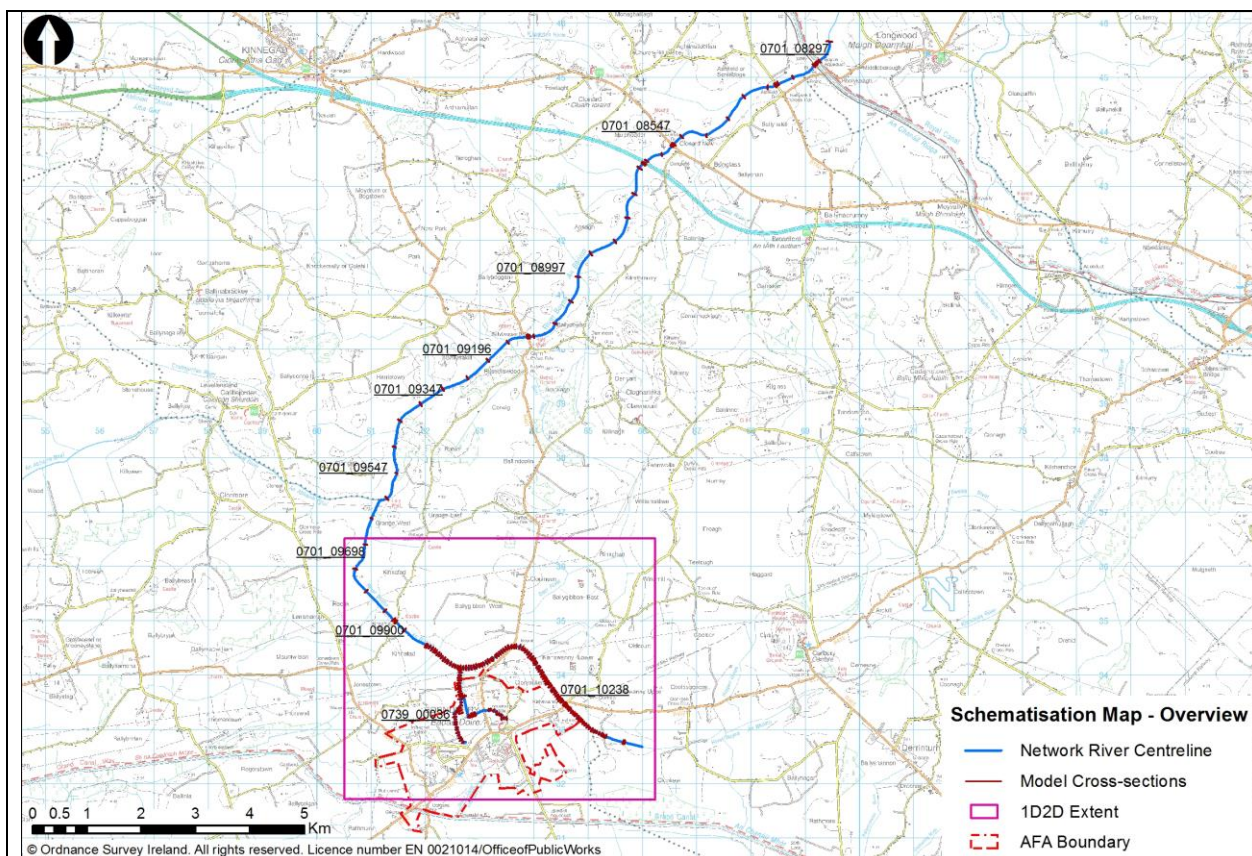


Figure 4.4.3: Overview of Model Schematisation

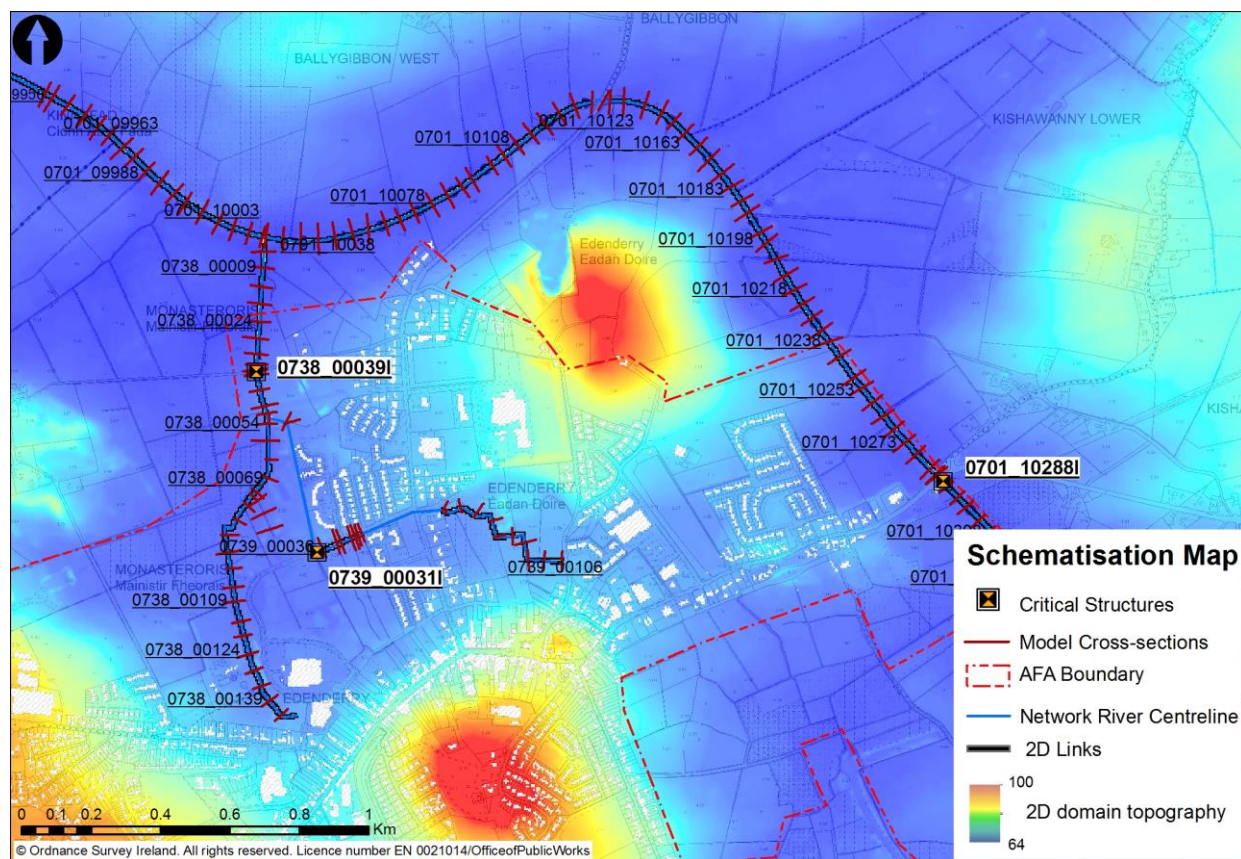


Figure 4.4.4: Model Schematisation AFA Overview

(2) x-y Coordinates of River (Upstream extent):

River Name		x	y
0701	0701 R. Boyne	265989	232679
0738	0738 Weavers Drain	262730	232763
0739	0739 Weavers Trib	263497	233205

(3) Total Modelled Watercourse Length:

24.2km (approx)

(4) 1D Domain only Watercourse Length:16.2km
(approx)**(5) 1D-2D Domain****Watercourse Length:**

8km

(approx)

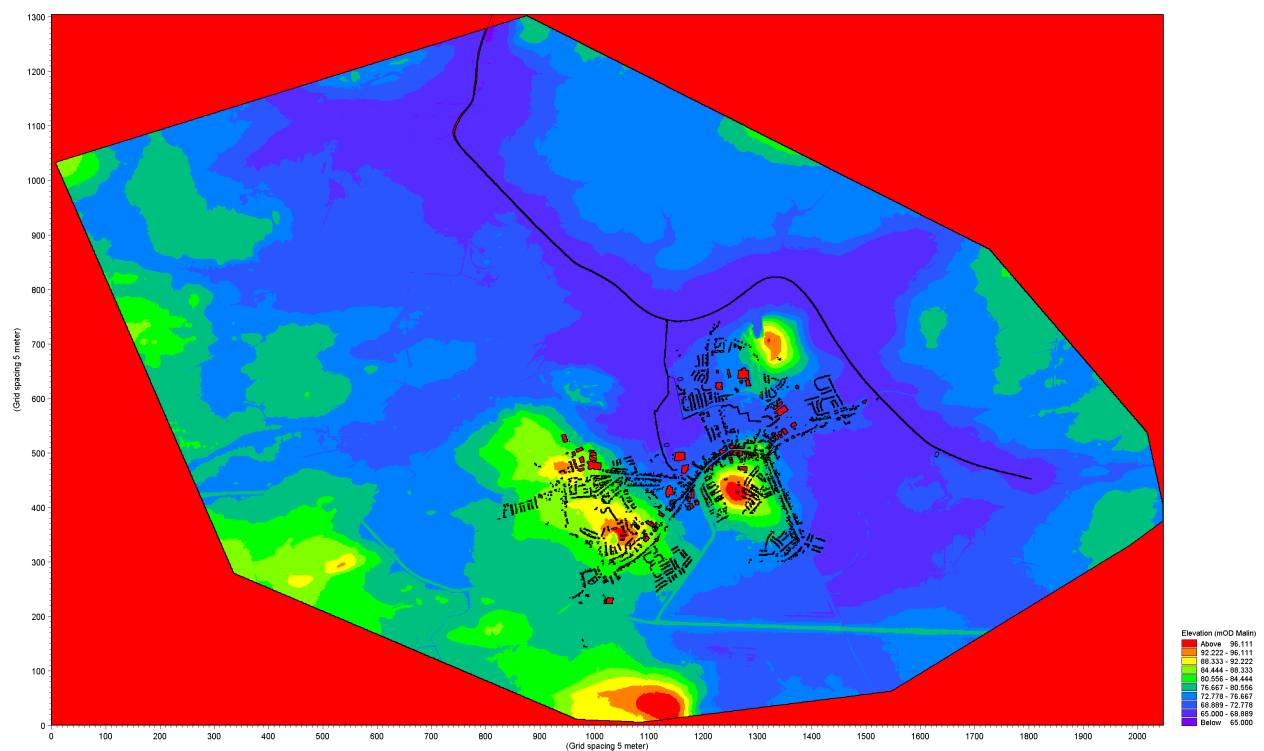
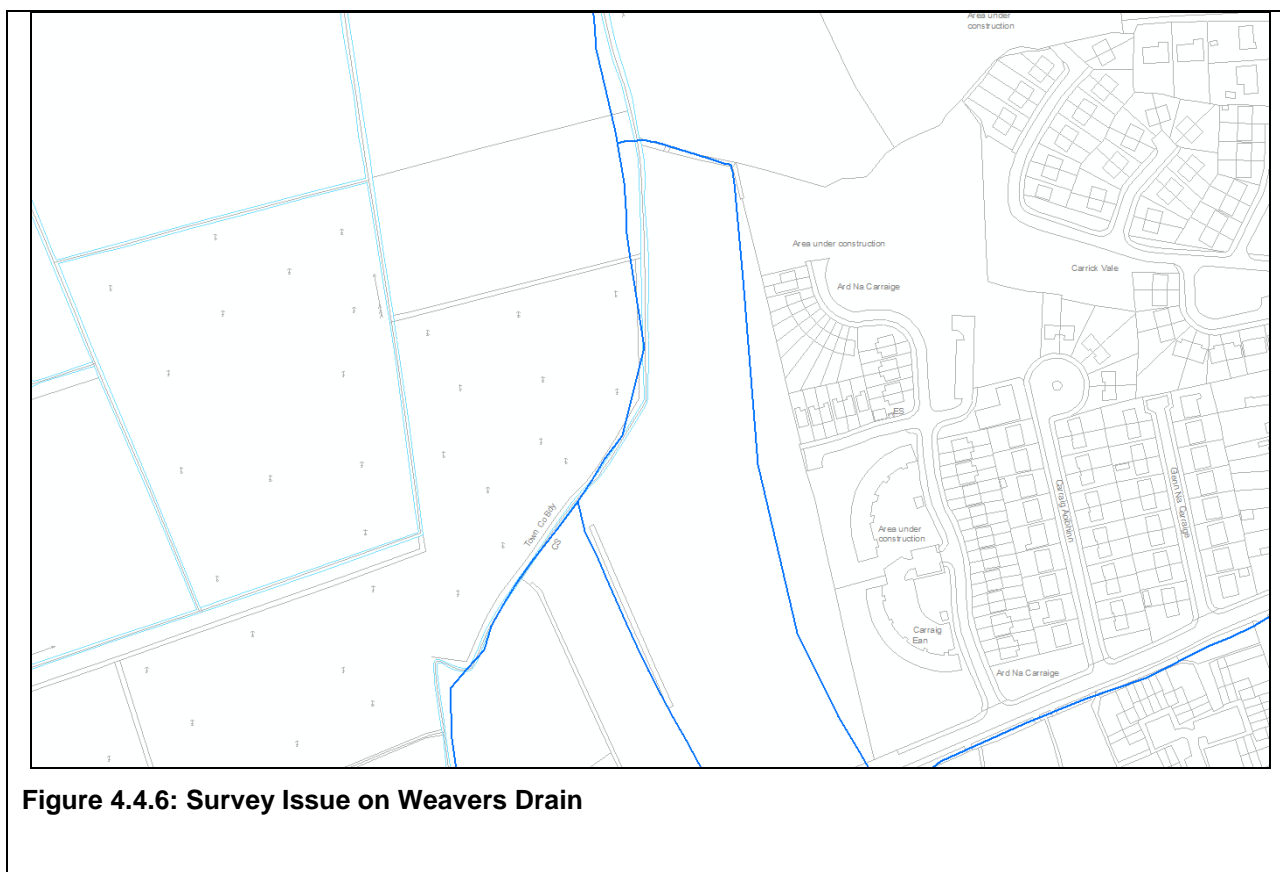
(6) 2D Domain Mesh Type / Resolution / Area:Rectangular / 5 metres / 66.85km²**(7) 2D Domain Model Extent:****Figure 4.4.5: Edenderry Model 2D Model Domain**

Figure 4.4.5 illustrates the bathymetry file showing the modelled extents of Edenderry and the general topography of the catchment. The 2D domain was generated using LiDAR survey data and created as a 5m grid rectangular mesh. No post processing was required to the bathymetry file.

(8) Survey Information**(a) Survey Folder Structure:**

First Level Folder	Second Level Folder	Third Level Folder
<i>Murphy_E07_M01_WP2_0701L_120625</i> Where: Edenderry Murphy – Surveyor Name	GIS and Floodplain Photos	Floodplain Photos and Shapefiles
		Structure Register

E07 – Eastern CFRAM Study Area, Hydrometric Area 07 M01 – Model Number 1 WP2 – Work Package 2 0701I– River Reference ID 120625 – Date Issued (25 th Jun 2012)	Ascii													
	Drawings and PDFs													
	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)													
(b) Survey Folder References:														
<table border="1"> <thead> <tr> <th>Reach ID</th><th>Name</th><th>File Reference</th></tr> </thead> <tbody> <tr> <td>0701</td><td>BOYNE</td><td> Murphy_E07_M01_WP2_0701I_120625 Murphy_E07_M01_WP2_0701G_120625 Murphy_E07_M01_WP2_0701H_120625 </td></tr> <tr> <td>0738</td><td>WEAVERS DRAIN</td><td>Murphy_E07_M01_WP2_0738_120625</td></tr> <tr> <td>0739</td><td>WEAVERS DRAIN TRIB</td><td>Murphy_E07_M01_WP2_0739_120625</td></tr> </tbody> </table>			Reach ID	Name	File Reference	0701	BOYNE	Murphy_E07_M01_WP2_0701I_120625 Murphy_E07_M01_WP2_0701G_120625 Murphy_E07_M01_WP2_0701H_120625	0738	WEAVERS DRAIN	Murphy_E07_M01_WP2_0738_120625	0739	WEAVERS DRAIN TRIB	Murphy_E07_M01_WP2_0739_120625
Reach ID	Name	File Reference												
0701	BOYNE	Murphy_E07_M01_WP2_0701I_120625 Murphy_E07_M01_WP2_0701G_120625 Murphy_E07_M01_WP2_0701H_120625												
0738	WEAVERS DRAIN	Murphy_E07_M01_WP2_0738_120625												
0739	WEAVERS DRAIN TRIB	Murphy_E07_M01_WP2_0739_120625												
(9) Survey Issues:														
<p>The surveyed river centreline on the Weavers Drain did not agree with the OSi vector mapping (see Figure 4.4.6). A review of the centreline found it to be inaccurate. Weavers Drain centreline was therefore corrected and updated in the network file. The cross section chainage was also adjusted to place the cross sections in the right location.</p>														



4.4.3 Hydraulic Model Construction

(1) 1D Structures (in-channel along modelled watercourses):	See Appendix D Number of Bridges and Culverts: 15 Number of Weirs: 1
--	--



Figure 4.4.7: Photograph of Critical Structure on Weavers Drain Trib

Critical structure identified on the Weavers Drain Trib at chainage 870m. This culvert is located immediately downstream of a right angle bend and out of bank flooding occurs here.



Figure 4.4.8: Photograph of Critical Structure on River Boyne

Critical structure identified on the River Boyne at chainage 1652m



Figure 4.4.9: Photographs of Critical Structure on Weavers Drain

Critical structure identified on Weavers Drain at chainage 1108m. This culvert restricts flow during the 1% and 0.1% AEP events and causes a backwater effect upstream.

**(2) 1D Structures in the 2D domain
(beyond the modelled watercourses):**

None

(3) 2D Model structures:

None

(4) Defences:

Type	Watercourse	Bank	Model Start Chainage (approx.)	Model End Chainage (approx.)
Informal - None				

(5) Model Boundaries - Inflows:

Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report - Section 4.1 and Appendix D). The boundary conditions implemented in the model are shown overleaf.

Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Boundary ID
Open	Inflow	River Boyne	0	0	07_1873_1
Point Source	Inflow	River Boyne	1286.58	0	07_348_3
Point Source	Inflow	River Boyne	3138.7	0	07_988_5
Point Source	Inflow	River Boyne	4700	0	07_504_5_RPS
Point Source	Inflow	River Boyne	6120	0	07_303_3
Point Source	Inflow	River Boyne	8361.69	0	07_1102_4
Point Source	Inflow	River Boyne	12486.4	0	07_328_2
Point Source	Inflow	River Boyne	12510.7	0	07_485_3
Point Source	Inflow	River Boyne	14609.4	0	07_1236_11
Point Source	Inflow	River Boyne	17937.5	0	07_863_3
Open	Q-h	River Boyne	21458	0	07007_RPS
Distributed Source	Inflow	WEAVER DRAIN	23	1431	Top-up flow between 07_108_U & 07_265_3
Distributed Source	Inflow	WEAVERS DRAIN TRIB	18	863	07_108_2_RPS
Open	Inflow	WEAVER DRAIN	0	0	07_108_U
Open	Inflow	WEAVERS DRAIN TRIB	0	0	07_109_U
Point Source	Inflow	River Boyne	19373.5	0	07_234_4
Distributed Source	Inflow	River Boyne	0	1663	Top-up between 07109_RPS & 07_1873_1
Distributed Source	Inflow	River Boyne	1663	21000	Top-up between 07007_RPS & 07109_RPS

**Figure 4.4.10 shows the inflow hydrograph to the River Boyne during the 0.1%AEP event at HEP
07_1873_1_RPS**

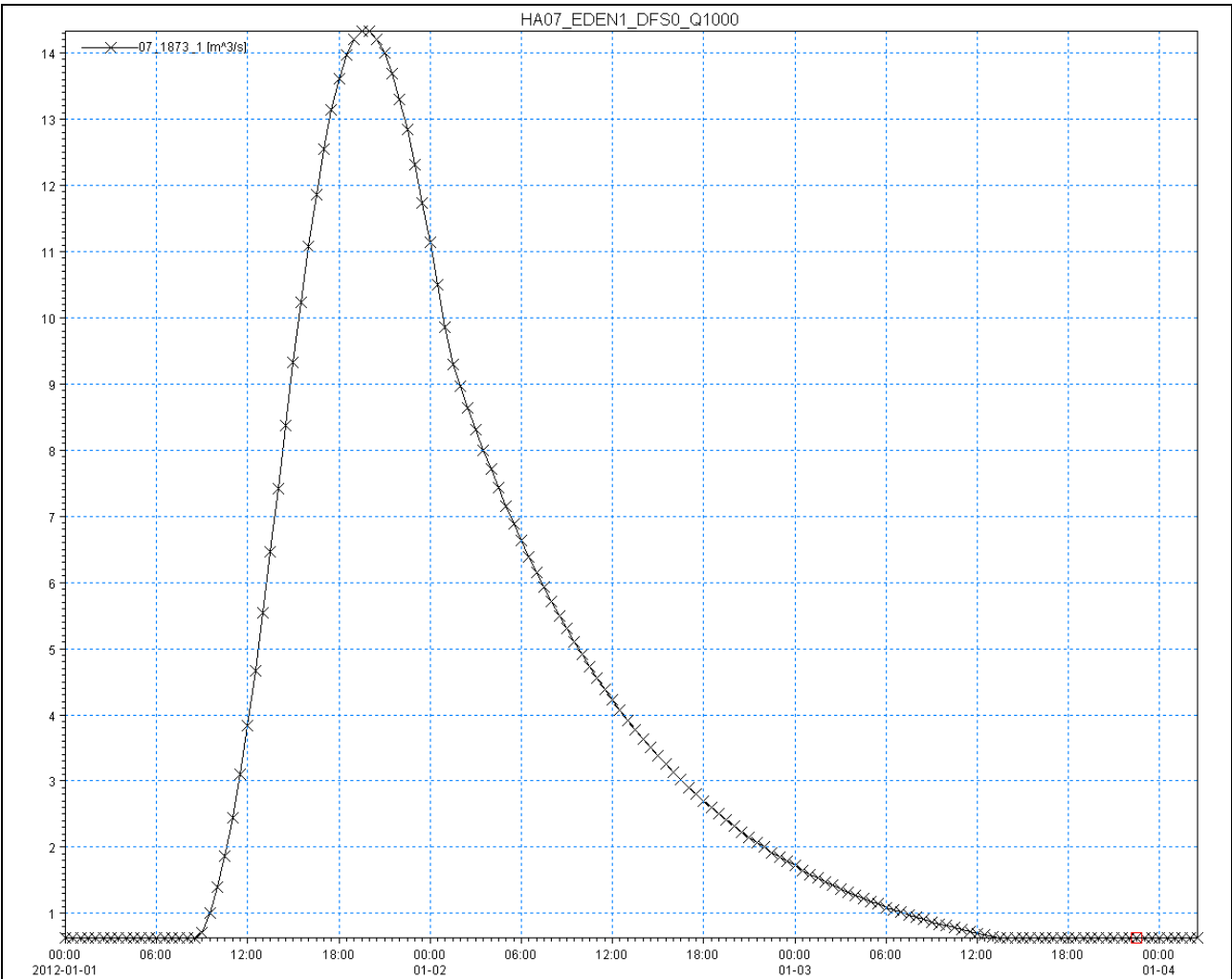


Figure 4.4.11: Inflow Hydrograph for the River Boyne during a 0.1%AEP Event

The upstream boundaries of Weavers Drain and Weavers Drain Trib are located at or close to the highest point in their catchments. Therefore the flow along each watercourse is best represented by a distributed flow. This was carried out as shown in the table above however a nominal inflow was also provided at the upstream HEPs in order to ensure the model runs start in a stable condition.

No rating review was carried out on any gauging station within the Edenderry model and no change was made to the hydrology for this model during the calibration process. See Section 4.1.5(2) which discusses model updates for Final deliverables.

(6) Model Boundaries – Downstream Conditions:

The downstream boundary condition is a Q-h relationship, generated based on the manning equations and uses the downstream extent of the model with a slope of 0.001.

This is located downstream of the Royal Canal on the River Boyne (Chainage 21458).

To ensure continuity between adjacent models, in this case between the Edenderry model and the Trim model located downstream, a check was carried out to ensure that the flow output from the Edenderry model

matched that of the inflow to the Trim model. As such the QH relationship at the downstream extent of the Edenderry model and the upstream extent of the Trim model was compared and is shown below.

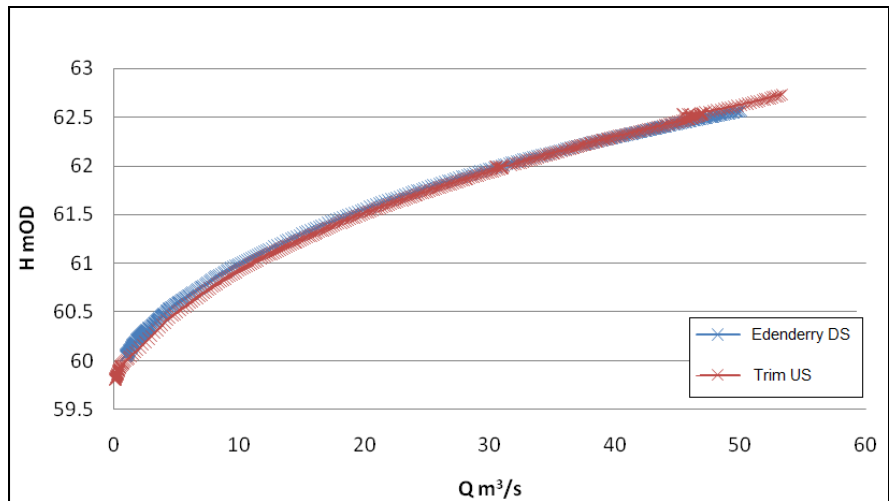


Figure 4.4.12: Comparison of QH Relationship between Adjacent Models Edenderry and Trim

A good correlation was found giving confidence that the downstream boundary is representative and that there is no significant control structure downstream of the Edenderry model controlling the water levels.

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.020	Maximum 'n' value: 0.060
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.060	Maximum 'n' value: 0.060
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.013 (Inverse of Manning's 'M')	Maximum 'n' value: 0.065 (Inverse of Manning's 'M')

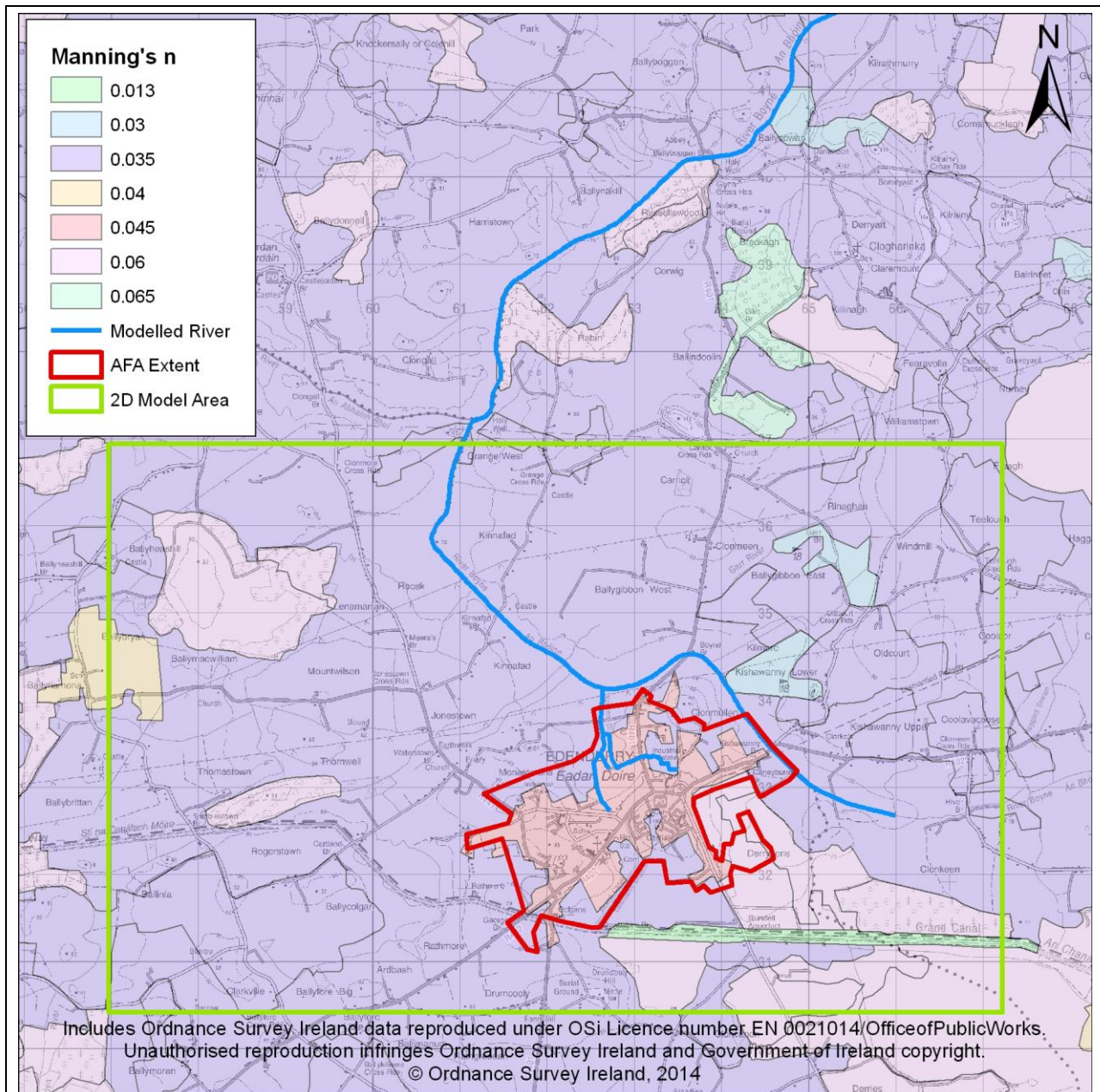


Figure 4.4.13: Floodplain Roughness Values in 2D Domain

This map illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset.

The surrounding rural area to Edenderry is assigned a Manning's n of 0.035 with the exception of Edenderry Golf Course on the north side of the River Boyne which has a Manning's n value of 0.030. Edenderry is assigned a Manning's n value of 0.045.

(d) Examples of In-Bank Roughness Coefficients**Weaver Drain Trib**

Figure 4.4.14: Example of roughness coefficients on Weavers Drain Trib at cross section 0739_00041

Manning's $n = 0.040$

Straight stream with vegetation

River Boyne

Figure 4.4.15: Example of roughness coefficients on River Boyne at cross section 0701_10338

Manning's $n = 0.040$

Standard natural stream with vegetation

Lat: 53° 20' 34.33" N
Lon: 7° 3' 38.33" W



Figure 4.4.16: Example of roughness coefficients on Weaver Drain at cross section 0738_00129

Manning's $n = 0.060$

Straight stream with heavy vegetation

4.4.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact 1% AEP hydraulic model within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a moderate increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.4.17. This outcome indicates that the Edenderry model demonstrates a moderate sensitivity to changes in roughness parameters. The increase in flood extents affects a forested area off Weavers Drain. No additional AFA receptors are impacted by the increase in flood extent when compared to the 1% AEP design event.

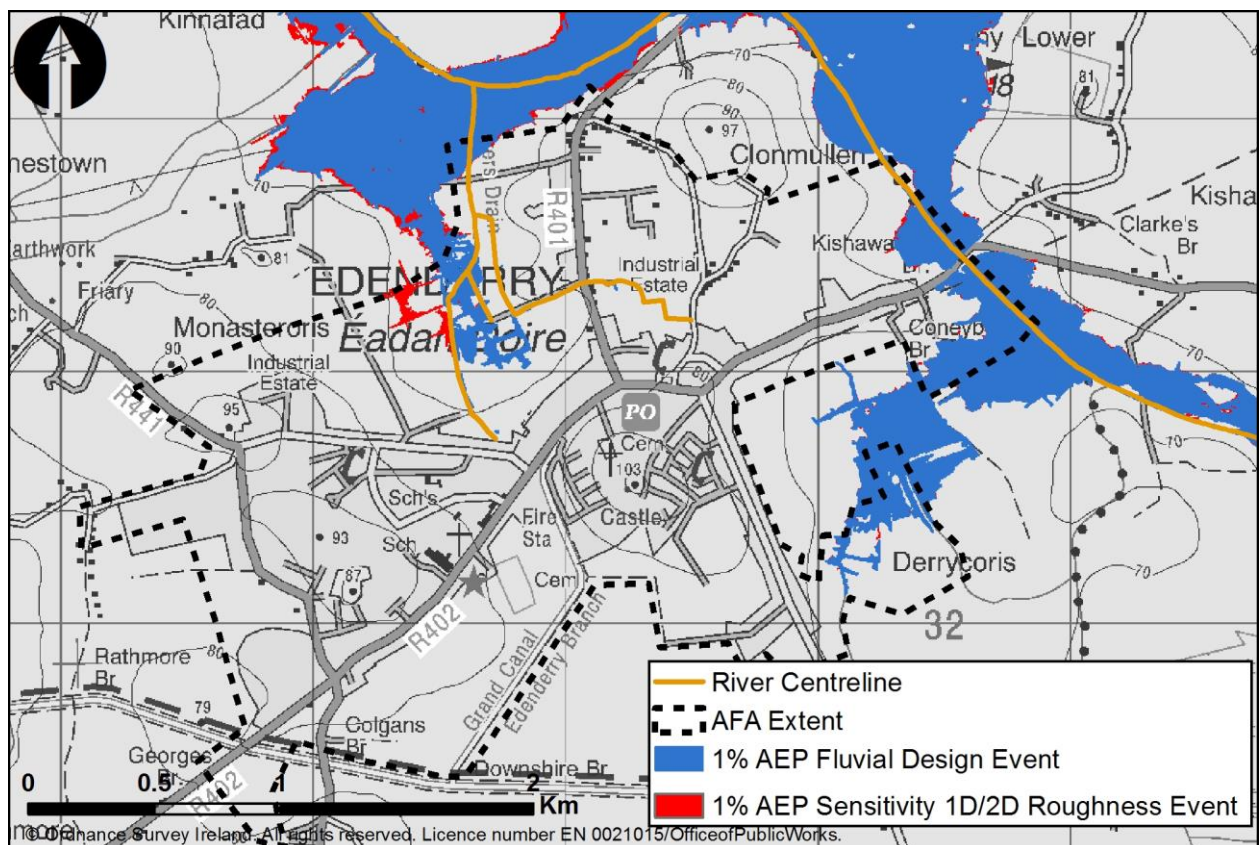


Figure 4.4.17: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in 1D/2D Roughness

- b) Downstream boundary increase –The Edenderry downstream boundary condition is based upon a model generated Q-h relationship, refer to section 4.4.3(6) of this report for further detail. This downstream boundary is located at the modelled downstream extent of the Royal Canal on the River Boyne (chainage 21,550m); which is located approximately 17km from the edge of Edenderry AFA, with a 4m difference between the upstream and downstream bed-levels. It can be determined from these modelled dimensions that the Edenderry downstream boundary has no impact upon Edenderry

AFA, subsequently there is no requirement to assess the sensitivity of the downstream water level.

- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Edenderry model was assessed as having differential levels of certainty relating to inflows. The Edenderry model was assessed as having differential levels of certainty relating to upstream or downstream watercourse position. There is a high certainty associated with the Boyne main channel flows located downstream of the AFA. Whereas upstream flows demonstrate uncertainties associated with low quality gauge information (refer to the Hydrology Report IBE0600Rp0012 *for further detail*); factors of 1.37 and 1.68 are applied to design flows for the sensitivity simulation. Figure 4.4.18 shows that the Edenderry model has a high sensitivity to increased inflow parameters; this is reflected by the increase of flood extent. Significant increase of the flood extent affects Carrickhall and Gleann Na Carraige areas of Edenderry and the area within a close proximity to the Weavers Drain. Increased flood extents impact 1 receptors located within the AFA, whereas no properties were affected during the 1% AEP design event.

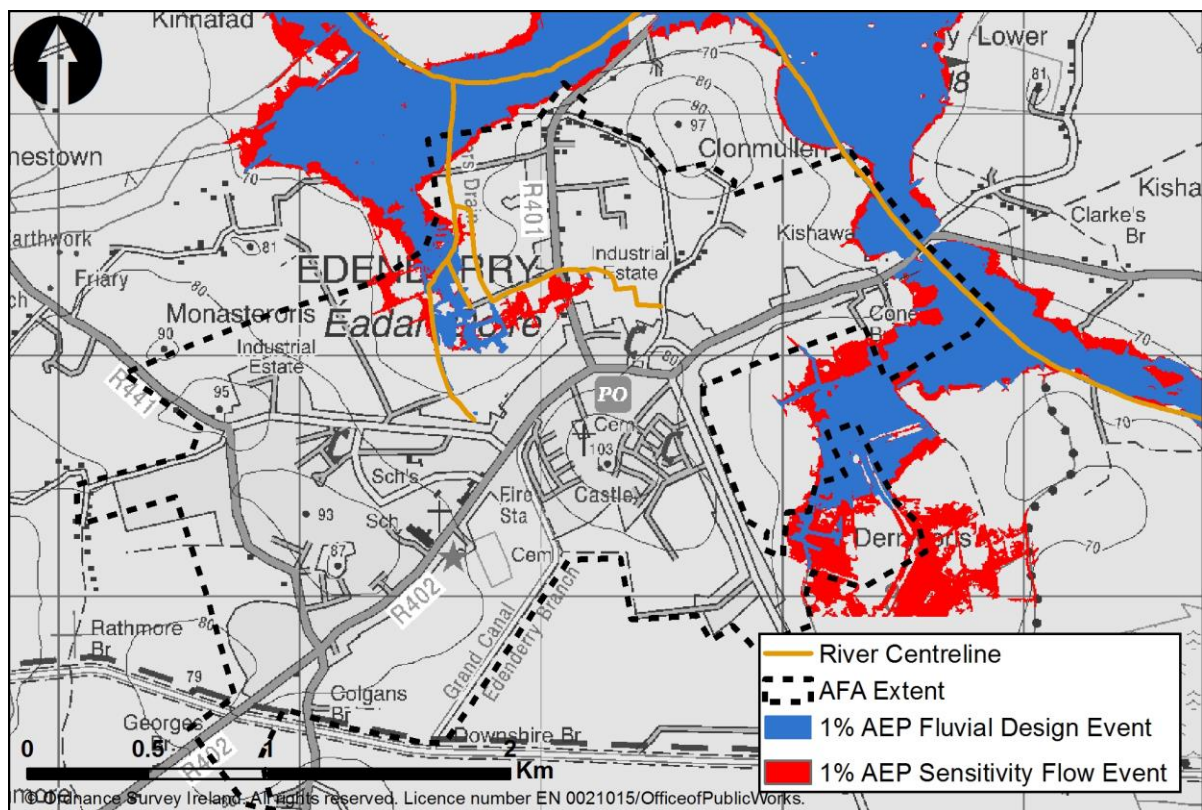


Figure 4.4.18: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event

- d) Variation in the timing of tributaries – A sensitivity adjustment has been applied to timings of tributary hydrographs to assess the effect on the model. Tributary hydrographs have been moved by up to 10% of the graph duration to bring peak flows closer to the main channel peak flow. There is a minimal increase in flood extents within the AFA, as shown in Figure 4.4.19. The Edenderry model is considered to have a low sensitivity to timing of tributaries and no receptors are impacted by the increase of flood extents within the AFA.

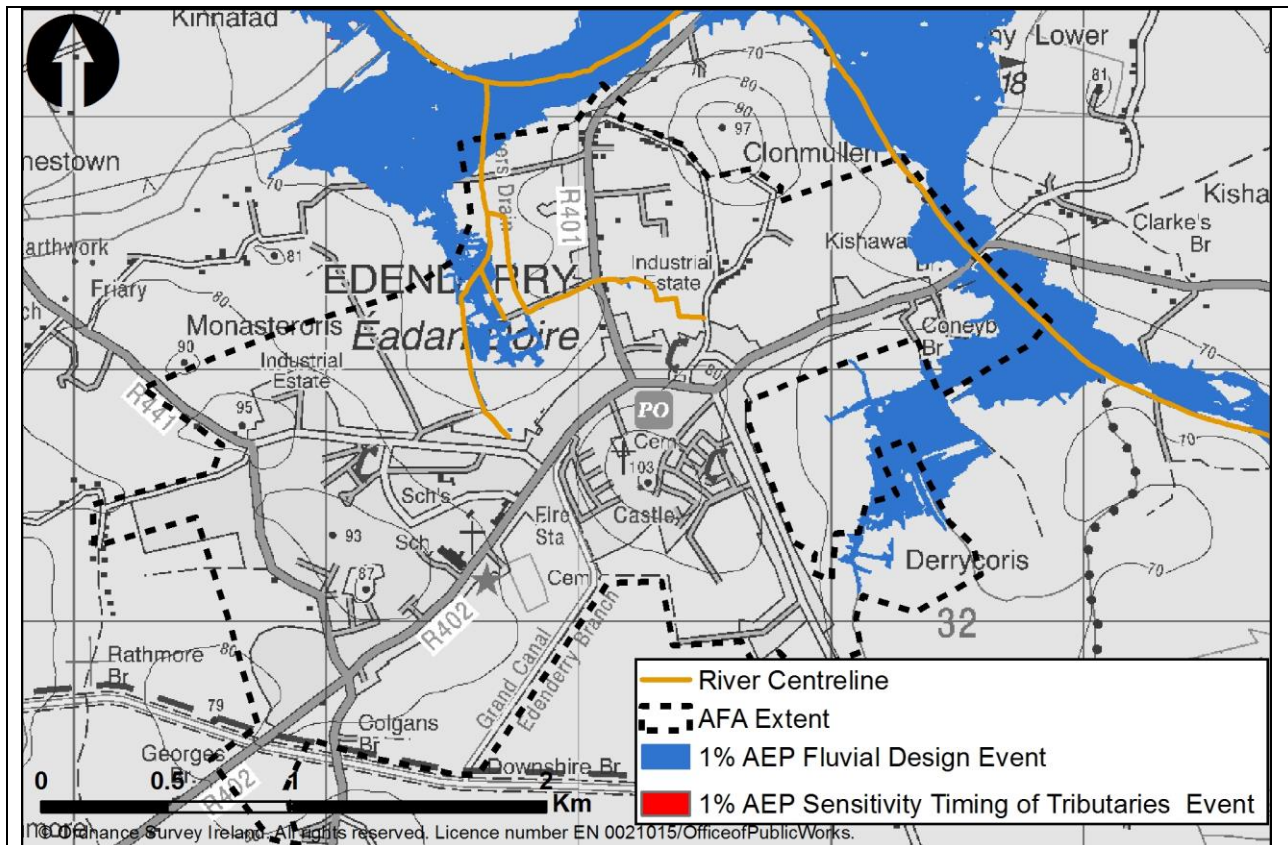


Figure 4.4.19: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Timing of Tributaries

e) Flow volume – A sensitivity adjustment factor of 2 has been applied to flood duration to assess the effect on the model as flood durations in this case have been derived from observed data with some uncertainty at flood flows. Figure 4.4.20 demonstrates that the Edenderry model indicates a moderate sensitivity to flow volume parameters adjustment, which is resultant of a moderate increase to the flood extents. Similar to the 1% AEP design event, this change does not have any further impact upon receptors located within the AFA boundary.

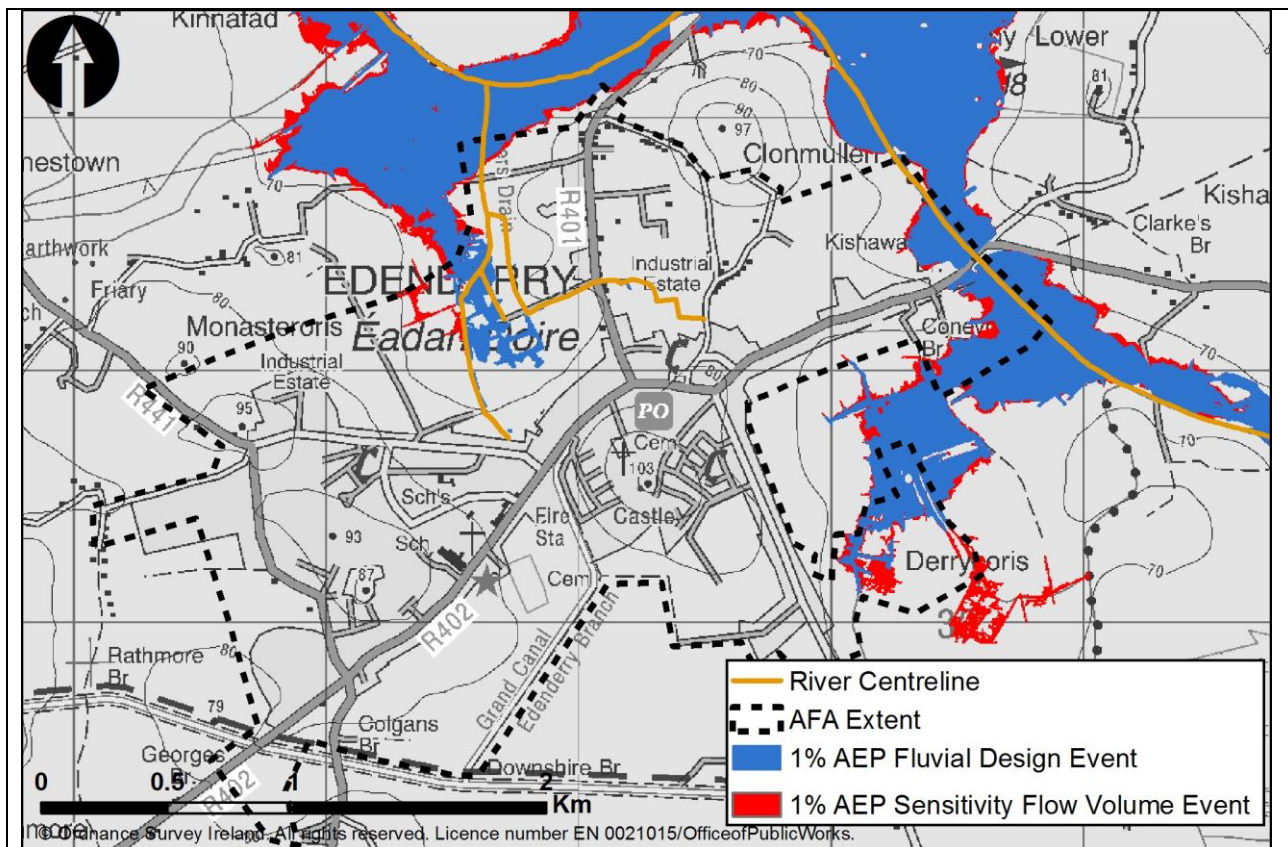


Figure 4.4.20: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Volume Event

- f) Afflux/Head loss at key structures – A number of simulations were carried out to assess the sensitivity of flood extents to changing the head loss coefficients of key structures. Two structures were assessed including 0701_10288I and 0739_00071I on the River Boyne and Weavers Drain, respectively. The Edenderry model has shown it has a low sensitivity to parameter changes as there was little change in flood extents and no impact to receptors as shown in Figure 4.4.21.

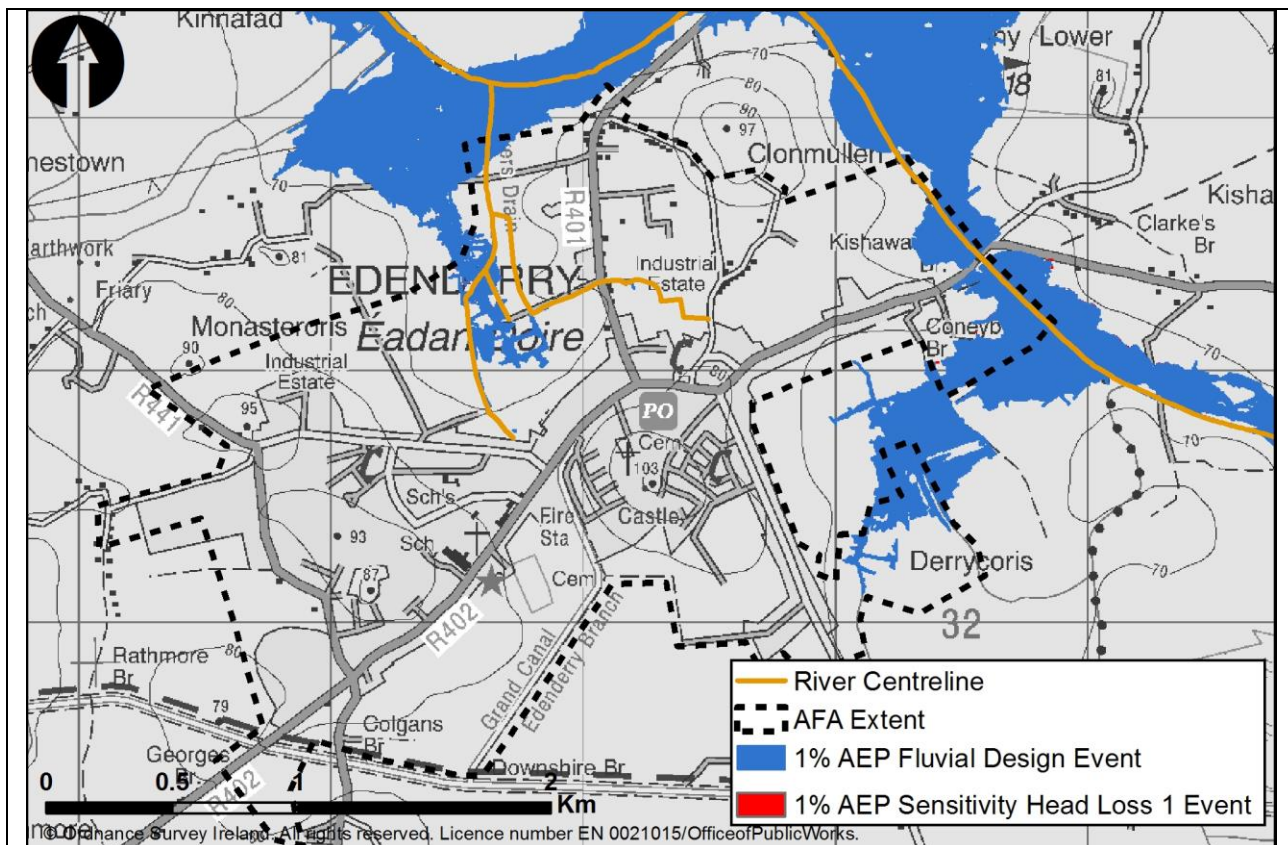


Figure 4.4.21: Comparison between 1% AEP Design Event and 1% AEP Sensitivity (Head Loss 1)

Table 4.1 summarises the outcomes of the above sensitivity simulations or evaluations that were considered for the Edenderry model. Of these parameters, the model is most sensitive to increase in model inflows. Table 8.1 states that the Edenderry model was assessed as having differential levels of certainty relating to upstream or downstream watercourse position. There is a high certainty associated with the Boyne main channel flows located downstream of the AFA. Whereas upstream flows demonstrate uncertainties associated with low quality gauge information. The model is less sensitive to the other parameters, with the resulting analysis identifying low increases in flood extents. The 1% AEP fluvial design event did not impact any receptors located within the AFA. In comparison, sensitivity impact is considered as relatively high.

Table 4.1: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	Moderate	-
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	High	High
1% AEP Sensitivity Timing of Tributaries	Low	-
1% AEP Sensitivity Flow Volume Event	Moderate	-
1% AEP Sensitivity Head Loss 1 Event	Low	-

4.4.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0600Rp0004_HA07 Inception Report unless otherwise specified):

(a) Aug 2008	The River Boyne overflowed in the Edenderry area during this flood event and flooded low lying land near the river. The mean daily flow at Kishawanny Weir, obtained from the EPA Hydronet website, reached 3.37m ³ /s compared to an average daily flow of 0.45m ³ /s from all available records. The peak flow recorded for this event was 3.397m ³ /s. From available information, the flooding appeared to mostly affect agricultural land, although parts of Edenderry Golf Club were flooded also. Aerial photographs taken during the flood do not indicate flooding in the town itself or flooding of roads or houses.
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Figure 4.4.22: Photograph of Aug 2008 Floods at Edenderry Golf Club

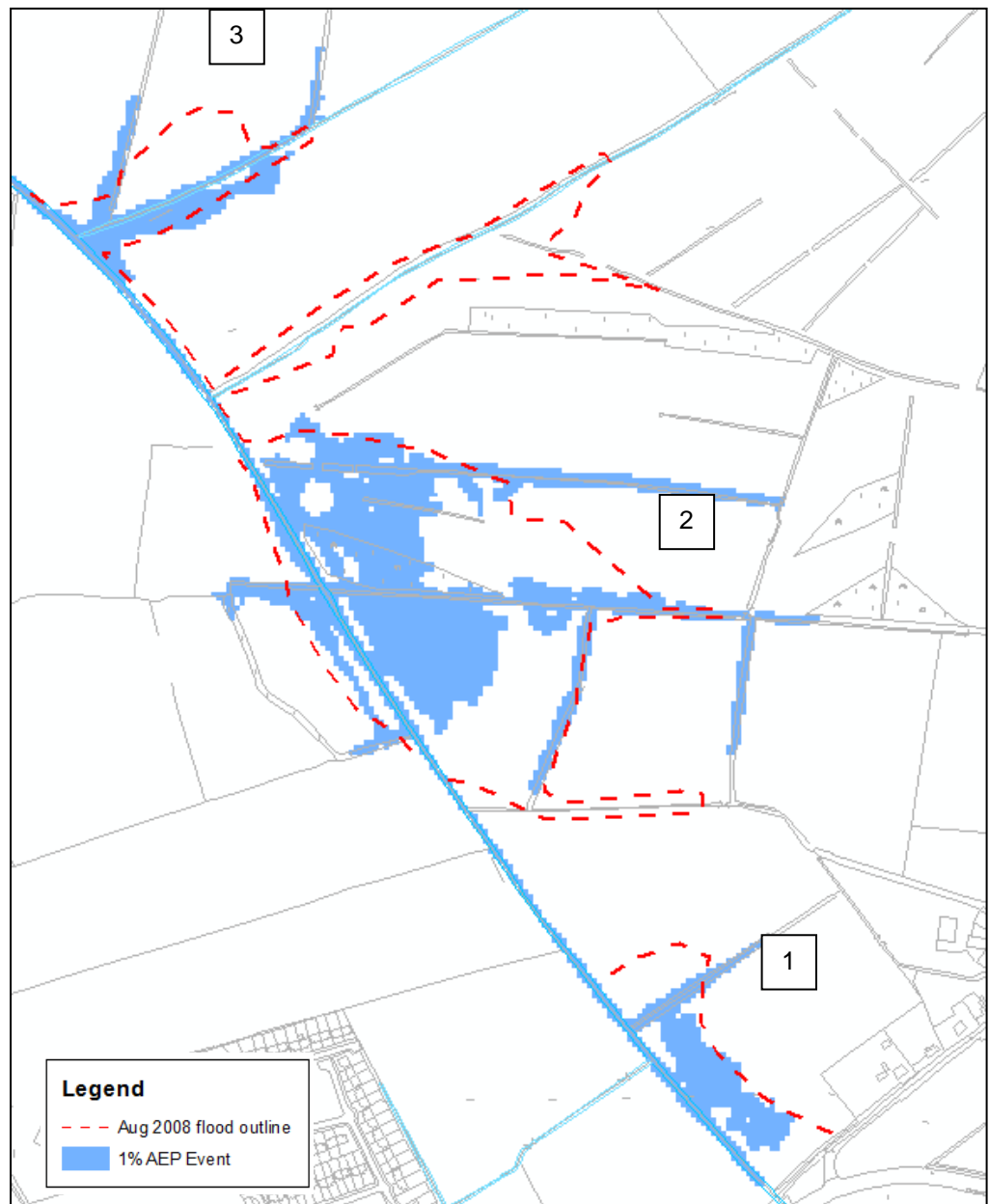


Figure 4.4.23: Comparison between Modelled 1% AEP Event and Estimated Aug 2008 Flood Extent

A single site flood frequency analysis was carried out at the Boyne Aqueduct gauging station using the AMAX records from 1953 to 2008. The analysis estimated the 2008 event to be around a 5%AEP event. When comparing the historical flood with the modelled flood extents at the area where aerial photography captured the flood extent, at Edenderry Golf Club, it shows that the flood event was greater than a 10%AEP event but perhaps closer to a 1% AEP event. As the Boyne Aqueduct is quite remote from Edenderry Golf Club and a few significant tributaries join the River Boyne in between little confidence can be afforded to this single site analysis alone.

	<p>Kishawanny Weir gauging station was also considered for single site analysis but due to the short term records available it was considered inappropriate. Rainfall data was therefore assessed in the Edenderry area however records for all of the stations (Edenderry (the tunnel), Edenderry G.S. and Edenderry (Ballinla)) ended before the year 2000 and couldn't be considered directly and as such processed rainfall radar data (see Hydrology Report) was therefore used instead. The precipitation estimates at Kishawanny Weir for the 2008 event were used to calculate the rainfall frequency based on the FSU DDF model. A frequency of approximately 5% AEP was calculated.</p> <p>At first glance it may appear that the model is underestimating flood extents however this event was preceded by two similarly large floods as well as other minor events in the course of 10 days. Further to this flooding coming directly from the drains feeding into the River Boyne and affecting the agricultural land adjacent to the Boyne are not modelled and will in all likelihood contribute to the flooding in the Boyne floodplain. Coupled with this, the recorded flow was $3.4\text{m}^3/\text{s}$, considerably less than the modelled 10%AEP event flow at $5.5\text{m}^3/\text{s}$. All of the available data indicates that the event is broadly consistent with the modelled flood extents.</p>
(b) Nov 2002	<p>Information was found for a flood event which occurred in Drogheda, Navan, Trim and Edenderry in November 2002. In Edenderry, the only available information is a photo which indicates flooding at Kishawanny Bridge. It appears as though only low lying lands adjacent to the river were flooded and that no flooding of properties, or assets in the town itself, occurred.</p> <p>The model results show that one of the most susceptible areas to flooding is at the Kishawanny Bridge. This is consistent with the historical flood records.</p>
(c) Feb 2002	<p>This event was caused by heavy rainfall causing the River Boyne to overtop its banks. In Edenderry, the available photos indicate that flooding of Edenderry Golf Club and other low lying lands adjacent to the river occurred. No information is available to indicate flooding of roads or properties.</p> <p>The model results are consistent with the historical evidence that no properties are at risk up to the 0.1% AEP event from the River Boyne.</p>
(d) Nov 2000	<p>Information was found for a flood event which occurred in Drogheda, Navan, Trim, Baltray, Edenderry and Mornington in November 2000. Floods were caused by heavy rain and storm force winds. In Edenderry, flooding of low lying lands near the river occurred. However, at the time of the flooding the road near Kishawanny Bridge was being realigned and it is possible that these works may have had an effect on the river flow.</p> <p>The model results show that one of the most susceptible areas to flooding is at the</p>

Kishawanny Bridge. This is consistent with the historical flood records.

(2) Post Public Consultation Updates:

Following comments received during informal public consultation and formal S.I. public consultation periods in 2015, analysis of the results of the rating reviews and their impact on the hydrological analysis for the UoM07 watercourses was reviewed and finalised. The updated inflow hydrograph at HEP 07_1679_5 is shown in Figure 4.4.24.

A review of the Manning's n values was undertaken along the River Boyne and Weavers Drain Tributary, along with the Manning's n values and head loss factors for all structures within the model. All values were increased in order to better represent the flood extents reported through the consultation process, whilst still being representative of the conditions on-site. General model updates were also applied to refine model resolution and improve model stability.

A revised set of flood hazard and risk mapping has been issued as Final to reflect these changes.

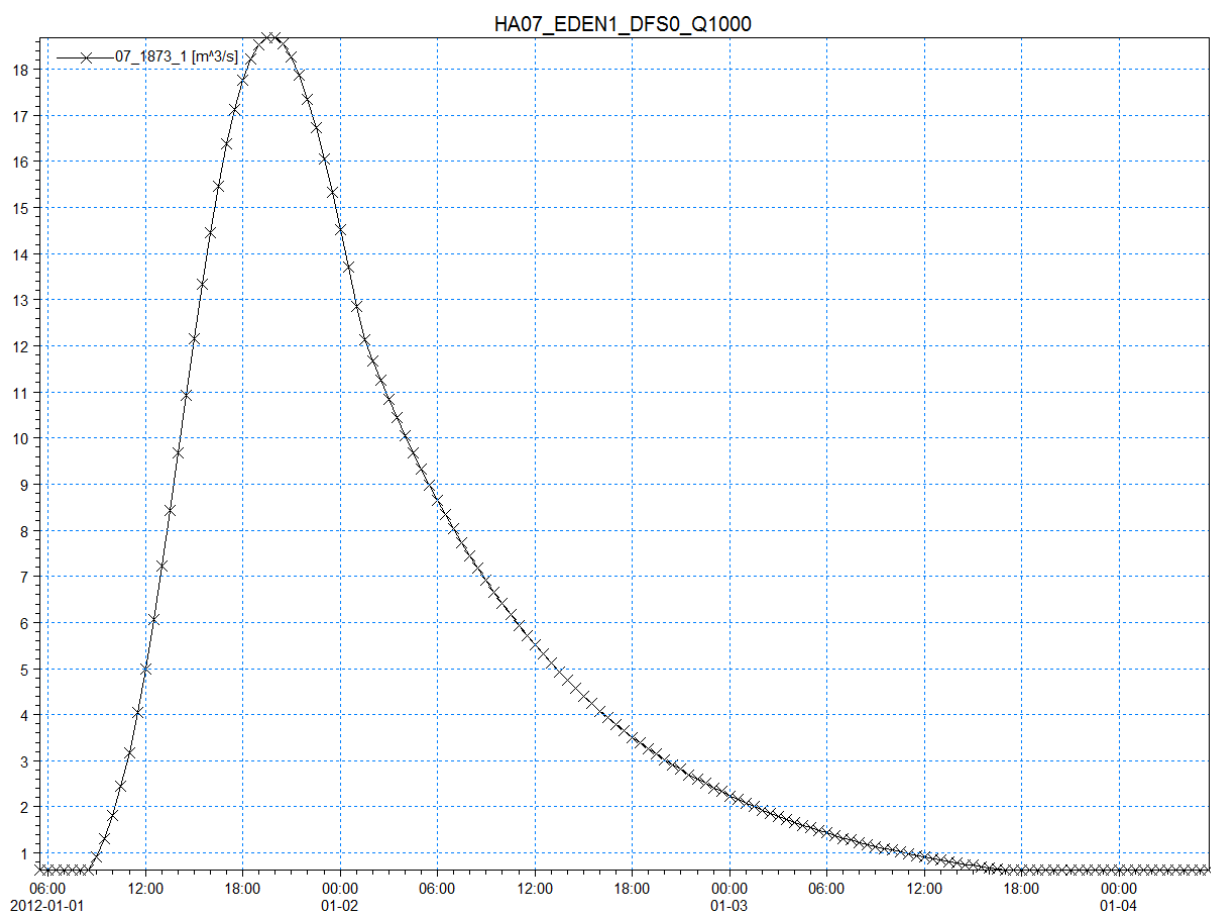


Figure 4.4.24: Inflow Hydrograph for the River Boyne during a 0.1% AEP Event

(3) Standard of Protection of Existing Formal Defences:

Defence Reference	Type	Watercourse	Bank	Modelled Standard of Protection (AEP)
None				

(4) Gauging Stations:

There are two gauging stations within the model extent, both have water level and flow information available.

(a) Kishawanny Weir (07109)

Operated by the EPA. It is located on the River Boyne at chainage 1623m within the Edenderry model. Rating information is available for this station.

An attempt was carried out to calibrate the model to the EPA rating curve at Kishawanny Weir gauging station. It was found that the rising limb of the model's Q/H curve underestimated the water level for any given flow compared with the EPA rating curve and the falling limb overestimated the same. This is known as a hysteresis loop where the attenuation of the flow in the flood plain causes a lagging effect as the flood disperses. Due to this effect it is likely that the spot gaugings recorded would have a high degree of scatter, with some reflecting measurements taken on the rising limb and others reflecting measurements taken on the falling limb of the flood hydrograph in question. The rating curve taking account of this scatter would be shown as an average between the rising and falling gaugings. However no spot gaugings were made available for this station and as such this cannot be confirmed and therefore little confidence can be afforded to the EPA rating curve when considering flood flows. The result of this is that the Kishawanny gauging station is unsuitable to be used for calibration of the model to specific flood event hydrometric data.

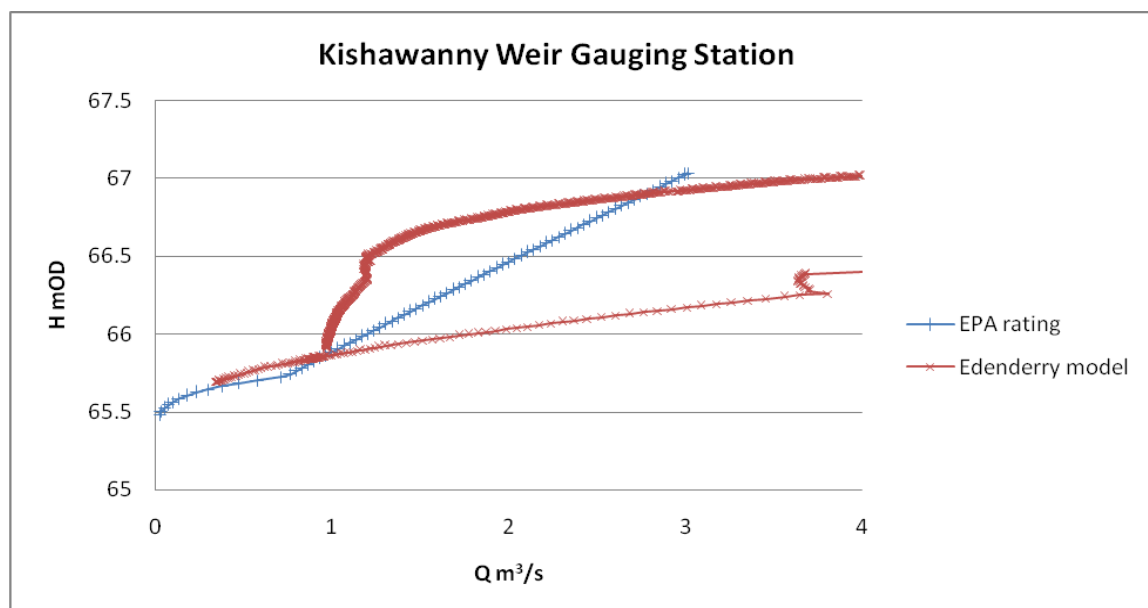


Figure 4.4.25: QH Relationship comparison between Edenderry Model and EPA Rating Curve at Kishawanny Weir

(b) Boyne Aqueduct (07007)

An OPW water level and flow gauge is located on the River Boyne at chainage 20954m within the Edenderry model. Rating information and gaugings are available for this station. However all gaugings are within bank level.

An attempt was made to verify the model to the recorded gaugings. A good correlation was achieved at low flows but as the water level approaches top of bank the scatter of the spot gaugings increases and the OPW rating curve diverges from the Edenderry model curve albeit to within acceptable limits (less than 400mm). Furthermore the modelled curve can also be considered to fall within the range of the spot gaugings although these display a large degree of scatter. It was observed that dense tree brush overhangs the river at this location and is likely to influence the QH relationship. A sensitivity test was carried out using different roughness coefficients to represent the brush however this had little impact on water levels in the model. The gauging station is also located between two large bridges. Another sensitivity test was carried out increasing the headloss created by the bridges. However, again, this had little influence on water levels. It is concluded that the model is calibrated to the observed spot gaugings within acceptable tolerances and that the model has a low sensitivity along this reach reducing the potential margin for error from model parameters.

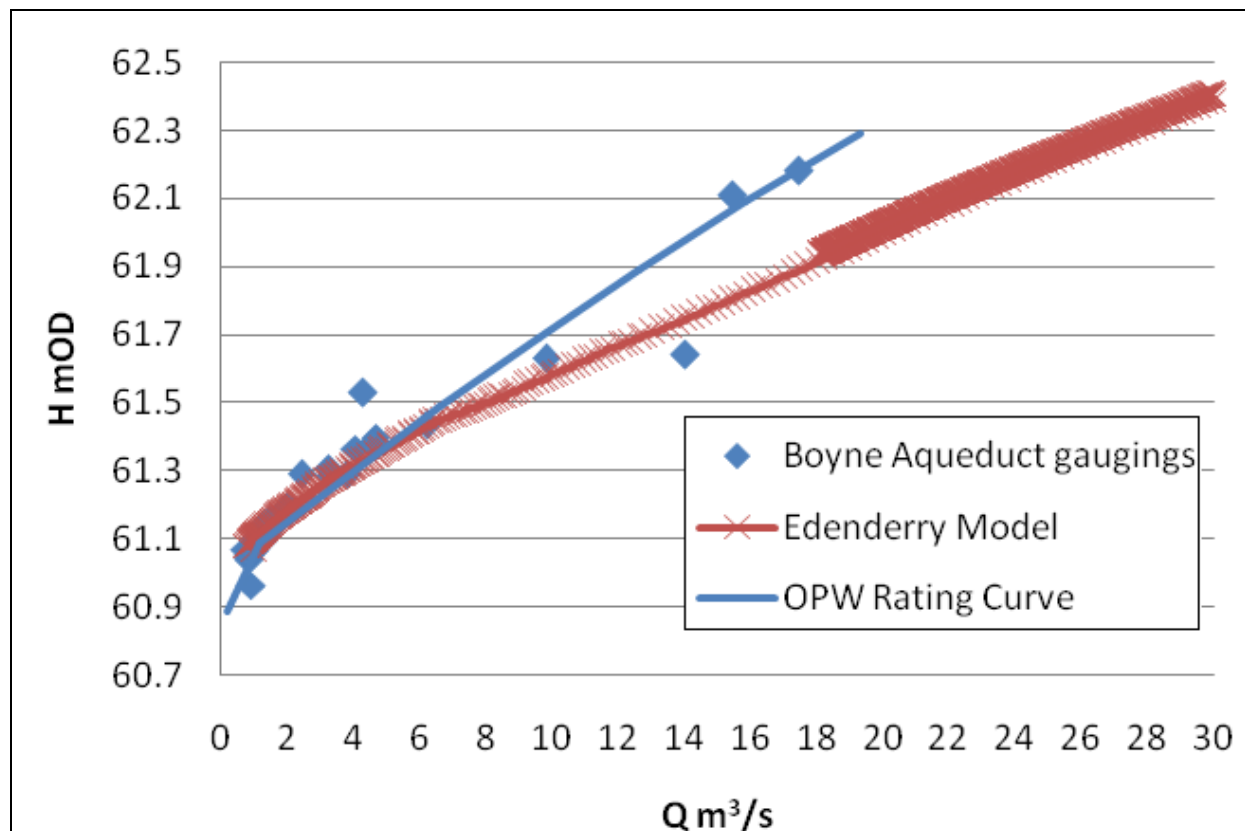


Figure 4.4.26: QH Relationship Comparison between Edenderry model and EPA Rating Curve at Boyne Aqueduct

Summary of Calibration

The historical records and the model outputs have both identified the same areas as being particularly susceptible to flood risk, namely the agricultural land around Kishawanny Bridge and other agricultural areas along with parts of Edenderry Golf Club.

The gauging stations data was found to be of limited use in model verification due to either the low quality of data or the lack of data in out of bank flow. The Boyne Aqueduct gauging station data did however support good verification for in bank flow at the downstream reach of the model.

While there is a limited amount of information with which to calibrate the model to, what information there is including the feedback from Local Authority review increases confidence in the model accuracy.

A comparison between estimated and modelled flows at HEP points was carried out and the details of which are presented in section 4.4.5 (5). A good correlation is found on the River Boyne but a noticeable difference is present for the Weavers Drain and tributary. An explanation and review of this difference is included in section 4.4.5 (5).

A mass balance plot was carried out to assess the difference between the discharge into the model and the volume of water stored with the discharge out of the model. Results showed a difference of 0.81%. While there is a small discrepancy between the flow in, the volume stored and the flow out it is within the acceptable limits as stated in the Environment Agency's Fluvial Design Guide.

The Edenderry Model seems to represent the historical information well however due to this data being limited a low confidence can only be assigned to it.

(5) Comparison of Flows:

Table 4.2 provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

Table 4.2: Modelled Flows and Check Flows

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m3/s)	Model Flow (m3/s)	Diff (%)
WEAVERS DRAIN TRIB 1285	10%	0.68	0.32	-53
07_108_2_RPS	1%	1.27	0.32	-75
	0.1%	2.30	0.32	-86
WEAVER DRAIN 23.395	10%	0.67	0.67	0
07_108_U	1%	1.26	1.26	0
	0.1%	2.28	2.28	0
WEAVER DRAIN 1454.22	10%	1.69	1.87	+11
07_265_3	1%	3.17	2.01	-37
	0.1%	5.74	3.49	-39
RIVER BOYNE 40	10%	6.69	6.69	0
07_1873_1_RPS	1%	11.42	11.44	0

	0.1%	18.70	23.23	+24
RIVER BOYNE 1652	10%	8.47	6.22	-27
07109_RPS	1%	14.46	10.62	-27
	0.1%	23.75	15.69	-34
RIVER BOYNE 20984.4	10%	55.25	47.10	-15
07007_RPS	1%	78.08	69.58	-11
	0.1%	106.78	100.73	-6

The table above provides details of flow in the model at every HEP inflow, check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

The Weavers Drain Tributary shows a rising percentage difference between the check flows and the modelled flows as the flood events get larger. This is due to the long culvert located at the downstream reach of the watercourse which restricts the peak flow at the confluence and causes out of bank flooding upstream which flows overland to the Weavers Drain during the 0.1%AEP event or attenuation in the channel itself for the 1% and 10% AEP events.

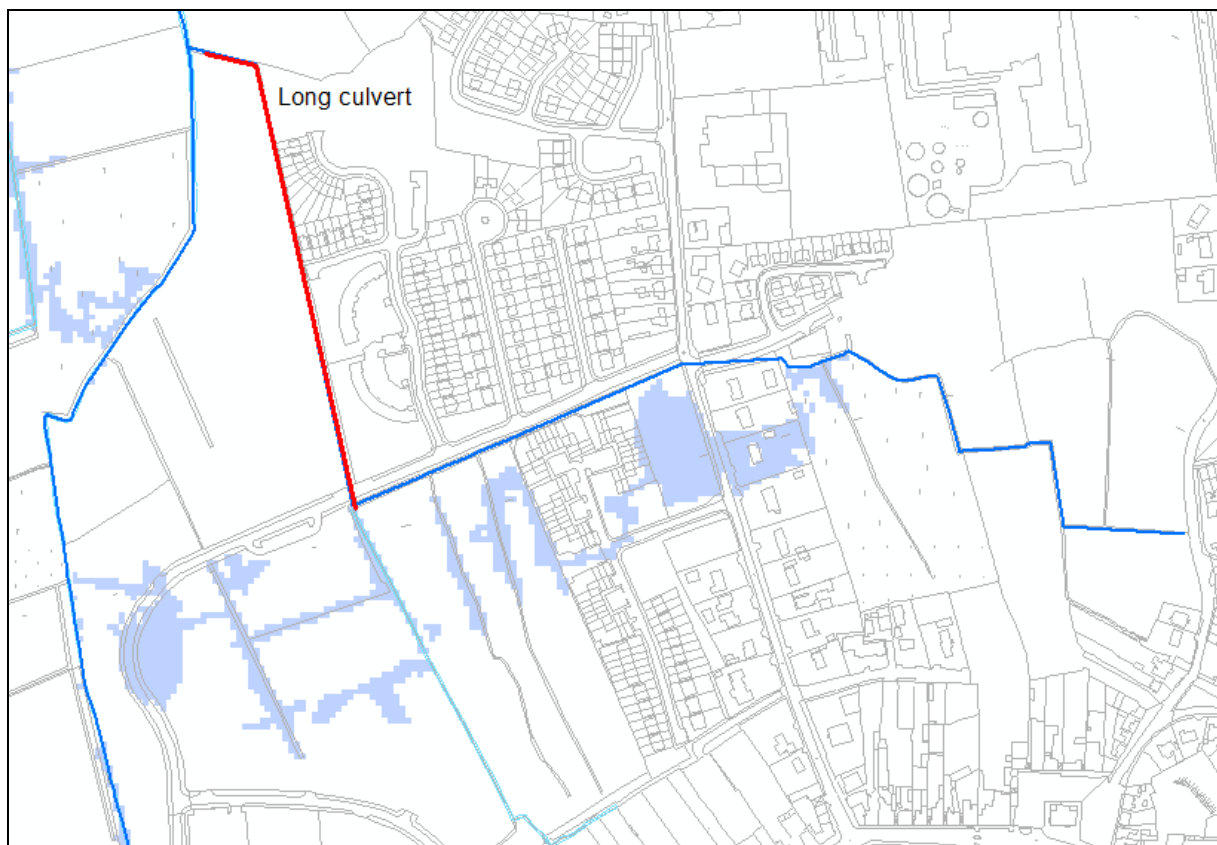


Figure 4.4.27: 0.1%AEP Flood Extent on the Weavers Drain Tributary

The upstream extent of Weavers Drain shows good correlation between the check flows and modelled flows however the downstream extent at chainage 1454m shows a significant difference in flows during the 1% and 0.1% AEP events. This is due to the culvert located at chainage 1108m acting as a hydraulic control structure during a 1%AEP event. The flow is restricted through the culvert therefore reducing the

flow at the downstream extent of the Weavers Drain during the 1%AEP and the 0.1%AEP events. The channel upstream of the culvert stores the water during the 1%AEP event however during the 0.1%AEP event the water level rises sufficiently to cause out of bank flooding. This further reduces the flow at the downstream extent, therefore increasing the percentage difference between modelled and check flows.

(6) Other Information:

The watercourses in the Edenderry model are part of the arterial drainage scheme. As such they were assessed and modified to provide improved drainage to land. Part of this scheme involved delineating the benefiting land. This involved a walkover survey by land valuers to identify flood prone and marshy land that would benefit by the improved drainage. An indication can therefore be obtained from the ADS benefiting lands as to the extent of a river's floodplain and therefore be used as verification on the Edenderry model. A review was carried out for the River Boyne and Weavers Drain and tributary.

Generally the flood extents follow the ADS benefitting lands although extents differ where the model does not take account of certain watercourses in addition it should be noted that the delineation of ADS benefitting land includes land improved by drainage and not solely flood prone land.

At the upstream reach of the model the modelled flood extents agree with the ADS where there is a wide floodplain and where the floodplain is quite narrow and shown in the Figure 4.4.28.

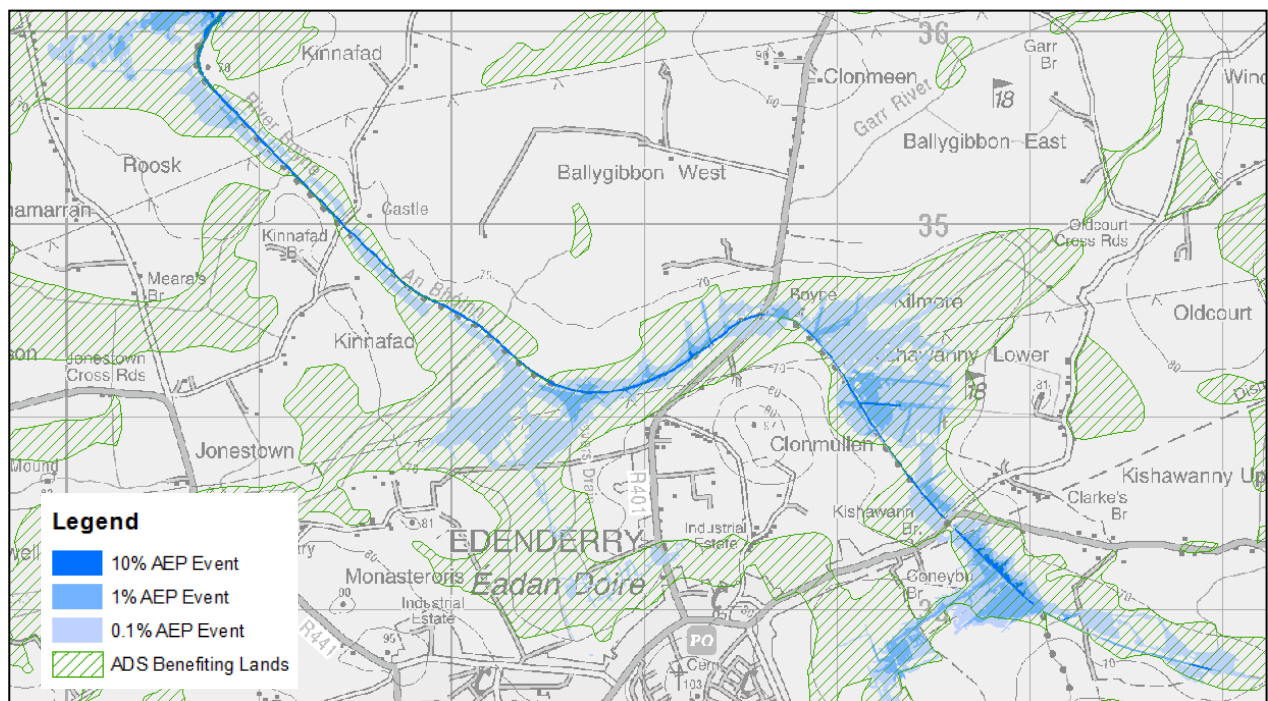


Figure 4.4.28: Comparison between ADS Benefitting Lands and Modelled Flood Extents at the Upper Reach of the Edenderry model

When considering the MPW in the downstream reach of the model the ADS benefiting lands becomes less useful as the larger area has been delineated making it difficult to identify the floodplain as shown in Figure 4.4.29.

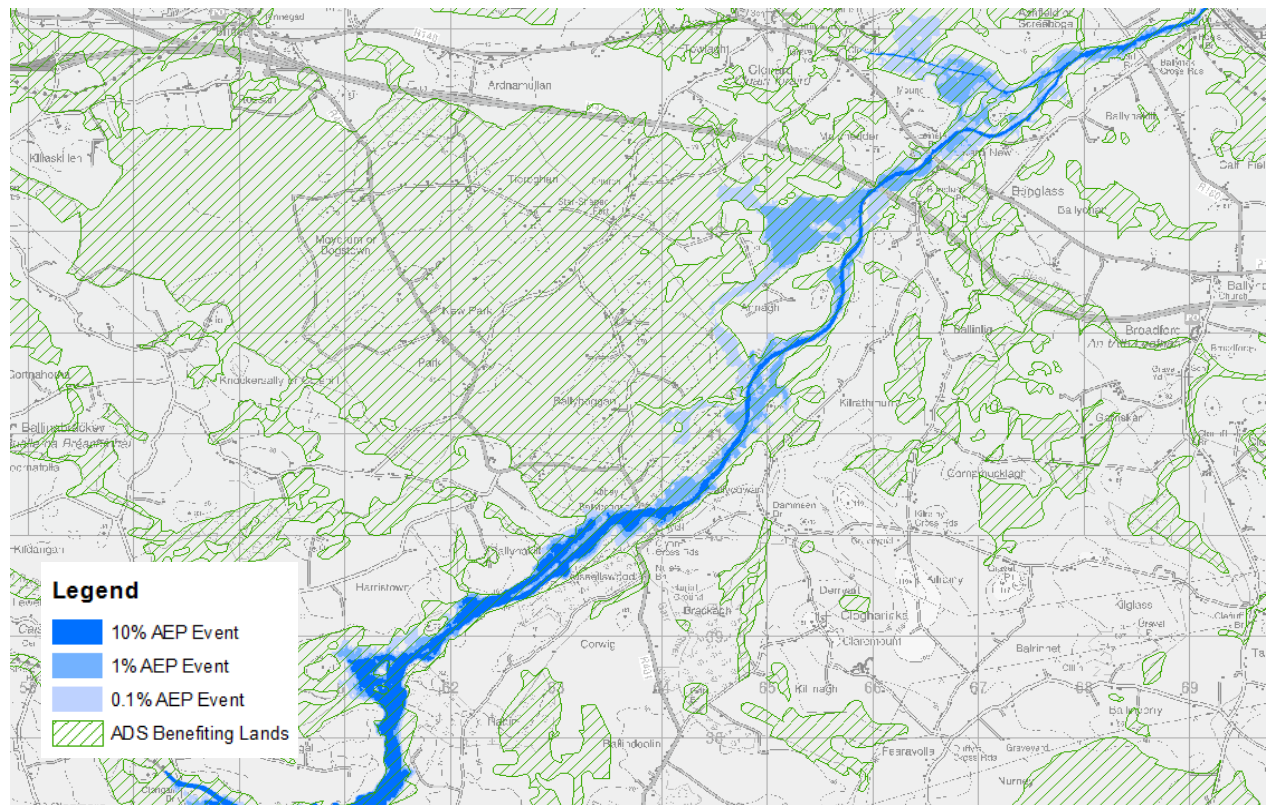


Figure 4.4.29: Comparison between ADS Benefitting Lands and Modelled Flood Extents at the Lower Reach of the Edenderry Model

4.4.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

(a) In order to represent the two bridges as shown in Figure 4.4.30 and 4.4.31 that are in close proximity to each other at chainage 1652m and avoid instabilities in the model, it was assumed that the upstream bridge which has the same cross sectional area would be the most hydraulically significant due to its location and could therefore be used to represent the two bridges as a single structure with a representative length which accommodates the overall length of both bridges.

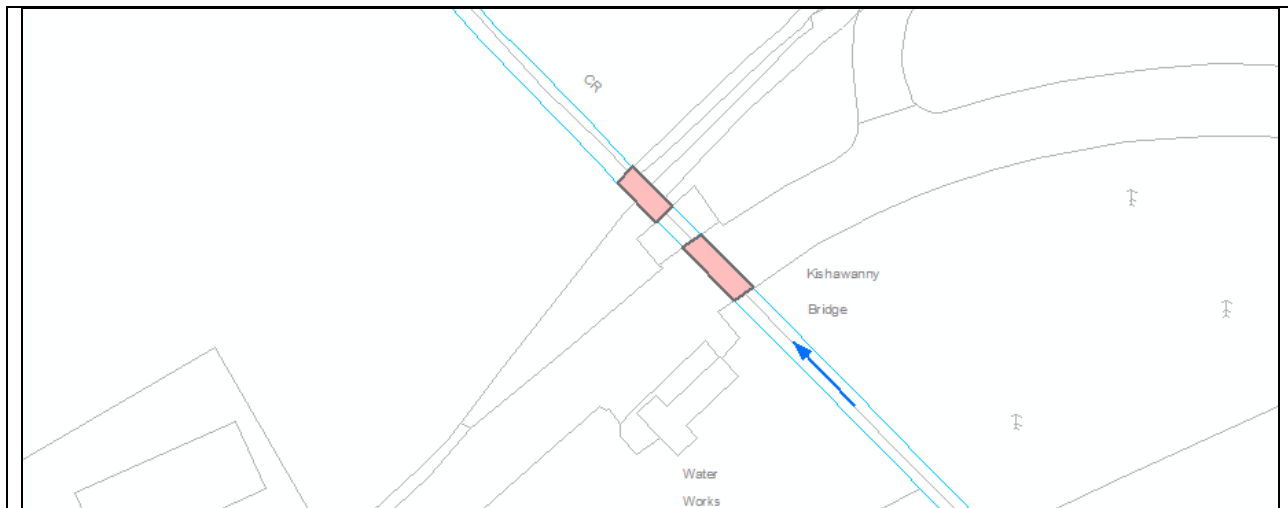


Figure 4.4.30: River Boyne Bridges (chainage 1652m)

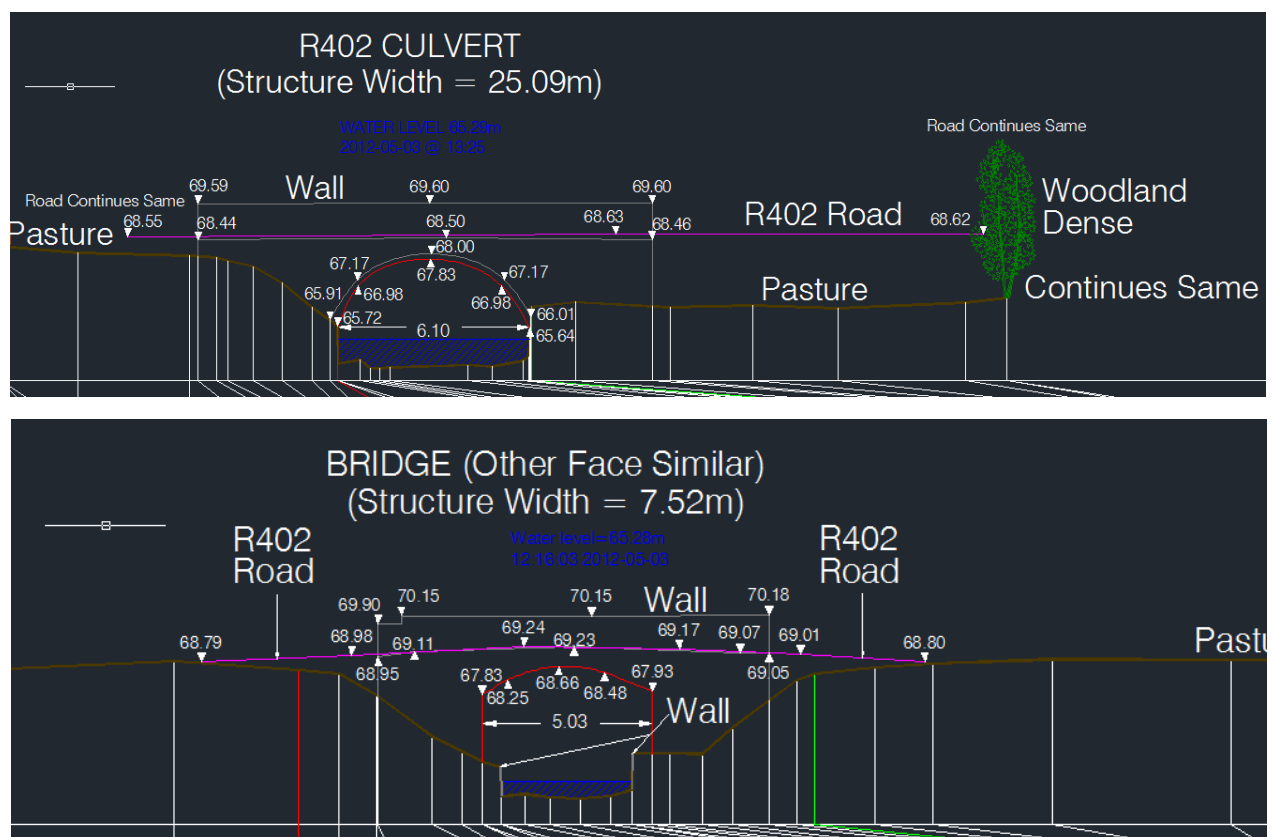


Figure 4.4.31: Cross Sections of River Boyne Bridges (chainage 1652m)

(2) Hydraulic Model Limitations and Parameters:

(a) A 2 second time-step for both the MIKE 11 and MIKE 21 models has been selected in order to achieve a successful model simulation for all design AEP events.

(b) It was noted that instability occurs at the upstream extent of the River Boyne during the 0.1% AEP event. However this has little impact on the flood extents produced by the flood event.

(c) The cross sections on the MPW (Ch8,475m – Ch21,458m) were extended beyond the limit of the topographical survey using the NDHM. A review was carried out post cross section extension to compare the topographical survey level with the NDHM. A good agreement was found between the two survey datasets and no adjustments were required to the extended cross sections.

Hydraulic Model Parameters:

MIKE 11

Timestep (seconds)	2
Wave Approximation	High Order Fully Dynamic
Delta	0.85

MIKE 21

Timestep (seconds)	2
Drying / Flooding depths (metres)	0.2 / 0.3
Eddy Viscosity (and type)	0.1 (Flux Based)

MIKE FLOOD

Link Exponential Smoothing Factor (where non-default value used)	All Rivers, 0.8
Lateral Length Depth Tolerance (m) (where non-default value used)	Default, 0.1

(3) Design Event Runs & Hydraulic Model Handover Notes:

The model shows that there is significant out of bank flooding in the upper reaches of the River Boyne on both banks which attenuates the flow further downstream. The flow backs up the connecting drains flooding other discrete areas also as shown in Figure 4.4.32.

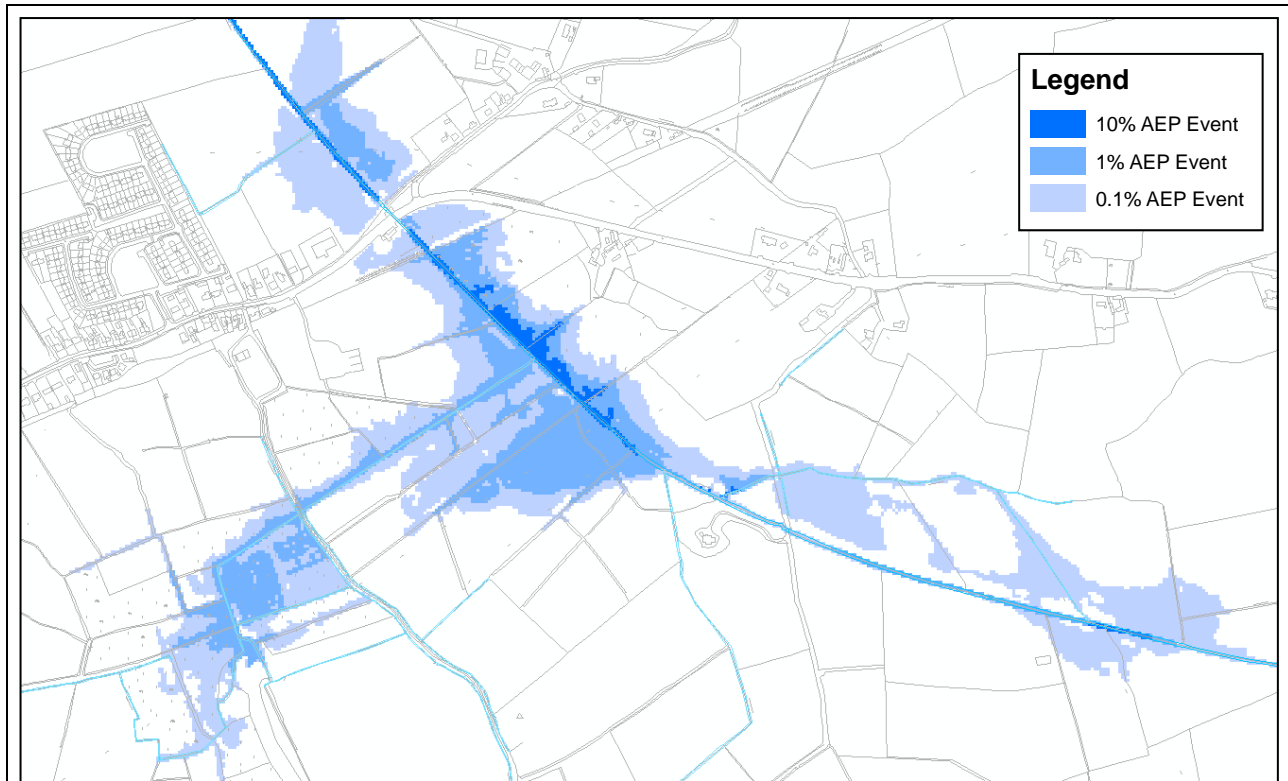


Figure 4.4.32: River Boyne Flooding Attributed to the Backing up of Surrounding Drains

The Kishwanny bridges and weir are significant hydraulic control structures contributing to the upstream attenuation previously mentioned. An upstream level of 67.3mOD is reached during the 0.1%AEP flood event but this does not surcharge the bridge completely. The backwater effect can be viewed in the long section presented in appendix D.

The model confirms the flood risk to Edenderry Golf Club with widespread flooding during the 1% AEP event.

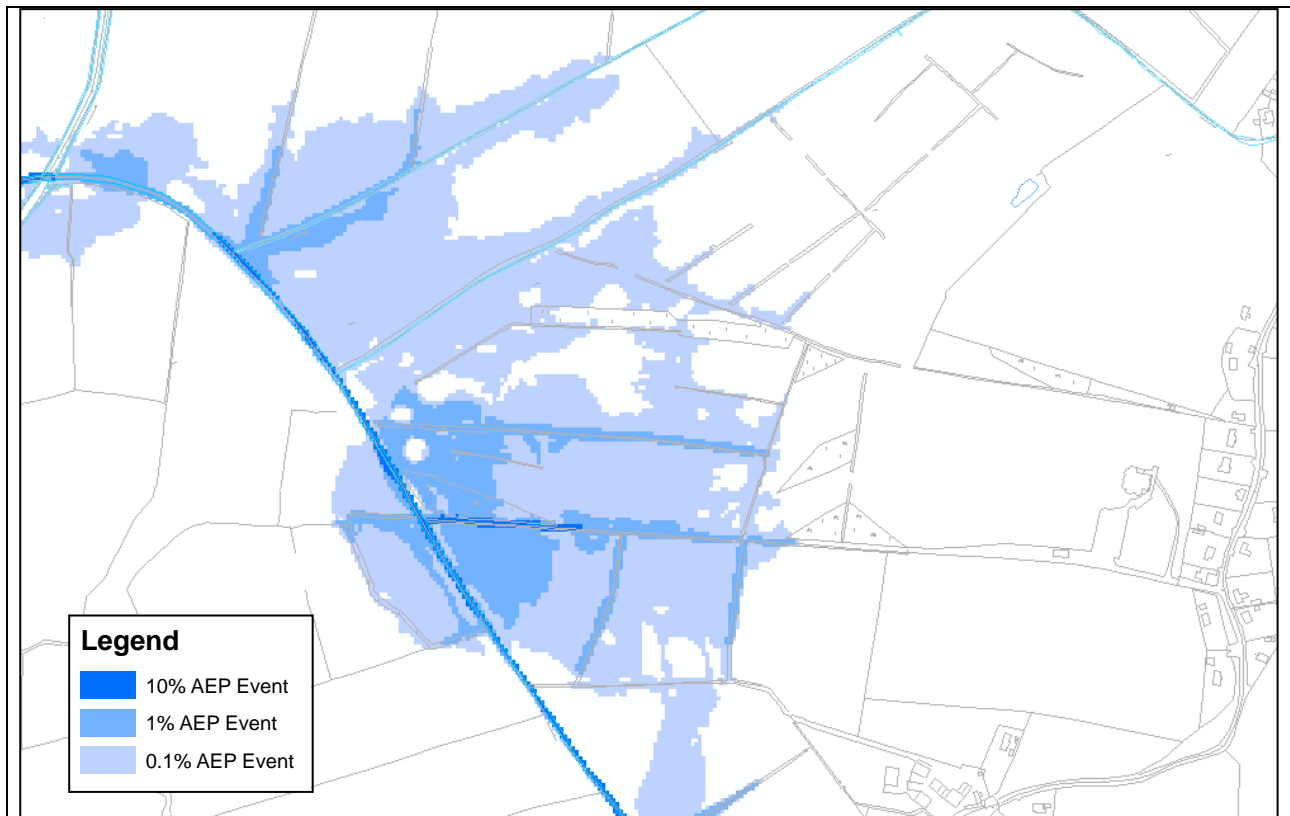


Figure 4.4.33: Flood Risk Extent (Edenderry Golf Club & Surrounding Area)

The other area of significant flooding occurs at the River Boyne/Weavers Drain confluence where both watercourses contribute to the flooding. The backwater effect on the Weavers Drain due to high water level in the River Boyne plays a significant role in this flooding mechanism.

The Weavers Drain is maintained and showed signs of dredging when the model survey was carried out. However the model shows that during the 0.1%AEP event out of bank flooding will occur at chainage 771m, close to the ring road, and flow into neighbouring drains before joining the flooding caused by the River Boyne. This is due to a culvert located downstream see Figure 4.4.34 which is restrictive to flow and causes a backwater effect. The 0.9m diameter culvert has a soffit level of 66.01mOD and water levels reach 66.64, 67.33 and 67.44mOD for the 10%, 1% and 0.1% AEP events respectively. The flood route does not affect any properties or other receptors apart from a single lane local road.

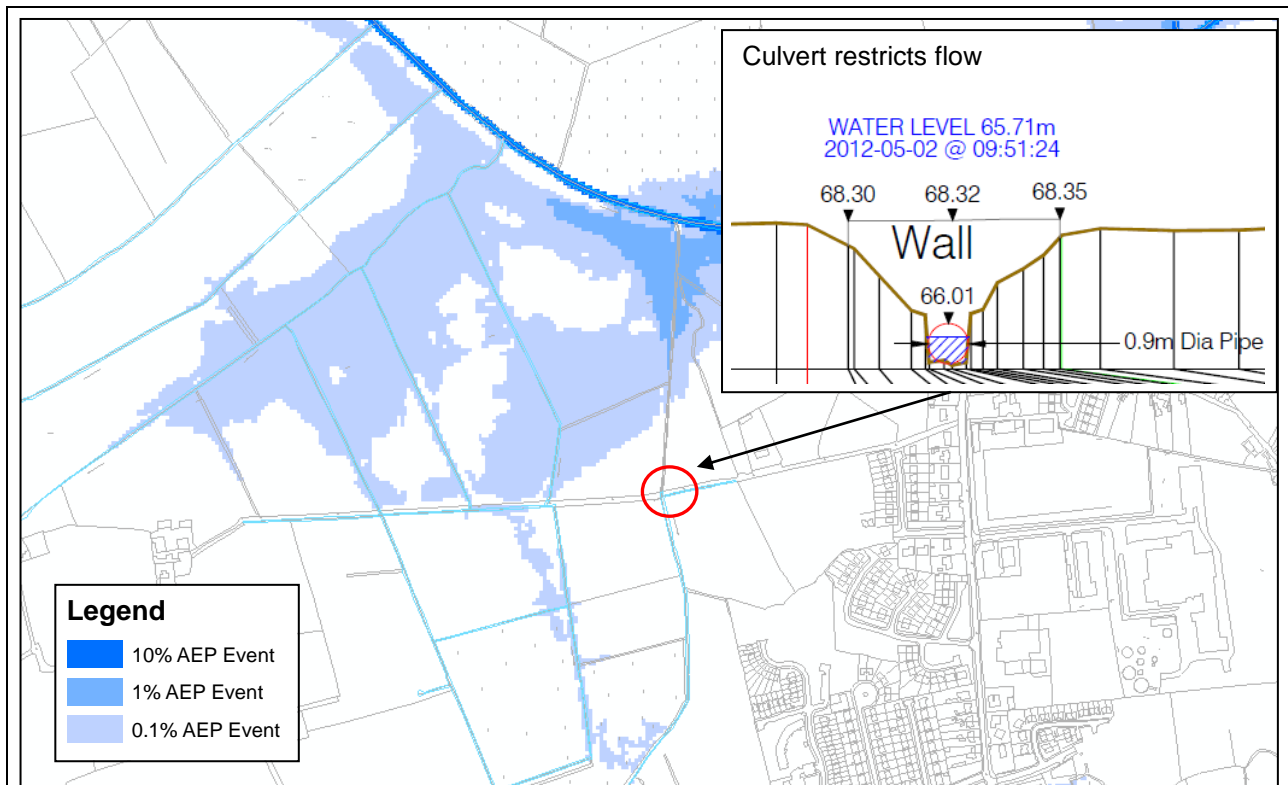


Figure 4.4.34: Location of the Weavers Drain Critical Structure (this undersize culvert contributes to out of bank flooding upstream)

The biggest flood risk comes from Weavers Drain Tributary. At chainage 862m the watercourse turns 90° and enters a long culvert. These combined head loss causes out of bank flooding which causes ponding adjacent to the ring road before it overtops the road and enters the Weaver Drain. Further upstream out of bank flooding occurs at chainage 440m (upstream of Carrick Road) during the 0.1% AEP event which flows around one property, over Carrick Road and around Carrickhall Close to the fields beyond.

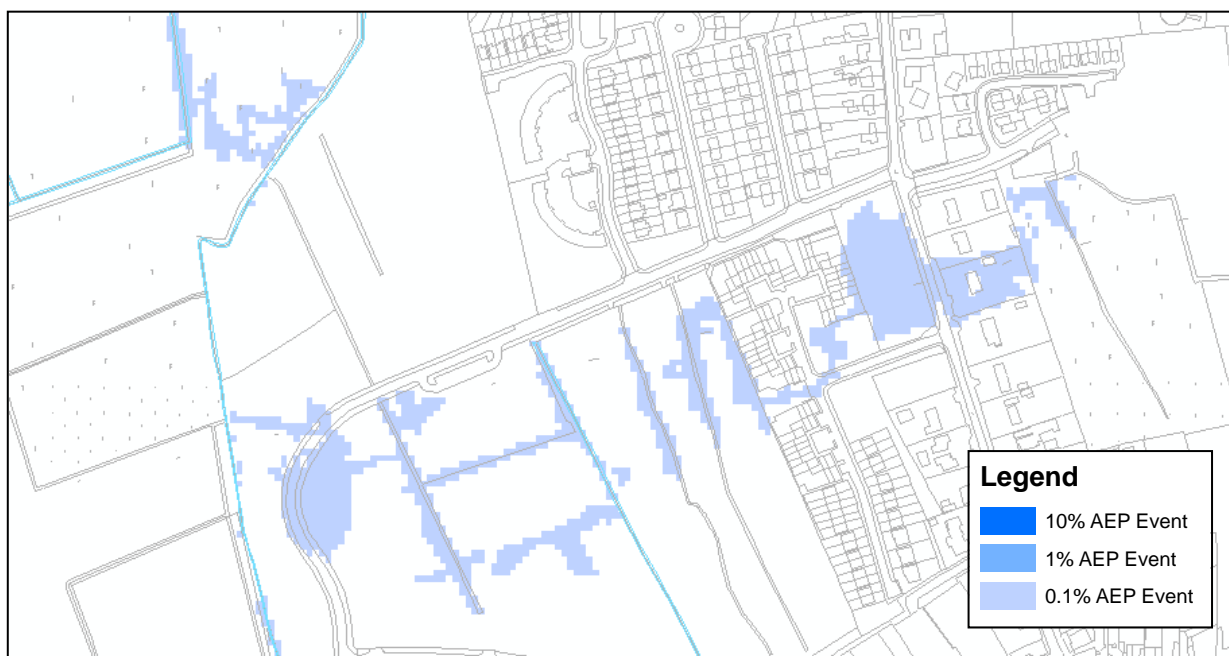


Figure 4.4.35: Weaver Drain Tributary, Flood Flow Path

(4) Hydraulic Model Deliverables:

Please see Appendix D for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Mark Wilson
Model Reviewed by:	Stephen Patterson
Model Approved by:	Malcolm Brian

4 HYDRAULIC MODEL DETAILS

4.5 JOHNSTOWN BRIDGE

Name	Local Authority	Unique ID	SSA	Status	Date
Johnstown Bridge	Kildare/Meath	70849	AFA	Final	08/05/2017

4.5.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600Rp0001_Flood Risk Review) highlighted Johnstown Bridge as an Area for Further Assessment for fluvial flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

The Johnstown Bridge model consists of the Blackwater (Enfield) River from Johnstown Bridge to its confluence with the River Boyne approximately 12 km to the north at Donore along a tributary of the Blackwater River, the Fear English River and its associated mill race. The full extent of modelled watercourses lies within the Arterial Drainage Scheme (ADS) and while these watercourses may be subject to dredging this model represents the current status as of June 2012.

There are two gauging stations along this reach of the Blackwater. The Johnstown Bridge gauging station has level data only. Castlerickard gauging station which is over 10 km downstream of the AFA extents has flow and level data. The gauging station, Castlerickard (07003 – OPW) was found during the rating review to have some uncertainty in the rating at Q_{med} despite having an FSU classification of B for the period post arterial drainage scheme (1975 onwards).

The main flood risk to the Johnstown Bridge AFA is from the Fear English River, a tributary of the Blackwater River with its confluence point at Johnstown Bridge. The Fear English River system has a total contributing catchment area of 21.75 km² but is ungauged and as such estimates of Q_{med} have been derived from catchment descriptor methods.

The upper River Blackwater and the Fear English River along with the split of the Fear English River, which appears to be an old mill race, are identified as HPWs and are modelled as 1D-2D using the MIKE suite of software. The River Blackwater beyond the downstream extent of the AFA is a MPW and has been modelled as 1D again using the MIKE suite of software. The transition from HPW (1D-2D) modelling to MPW (1D) modelling is located approximately 750m downstream of the AFA boundary. This allows for the WwTW located downstream of the AFA to be modelled within the 1D-2D domain.

No models exist upstream of the Johnstown Bridge model, the Longwood model is located downstream and models an overlapping reach of the Blackwater River from Longwood to the confluence with the River Boyne. Whereas the Longwood model has modelled this reach of the Blackwater River in a combination of

1D-2D and 1D modelling, the Johnstown Bridge model has modelled it using 1D modelling and is therefore considered to be less accurate. The presentation of the Johnstown Bridge flood extents therefore ends just upstream of the Longwood model and the remainder of the Blackwater River is presented in the Longwood model.

The Johnstown Bridge model was also used to provide the upstream flow hydrograph in the Blackwater River for the Longwood model. The downstream boundary condition for both the Johnstown Bridge model and the Longwood model is based on the water level hydrograph generated by the Trim model at the River Boyne/Blackwater River confluence. An illustration of the two models can be seen in Figure 4.5.1.

(2) Model Reference:		HA07_JOHN5
(3) AFAs included in the model:		Johnstown bridge
(4) Primary Watercourses / Water Bodies (including local names):		
<u>Reach ID</u>	<u>Name</u>	
0735	BLACKWATER	
0737	FEAR ENGLISH	
0740	SPLIT OF FEAR ENGLISH	
It should be noted that no local watercourse names were provided as part of the channel and structure survey of Reach ID 0740, and no local names could be identified from OSI or Google mapping. The watercourse name given above was therefore allocated for modelling purposes.		
(5) Software Type (and version):		
(a) 1D Domain: MIKE 11 (2011)	(b) 2D Domain: MIKE 21 rectangular mesh (2011)	(c) Other model elements: MIKE FLOOD (2011)

4.5.2 Hydraulic Model Schematisation

(1) Map of Model Extents:
Figure 4.5.1 and Figure 4.5.2 illustrate the extent of the modelled catchment, river centre line, HEP locations and AFA extents as applicable. The Blackwater catchment contains 2 Upstream Limit HEPs, 1 Downstream Limit HEP, 2 Intermediate HEPs, 7 Tributary HEPs and 1 Gauging Station HEPs.

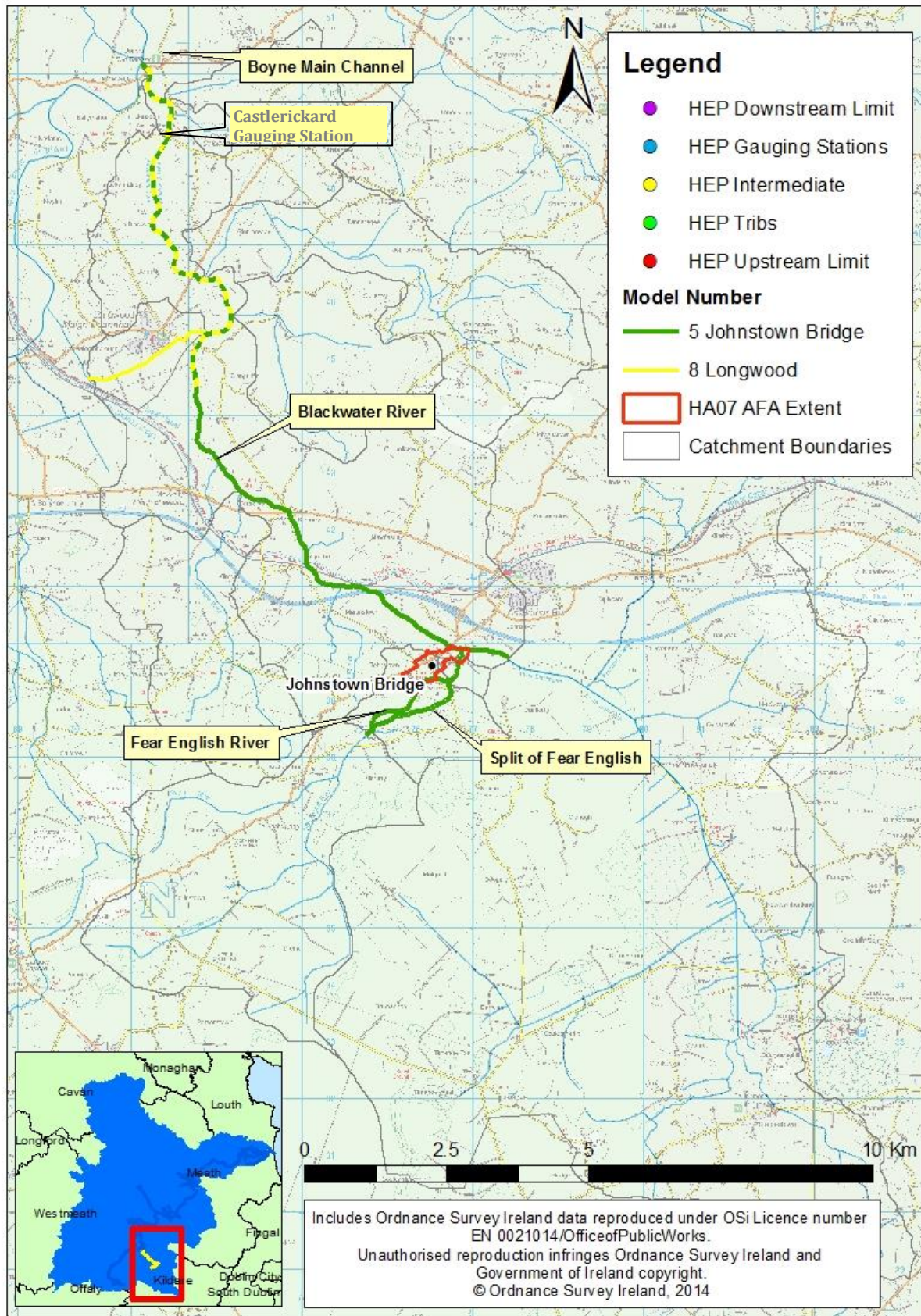


Figure 4.5.1: Johnstown Bridge Model Overview

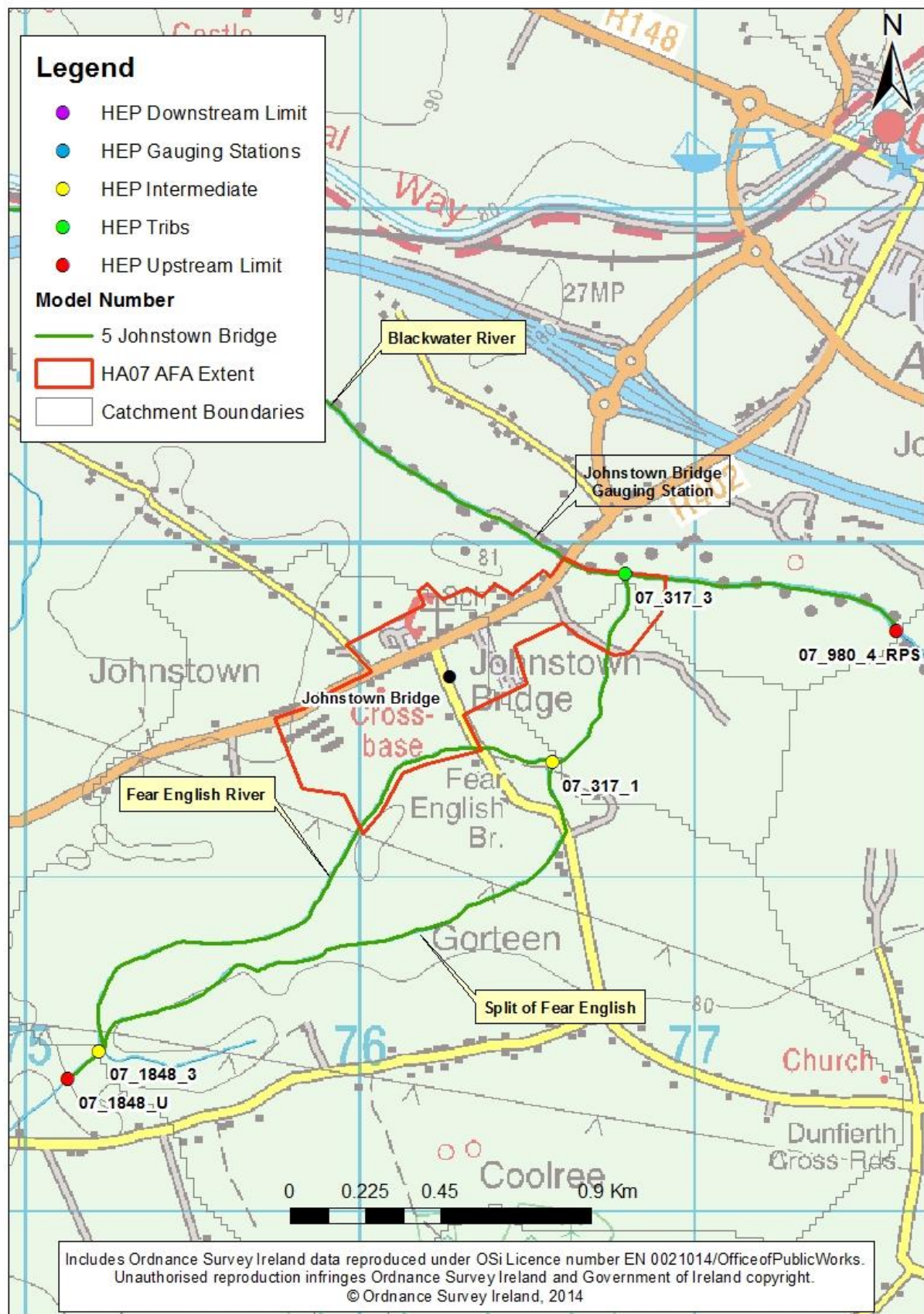


Figure 4.5.2: Johnstown Bridge Model AFA Overview

Figure 4.5.3 and Figure 4.5.4 show the location of the cross sections used in the model along with the location of the 2D links and the critical structures.

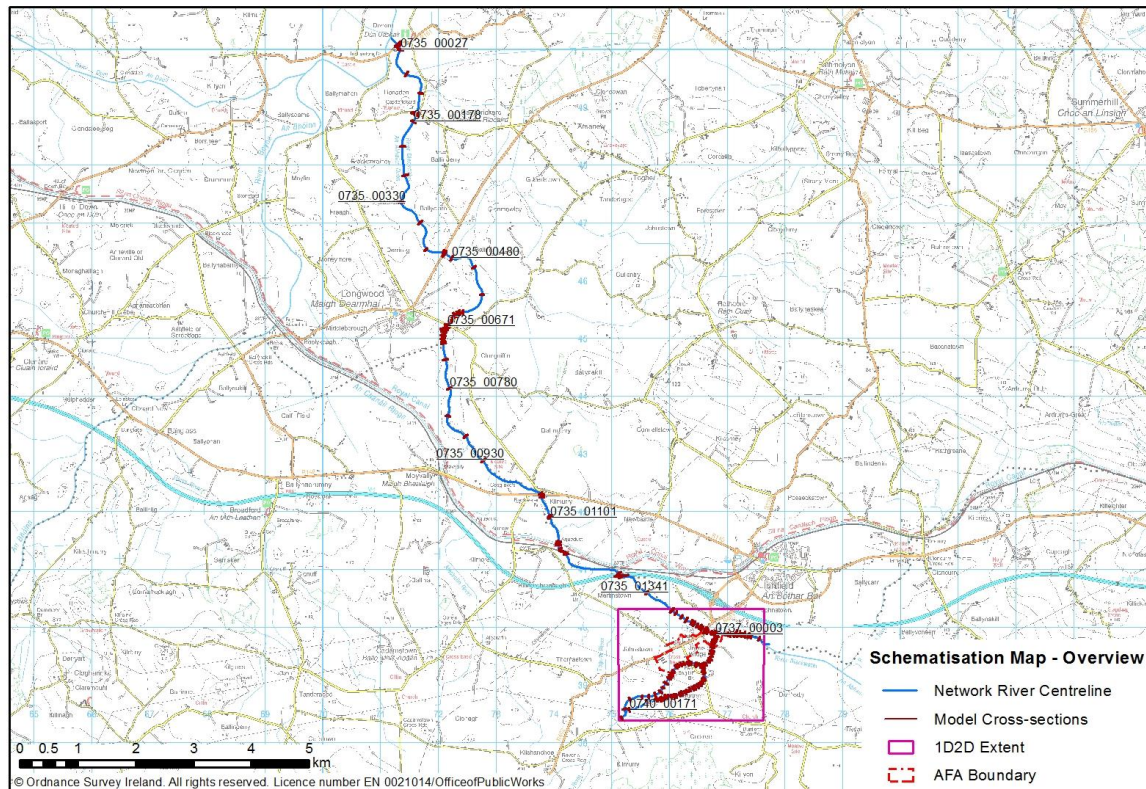


Figure 4.5.3: Overview of Model Schematisation

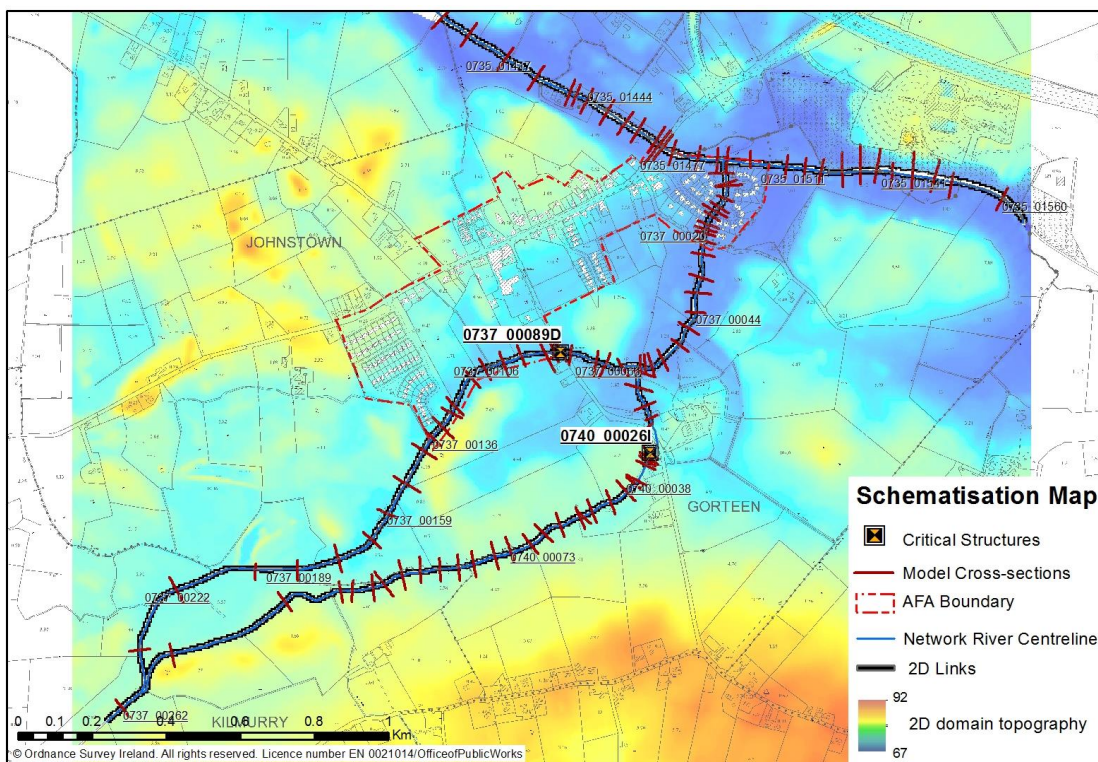


Figure 4.5.4: Model Schematisation AFA Overview

(2) x-y Coordinates of River (upstream extent):

River Name		x	y
0735	BLACKWATER	277601	239737
0737	FEAR ENGLISH	275126	238393
0740	SPLIT OF FEAR ENGLISH	275219	238471

(3) Total Modelled Watercourse Length:

20.26 km

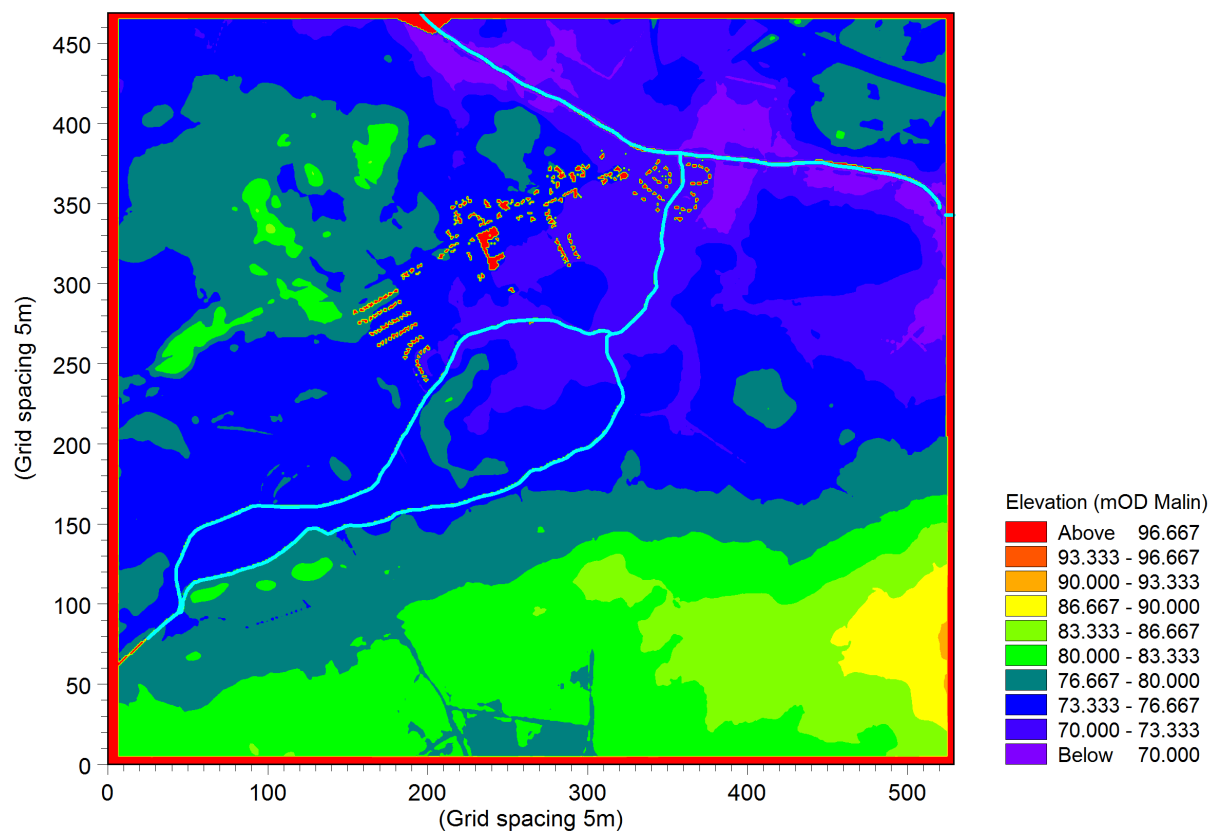
(4) 1D Domain only Watercourse Length:14 km
(approx)**(5) 1D-2D Domain
Watercourse Length:**6.26 km
(approx)**(6) 2D Domain Mesh Type / Resolution / Area:**Rectangular / 5 metres / 6.23 km²**(7) 2D Domain Model Extent:****Figure 4.5.5: Johnstown Bridge Model 2D Model Domain**

Figure 4.5.5 illustrates the bathymetry file showing the modelled extents of Johnstown Bridge and the general topography of the catchment. The 2D domain was generated using LiDAR survey data and created as a 5m grid rectangular mesh. No post processing was required to the bathymetry file.

(8) Survey Information		
(a) Survey Folder Structure:		
First Level Folder	Second Level Folder	Third Level Folder
Murphy_E07_M05_WP2_0735_120625 Johnstown Bridge Murphy: Surveyor E07: Hydrometric area 7 M05: Model Number 5 WP2: Work Package 2 0735: Reach ID 120625: Date issued (25 Jun12)	V0_0735_GIS and Floodplain Photos	Flood_Plane_Photos_and_Shapfiles
	V0_0735_Photos_Videos	0735_00016_DN
	V1_0735_Ascii	
	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	
(b) Survey Folder References:		
Reach ID	Name	File Reference
0735	BLACKWATER	Murphy_E07_M05_WP2_0735_120625
0737	FEAR ENGLISH	Murphy_E07_M05_WP2_0737_120625
0740	FEAR ENGLISH SPLIT	Murphy_E07_M05_WP2_0740_120625
(9) Survey Issues:		
No survey queries arose.		

4.5.3 Hydraulic Model Construction

All structures within the 1D model that have potential to overtop, such as bridges and culverts, were simulated using an overtopping weir representative of the associated parapet or deck. This allows for flood water to overtop a surcharged structure and avoids creating an artificially high backwater profile. Overtopping weirs were applied to all 29 bridges and culverts in the Johnstown Bridge model.	
(1) 1D Structures (in-channel along modelled watercourses):	See Appendix E Number of Bridges and Culverts: 29 Number of Weirs: 0



Figure 4.5.6: Photograph of Critical Structure on the Fear English River

Critical structure identified on the Fear English River at chainage 1783m which causes out of bank flooding due to capacity restrictions.



Figure 4.5.7: Photograph of Critical Structure on the Split of the Fear English River

Critical structure identified on the Split of Fear English River at chainage 1620m. The culvert inlet is a 0.9m diameter which restricts to 0.175m x 0.8m outlet. While starting as inlet control the culvert fills and surcharges under large flows causing a backwater effect upstream.

(2) 1D Structures in the 2D domain (beyond the modelled watercourses):		none		
(3) 2D Model structures:		none		
(4) Defences:				
Type	Watercourse	Bank	Model Start Chainage (approx.)	Model End Chainage (approx.)
Informal - None				
(5) Model Boundaries - Inflows:				
Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report - Section 4.5 and Appendix D). The boundary conditions implemented in the model are shown below.				

	Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
1	Open	Inflow	FEAR ENGLISH RIVER	0	0		07_1848_U
2	Distributed Source	Inflow	FEAR ENGLISH RIVER	0	2019.45		Top-up between 07_1848_U & 07_317_1
3	Distributed Source	Inflow	FEAR ENGLISH RIVER	2019.45	2670		Top-up between 07_317_1 & 07_317_2
4	Open	Inflow	BLACKWATER	0	0		07_980_4
5	Point Source	Inflow	BLACKWATER	6120.86	0		07_985_9
6	Point Source	Inflow	BLACKWATER	9341.58	0		Top up between 07_40000_U & 07_40000_1
7	Point Source	Inflow	BLACKWATER	9933.6	0		07_948_3
8	Point Source	Inflow	BLACKWATER	10887.79	0		07_1688_6
9	Point Source	Inflow	BLACKWATER	14811.98	0		07_1363_6
10	Distributed Source	Inflow	BLACKWATER	0	14064		Top-up between 07_980_4_RPS & 07003_RPS
11	Distributed Source	Inflow	BLACKWATER	14064	15552		Top-up between 07003_RPS & 07_954_3
12	Open	Q-h	BLACKWATER	15718	0		07_954_3
13	Distributed Source	Inflow	SPLIT OF FEAR ENGLISH RIV	0	1871		Top-up between 07_1848_3 & 07_317_1

The hydrology was reviewed during the rating review at Castlerickard Gauging Station and was found to produce consistent results in the full Johnstown Bridge model. There was therefore no change made to the hydrology. Please view Section 4.5.5(2) which discusses model updates for Final deliverables.

A proportion of the distributed flow along the Fear English River was assigned to the Split of the Fear English River (64% of the distributed flow) to represent the proportion of the catchment draining to each watercourse. This is represented in the model by boundary ID "Top-up between 07_1848_U & 07_317_1" for the Fear English River and "Top-up between 07_1848_3 & 07_317_1" for the Split of the Fear English.

Figure 4.5.8 shows the Blackwater River inflow hydrograph during a 0.1%AEP event at HEP 07_980_4.

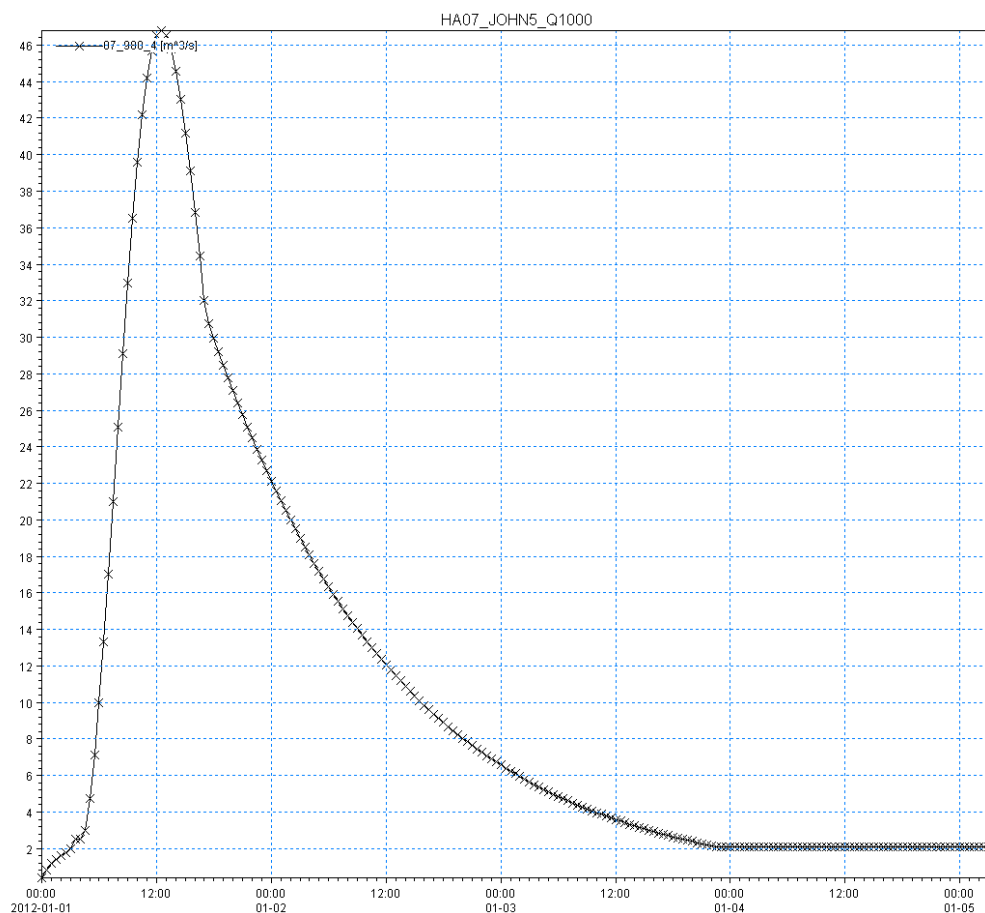


Figure 4.5.8: Inflow Hydrograph for the River Blackwater during a 0.1%AEP Event

**(6) Model Boundaries –
Downstream Conditions:**

The water levels generated from the adjacent Trim model at the River Boyne/River Blackwater confluence were taken as the downstream boundary condition on the River Blackwater. The influence of the River Boyne in creating a backwater effect along the River Blackwater was accounted for in this way. The figure below shows the downstream boundary water levels based on the River Boyne 0.1%AEP flood event.

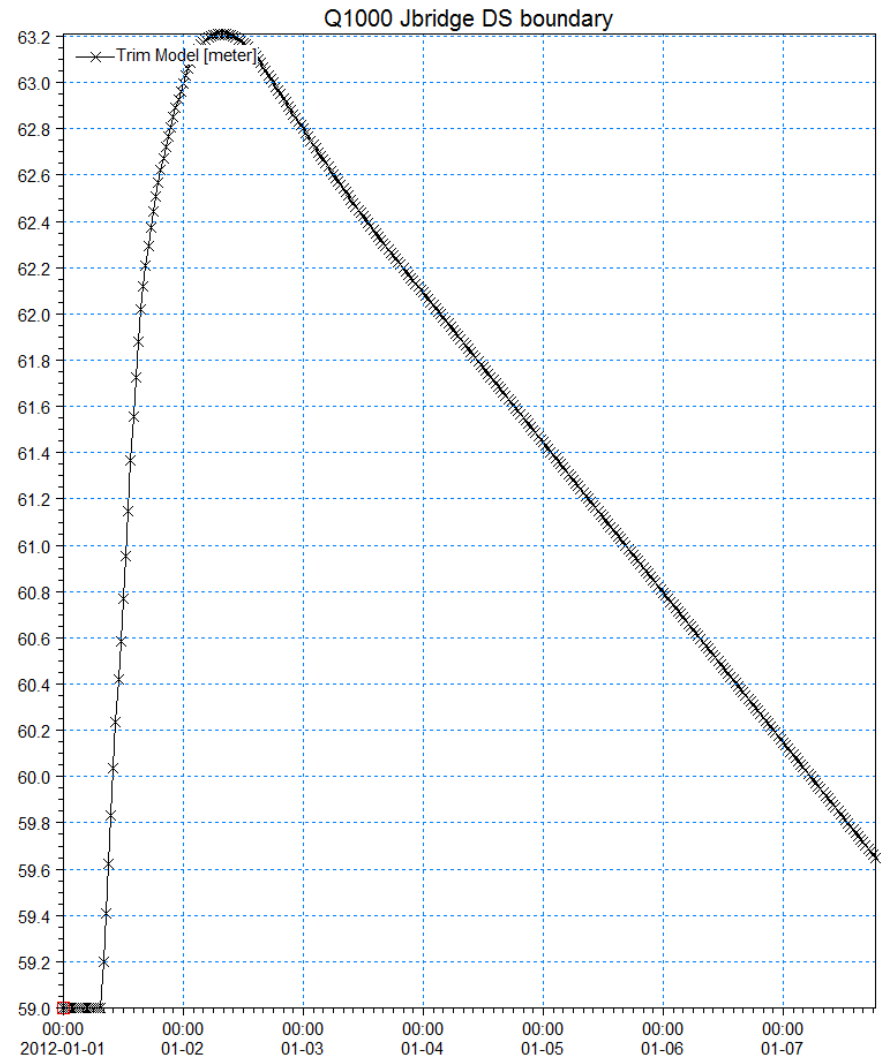


Figure 4.5.9: Downstream Boundary Water Levels based on the River Boyne 0.1%AEP Flood Event

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.040	Maximum 'n' value: 0.050
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.040	Maximum 'n' value: 0.070
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.035 (Inverse of Manning's 'M')	Maximum 'n' value: 0.045 (Inverse of Manning's 'M')

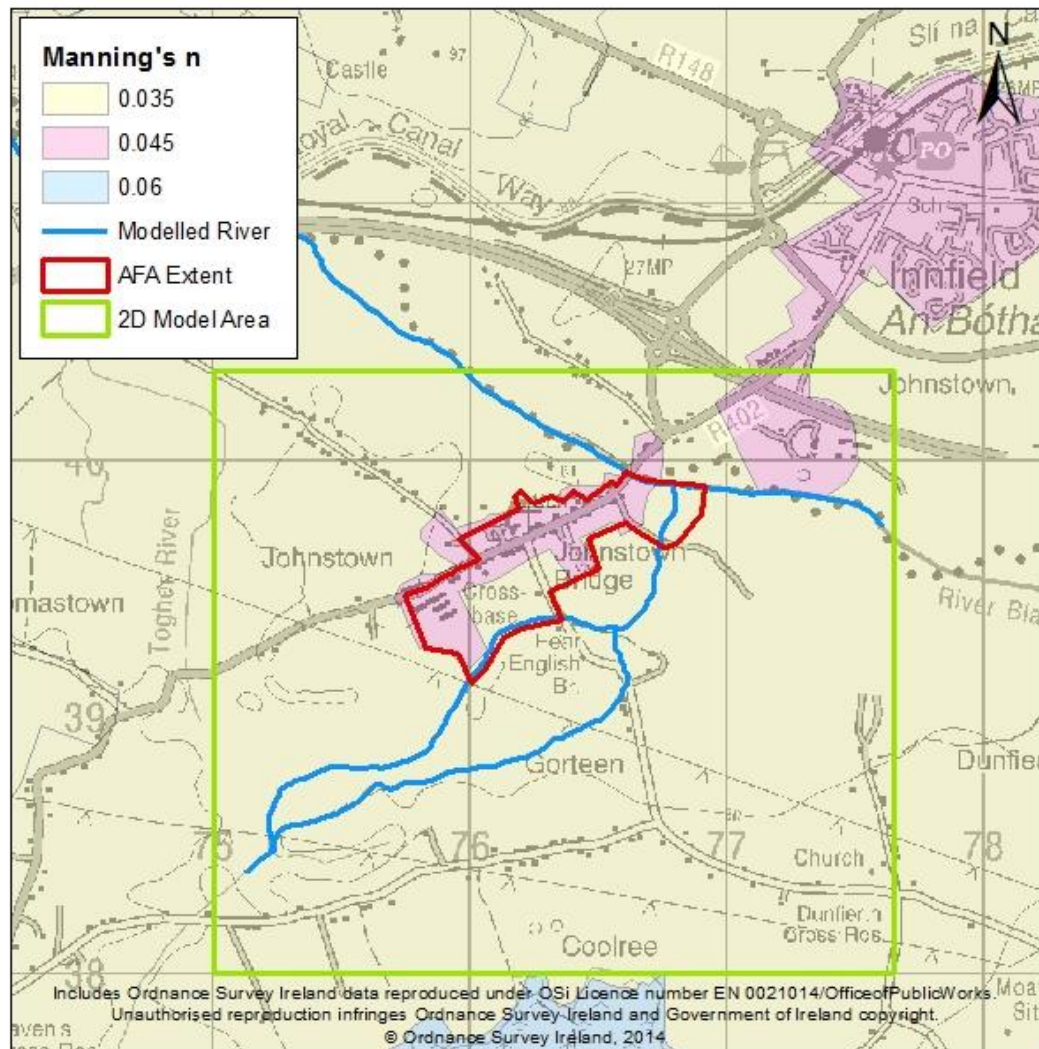


Figure 4.5.10: Floodplain Roughness Values in 2D Domain

This map illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset.

The surrounding rural area to Johnstown Bridge is assigned a Manning's n of 0.035 with the exception of a forested area on the north side of the River Blackwater which has a Manning's n value of 0.045. Johnstown Bridge is also assigned a Manning's n value of 0.045.

(d) Examples of In-Bank Roughness Coefficients

**Figure 4.5.11: Example of roughness coefficients
on the Blackwater River at cross section
0735_00016**

Manning's n value: 0.040

Clean winding with riffles



**Figure 4.5.12: Example of roughness
coefficients on Split of Fear English at cross
section 0740_00118**

Manning's n value: 0.050

Sluggish reach with dense vegetation



**Figure 4.5.13: Example of roughness coefficients
on the Blackwater River at cross section
0735_00666**

Manning's n value: 0.070 (on LHB)

Dense trees on left hand bank

4.5.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact 1% AEP hydraulic model within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a moderate increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.5.14. This indicates that the Johnston Bridge model demonstrates a high sensitivity to changes in roughness parameters. The increase in flood extents affects an additional 4 receptors, this accounts for a 200% increase compared to the design event. Properties are affected in the Dunfiert Park area of Johnston Bridge, especially those located within a close proximity to the Fear English watercourse.

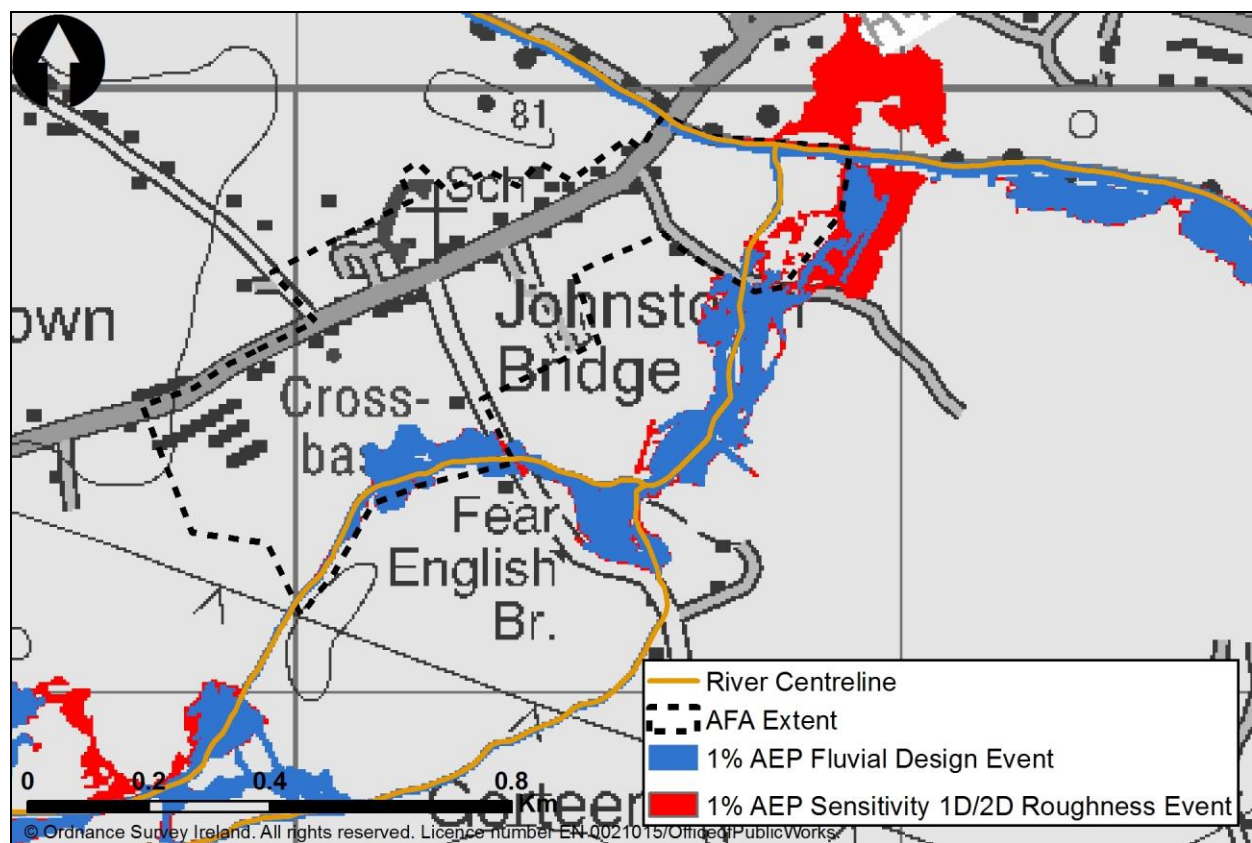


Figure 4.5.14 Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in 1D/2D Roughness

- b) Downstream boundary increase –The downstream boundary is located approximately 16km downstream of the AFA and is 8m lower than bed levels within the AFA. The downstream boundary parameter was evaluated as having no potential impact within the Johnstown Bridge AFA and therefore a sensitivity simulation was not required.

- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Johnston model was as having Medium/Low uncertainty/sensitivity (refer to the Hydrology Report IBE0600Rp0012 *for further detail*); factors of 1.37, 1.48 and 1.57 are applied to design flows for the sensitivity simulation. Figure 4.5.15 shows that the Johnston Bridge model has a high sensitivity to increased inflow parameters; this is reflected by the increase of flood extent. Significant increase in flood extents is located at the Dunferth Park, particularly those properties located within a close proximity to the Fear English River. These changes have a high impact on receptors as approximately 16 additional buildings are affected. This is a relative 800% increase when compared to the 1% AEP design results.

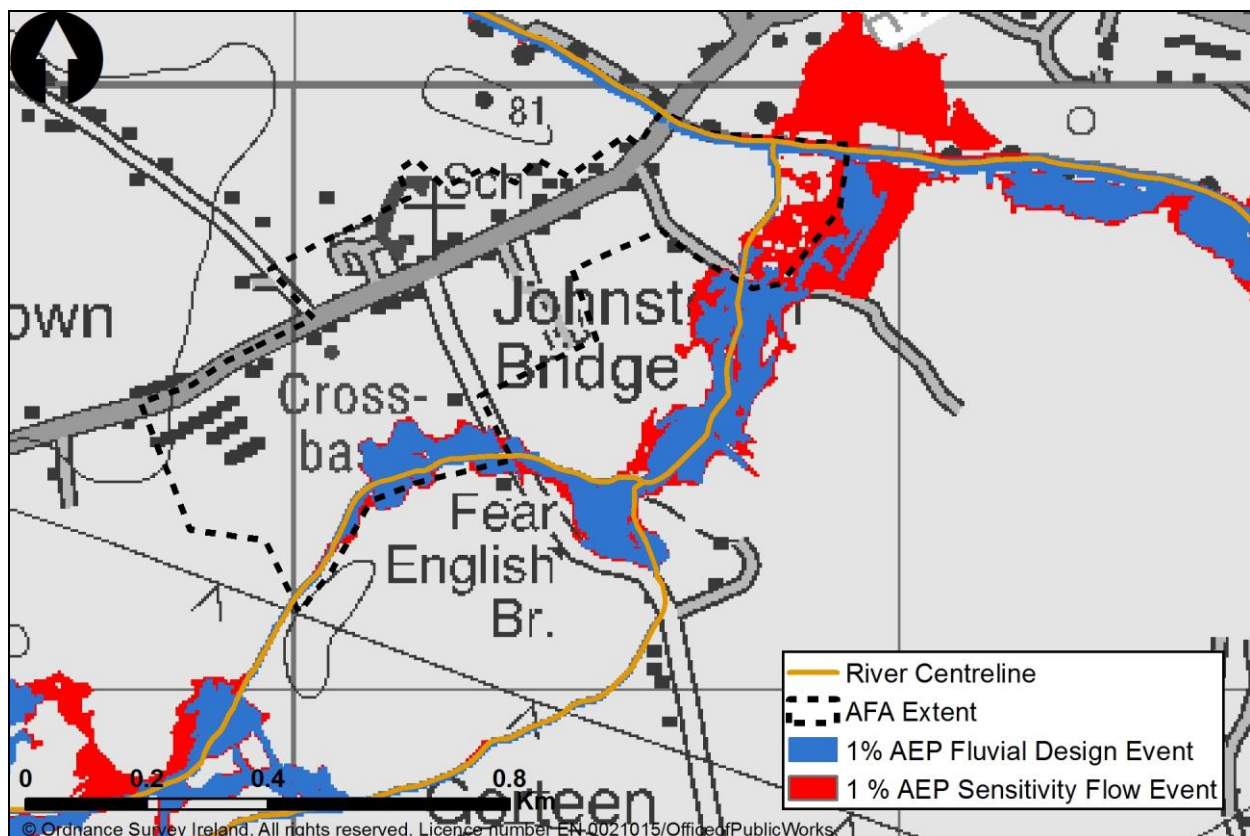


Figure 4.5.15: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event

- d) Afflux/Head loss at key structures – A number of simulations were carried out to assess the sensitivity of flood extents to changing the head loss coefficients of key structures. Two structures were assessed including 0737_00089D and 0737_00024J on the Fear English River. The Johnston Bridge model has shown it has a low sensitivity to parameter changes as there was little change in flood extents and no impact to receptors. Figure 4.5.16 is an example of the model results.

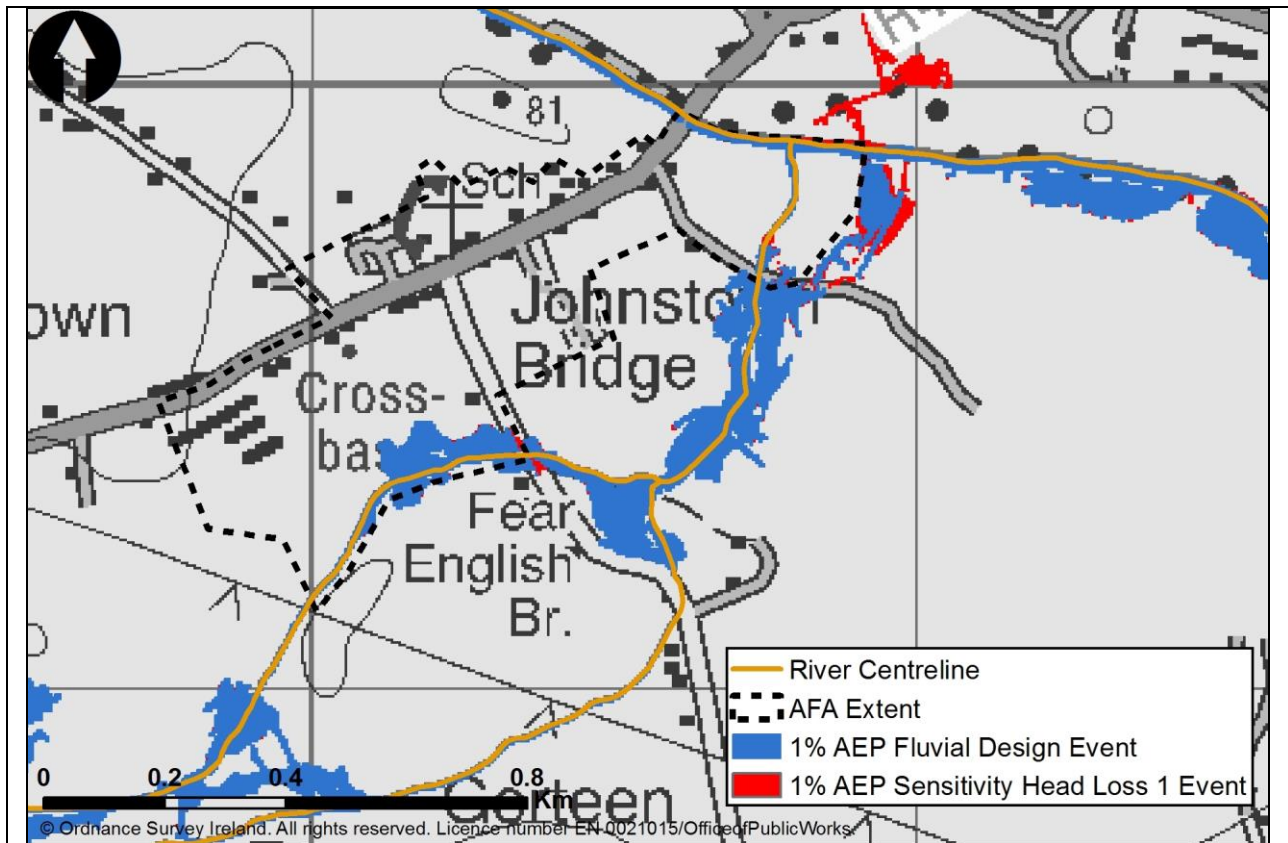


Figure 4.5.16: Comparison between 1% AEP Design Event and 1% AEP Sensitivity (Head Loss 1)

- e) Floodplain roughness decreased to lower bound values – The change in floodplain roughness values resulted in no alteration to flood extents within the AFA as shown in Figure 4.5.17. The Johnston Bridge model is considered to have a low sensitivity to 2D roughness parameters and no receptors are impacted within the AFA.

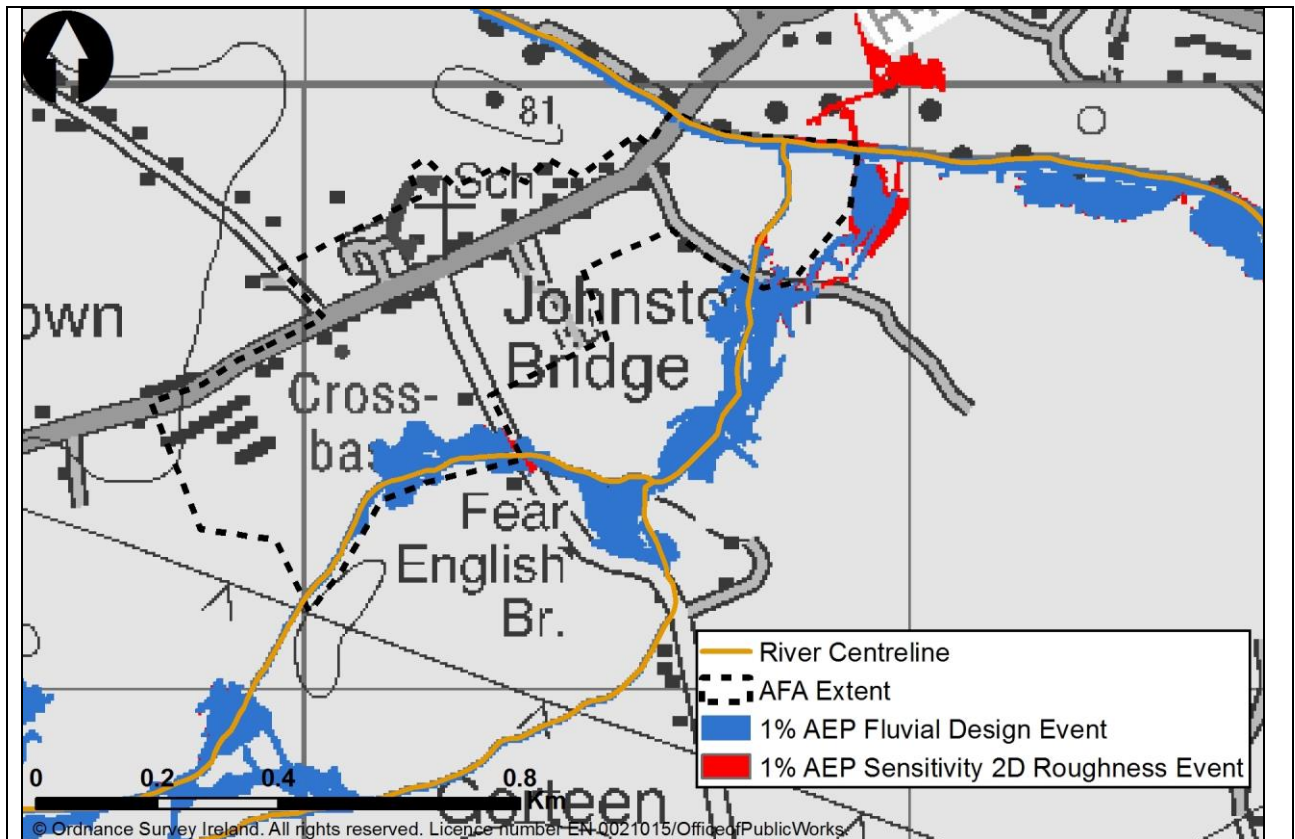


Figure 4.5.17 : Comparison between 1% AEP Design Event and 1% AEP Sensitivity 2D Roughness Event

Table 4.5.1 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Johnstone Bridge model. Of these parameters, the model is most sensitive to increase in model inflows and changes in the model roughness parameters. Table 8.1 in the Hydrology Report states that there is medium uncertainty/sensitivity associated with the hydrological inputs for this model. This moderate increase in flood extents results in a high impact to receptors within Johnstone Bridge. Impact is relatively high considering that a low number of properties (2) are impacted during the 1% AEP design event simulation.

The model is less sensitive to the other parameters, with the resulting analysis identifying relatively minor increases in flood extents with no further impact to receptors located within the AFA for the remaining sensitivity runs. Generally, the Johnstone Bridge model can be considered to have low sensitivity to changes in model parameters.

Table 4.5.1: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	Moderate	High
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	Moderate	High
1% AEP Sensitivity Head Loss 1 Event	Low	-
1% AEP Sensitivity Head Loss 2 Event	Low	-
1% AEP Sensitivity 2D Roughness Event	Low	Low

4.5.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0600Rp0004_HA07 Inception Report unless otherwise specified):

Recurring	Historical flood information is too ambiguous to inform the calibration process. The 10% AEP event shows limited flooding in Johnstown Bridge giving confidence that the recurring flooding referred to in the inception report is found elsewhere. Floodmaps.ie indicate that Enfield and Thomastown which are in the Johnstown Bridge area experience recurring flooding and it is to these areas that the inception report refers to.
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Summary of Calibration

The lack of historical flooding information at Johnstown Bridge gives little scope to calibrate the model to the larger flood events. However the model was calibrated to the spot gaugings at Castlerickard. This gauging station is subject to a rating review - please refer to IBE0600Rp0012_HA07 Hydrology Report. The Johnstown Bridge model's Q-h relationship was compared with that of the rating review model and the gauging station spot gaugings. The roughness values in the channel and at bridges were increased in order to agree with the initial rating review which showed good correlation with the spot gaugings as shown in Figure 4.5.18. Typically Manning's n values at bridges were increased to 0.035.

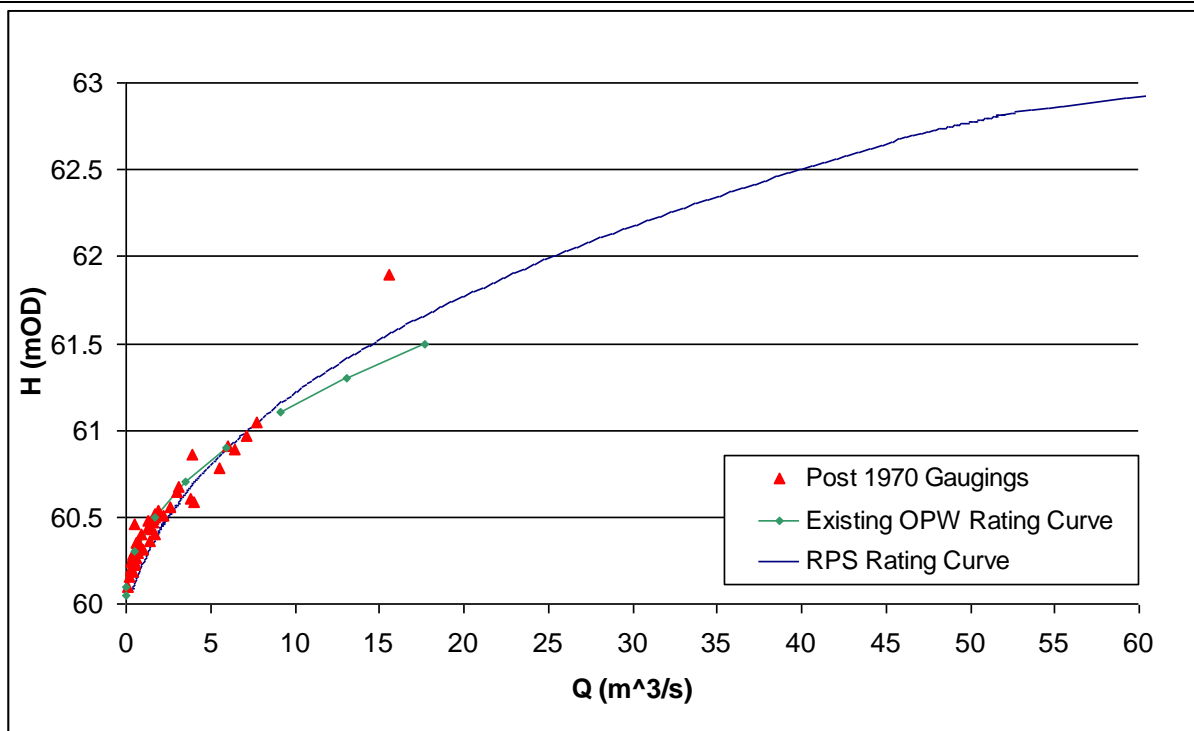


Figure 4.5.18: QH Relationship Comparison between Johnstown Bridge Model and OPW Rating Curve at Castlerickard Gauging Station

A comparison between estimated and modelled flows at HEP points was carried out and the details of which are presented in section 4.5.5 (5). A good correlation is found on the River Blackwater but a noticeable difference is present for the Fear English River. An explanation and review of this difference is included in section 4.5.5 (5).

A mass balance calculation was carried out and is discussed in section 4.5.6.

(2) Post Public Consultation Updates:

Following comments received during informal public consultation and formal S.I. public consultation periods in 2015, analysis of the results of the rating reviews and their impact on the hydrological analysis for the UoM07 watercourses was finalised. The updated inflow hydrograph at HEP 07_980_4 is shown in Figure 4.5.19. This resulted in the hydrology and mapping outputs being updated, however there were no changes made to hydraulics. A revised set of flood hazard and risk mapping has been issued as Final to reflect this change.

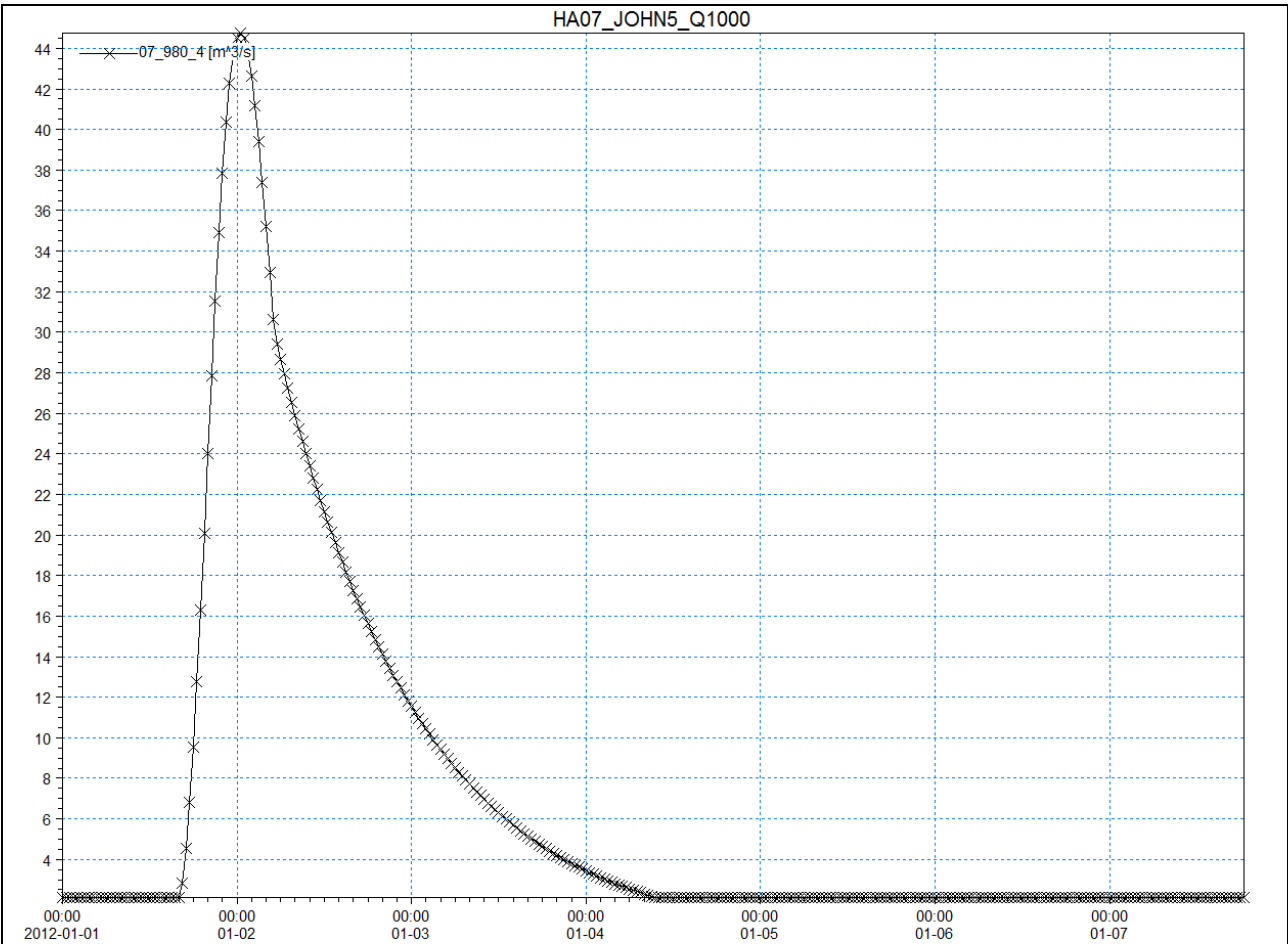


Figure 4.5.19: Inflow Hydrograph for the River Blackwater during a 0.1% AEP Event

(3) Standard of Protection of Existing Formal Defences:

Defence Reference	Type	Watercourse	Bank	Modelled Standard of Protection (AEP)
None				

(4) Gauging Stations:

There are two gauging stations within the model extent, one of which has water level and flow information available.

(a) Johnstown Bridge (07039)

An EPA water level gauge (07039) is located at Johnstown Bridge on the Blackwater at chainage 1302m. No flows are recorded at this site and no rating is available. It is therefore not possible to use the station to calibrate the model.

(b) Castlerickard (07003)

An OPW water level and flow gauge (07003) is located at Castlerickard on the Blackwater at chainage 14072m. Data is recorded continuously at 15min intervals. There are 65 spot water level and flow

gaugings recorded for the site from the 18th June 1975 to the 25th July 2001. This gauge has been given a B rating classification for the period from 1970 (i.e. flows can be determined up to the Q_{med} with confidence)

(5) Validation with MIKE NAM:

No comparison between NAM model results and observed data was possible due to lack of historical flood data.

(6) Comparison of Flows:

Table 4.5.2 provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

Table 4.5.2: Modelled Flows and Check Flows

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m3/s)	Model Flow (m3/s)	Diff (%)
FEAR ENGLISH RIVER 27.0765	10%	6.21	6.28	1.11
07_1848_U	1%	11.26	11.37	0.97
	0.1%	19.70	20.02	1.60
FEAR ENGLISH RIVER 2026.04	10%	7.40	7.73	4.27
07_317_1	1%	13.19	13.93	5.31
	0.1%	22.65	23.96	5.46
FEAR ENGLISH RIVER 2548.12	10%	7.53	7.73	2.59
07_317_3	1%	13.42	13.93	3.66
	0.1%	23.04	23.65	2.57
SPLIT OF FEAR ENGLISH RIVER 10	10%	-	0.61	-
07_1848_3	1%	-	1.85	-
	0.1%	-	2.89	-
BLACKWATER 45.473	10%	20.58	20.60	0.08
07_980_4_RPS	1%	30.95	32.66	5.53
	0.1%	44.75	50.79	13.49
BLACKWATER 15657.5	10%	33.03	34.65	4.90
07_954_3	1%	49.52	49.01	1.02
	0.1%	71.43	71.24	0.26

The table above provides details of flow in the model at every HEP inflow, check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided. The table shows that the flows in the River Blackwater are within approximately 6% of the estimated flows at the upstream boundary, with the exception of the 0.1% AEP event which has a difference of 13.5%. This percentage difference decreases to 5% at the downstream

extent.

The flow along of the Split of Fear English River was recorded in the table, however as this split takes place in close proximity to the Fear English River HEP inflow and that the distribution of flow along the Split would be difficult to estimate by hydrological analysis, no flow estimation was carried out and therefore no comparison is available.

The modelled flows along the Fear English River correlate well with the hydrological estimated flows.

However an increase in percentage difference (6% approx) at the downstream extent during the 1% and 0.1% AEP events can be seen. Given that the Fear English catchment is ungauged and therefore has an inherent uncertainty associated with hydrological estimation, flows with a 6% difference is considered to be adequately anchored to the HEP.

(6) Other Information:

The watercourses in the Johnstown Bridge model are part of the arterial drainage scheme. As such they were assessed and modified to provide improved drainage to land owners. Part of this scheme involved delineating the benefiting land. This involved a walkover survey by land valuers to identify flood prone and marshy land that would benefit by the improved drainage. A good indication can therefore be obtained from the ADS benefiting lands as to the extent of a river's floodplain. A review was carried out for the Blackwater River and the Fear English River.

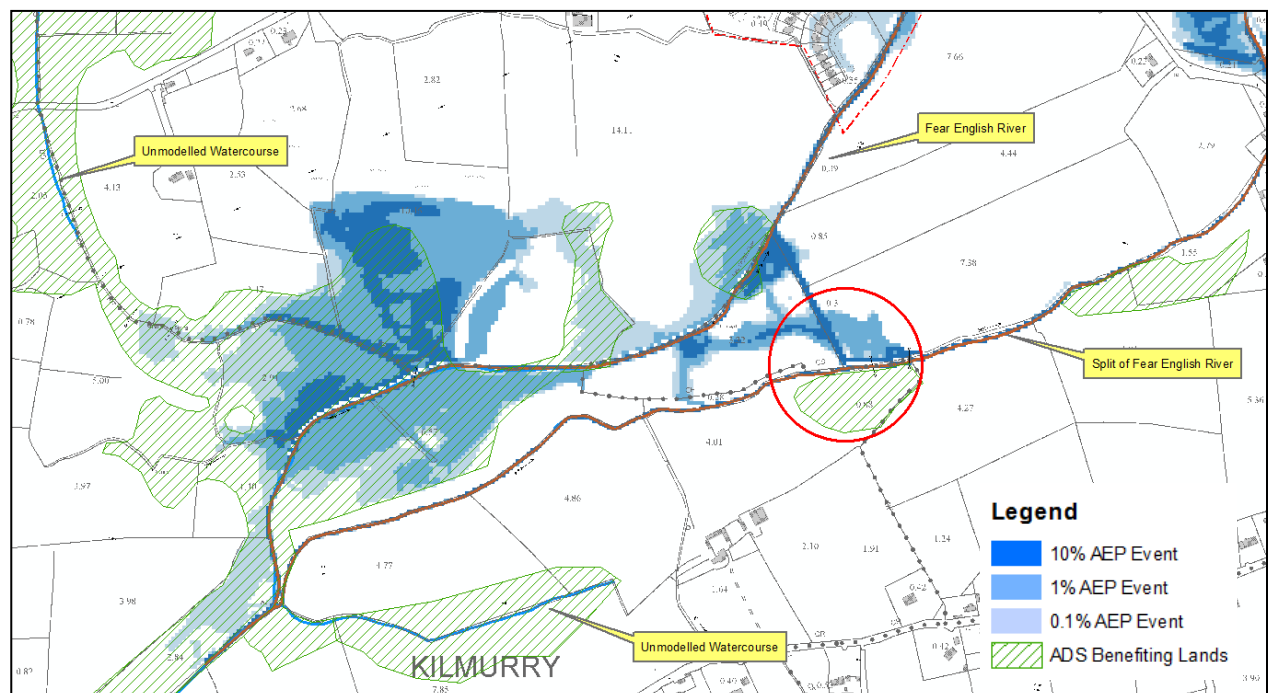


Figure 4.5.20: Comparison between ADS Benefitting Lands and Modelled Flood Extents (at the upper reach of the Fear English River)

The flood extents in the upstream section of the Fear English River and Split of Fear English River show a good correlation with the ADS benefiting lands. Ignoring the areas relating the unmodelled watercourses only one area stands out (circled in red) where land benefiting lies on the right bank and flooding occurs on the left bank. A review of this area found that the left bank is lower than the right bank confirming the direction on flooding, see Figure 4.5.21. It was also noticed the land to the right falls away from the river which may have given rise to ponding from overland flow and has benefited from improved drainage.

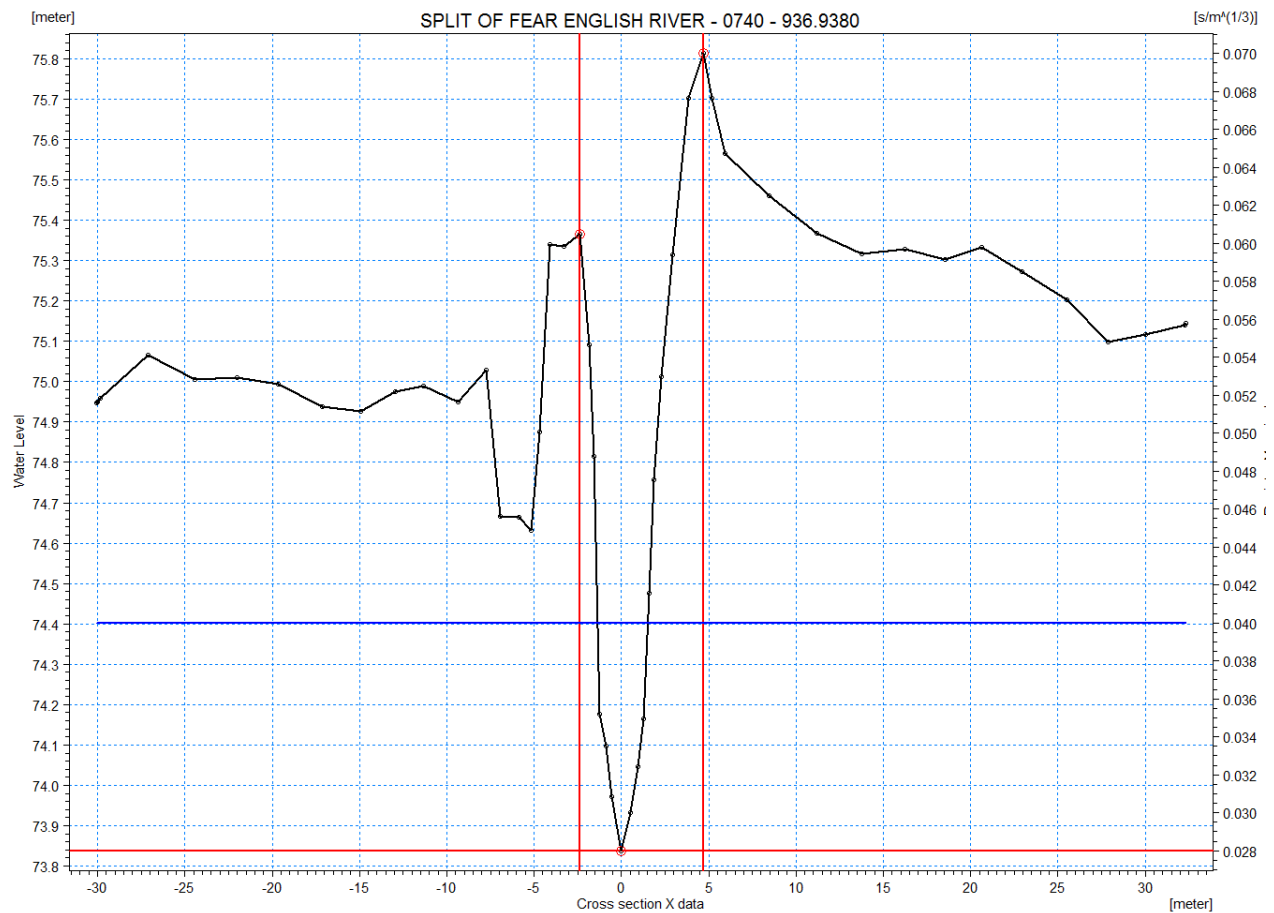


Figure 4.5.21: Cross Section of the Split of Fear English River channel (at chainage 936.94)

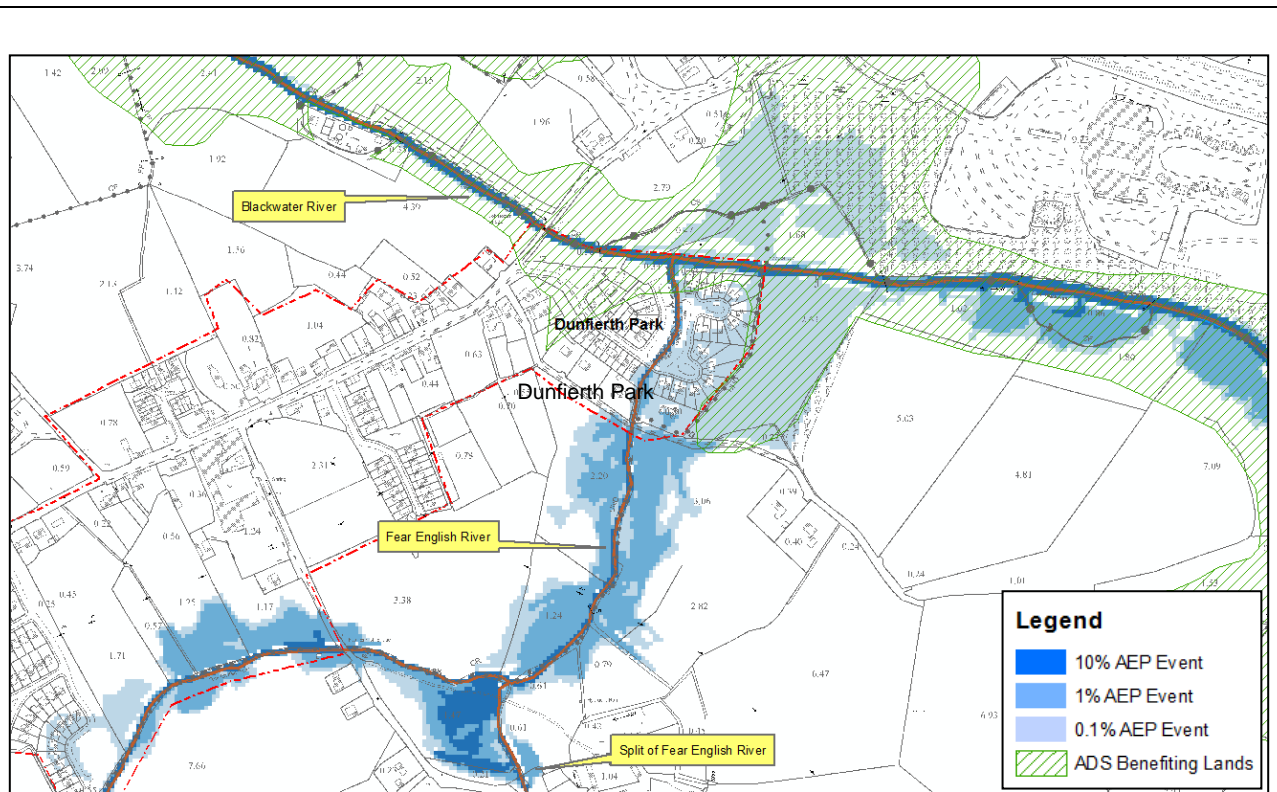


Figure 4.5.22 Comparison between ADS Benefitting Lands and Modelled Flood Extents (at the upper reach of the River Boyne and downstream reach of the Fear English River)

Figure 4.5.22 shows that the downstream reach of the Fear English shows significant flooding however this area does not show any ADS benefiting land. The flood extents of the upstream reach of the Blackwater River shows good correlation with the ADS benefiting lands including the land flooding immediately upstream of Dunferth Park.

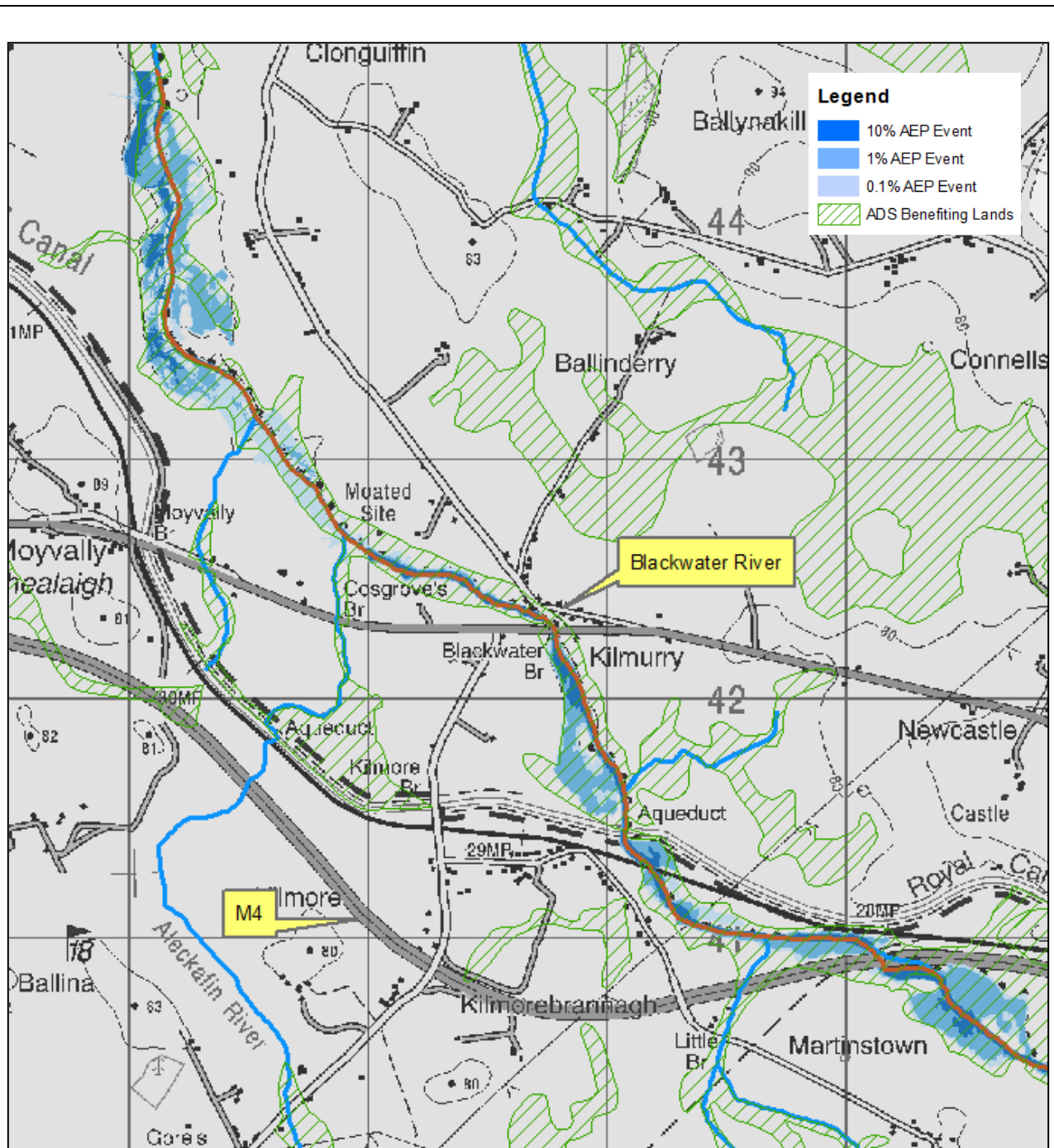


Figure 4.5.23: Comparison between ADS Benefitting Lands and Modelled Flood Extents (along the MPW of the Johnstown Bridge model)

As the flood extents of the Blackwater River transition from the HPW (1D-2D modelling) to the MPW (1D modelling) the correlation with the ADS benefiting lands remains relatively good. Extents differ where the model does not take account of certain unmodelled watercourses in addition it should be noted that the delineation of ADS benefiting land includes land improved by drainage and not solely flood prone land.

4.5.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

(a) A review of the roughness coefficients was carried out and extended into the out of bank areas along the 1D section of the River Blackwater. The roughness values were updated according to the calibration of the model at Castlerickard gauging station. This in general increased the roughness values from 0.03 to 0.04.

(b) The Split of Fear English River is an old mill race which includes a steep culverted section. The culvert is represented as a series of closed cross sections as recommended for long culverts. The culvert has a diameter of 0.9m but restricts to a very small outlet. This abrupt change in geometry caused instabilities in the model. To account for this the outlet was changed to a 0.9m diameter culvert to keep geometrical continuity and the roughness increased to represent the small outlet. Various roughness values were trialled, the head water level remained unaffected by the change to the outlet. However beyond an outlet Manning's n value of 0.030 the model reverted to being unstable. This roughness value was therefore used as the most representative of the steep culvert.

(c) The last bridge on the Fear English River at chainage 2604m was not included in the model as the soffit is curved above the top of bank level and the surrounding floodplain and doesn't represent a restriction to flow by the way of in-channel piers or structures. This was carried out to improve stability of the model.

(d) At chainage 1036m on the River Blackwater there is a series of structures in close succession. A footbridge is located immediately upstream from an arch bridge which has a pipe cutting across the downstream outlet. The structures are too close to be represented individually so a composite structure was created. This was created by taking the most restrictive geometry of the two bridges and increasing the length to span both bridges. This was represented by two structures in parallel one below the pipe and one above. This will force the flow either above or below the pipe.

(e) At chainage 1304m on the River Blackwater there is a pipe spanning the underside of the bridge approx half way up its abutment walls. This was represented by two structures in parallel, one below the pipe and one above. This will force the flow either above or below the pipe.

(f) The last two bridges on the River Blackwater (chainage 14067m & 15537m) have multiple additional arches on the floodplain. During extreme flood events out of bank flooding occurs and water would flow through these arches. As these bridges lie within the 1D section of the model the main channel and floodplain are represented by a single cross section and the bridge represented by a number of culverts in parallel. A sensitivity test was carried out to ensure the model was representing the arches. The water level was reduced when the additional structures were added showing that the additional structures were allowing water to flow through them.

(g) A mass balance plot was carried out to assess the difference between the flow into the model and the volume of water stored along with the discharge out of the model. Results showed a difference of 2.47%.

While there is a discrepancy between the flow in, the volume stored and the flow out; it is within the acceptable limits as stated in the Environment Agency's Fluvial Design Guide. However as the discrepancy is greater than 2% an assessment of the consequence of this error was carried out. It was found that an instability along the MPW could give rise to the calculated discrepancy. This however does not impact the peak flow or water level and therefore is of low consequence to the overall results of the model. The difference of 2.47% is therefore deemed acceptable.

(h) The cross sections on the MPW (Ch6,899m – Ch15,718m) were extended beyond the limit of the topographical survey using the NDHM. A review was carried out post cross section extension to compare the topographical survey level with the NDHM. It was found that for many cross sections a discontinuity was occurring between the two survey datasets. Each cross section was reviewed individually and adjusted so the NDHM tied in with the topography survey. The assumption being that the topographical survey is the more accurate. Where the NDHM data could be offset horizontally to tie in with the topographical survey this was done in the first instance. Secondly where the NDHM seemed to be vertically offset the entire cross section extension to the left or right of the original cross sections was adjusted accordingly to tie it in. Lastly where the discrepancy is minor or the previous two methods are deemed inappropriate the NDHM data was smoothed to tie in with the topographical data.

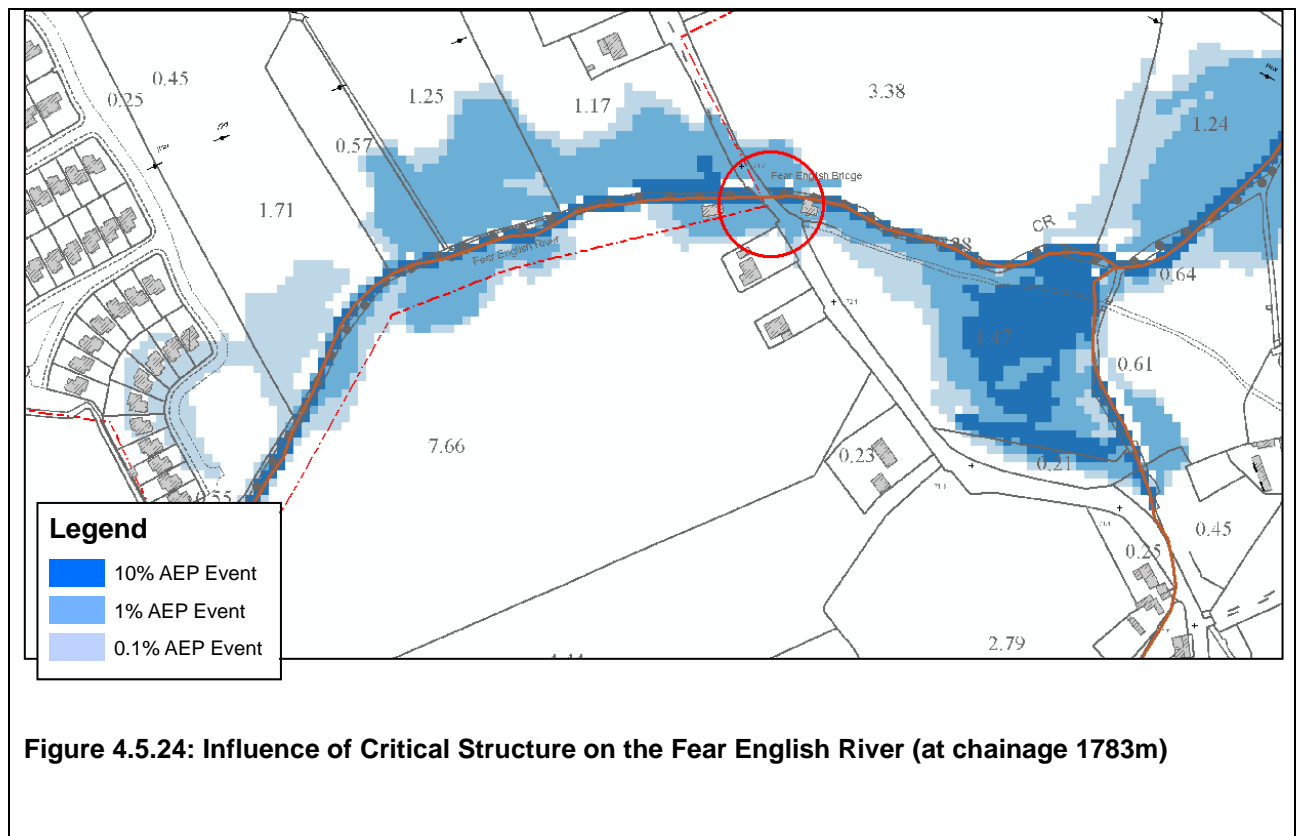
(2) Hydraulic Model Limitations and Parameters:

(a) A 2 second time-step for both the MIKE 11 and MIKE 21 models has been selected in order to achieve a successful model simulation for all return periods.

(b) The lateral links on the Split of Fear English River require an exponential smoothing factor of 0.2.

(3) Design Event Runs & Hydraulic Model Handover Notes:

The critical structure on Fear English River is a stone arch bridge at chainage 1783m (Figure 4.5.24) which restricts the flow and causes a water level difference of approx 0.6m during the 0.1%AEP event.



The steep culvert on the Split of Fear English River (chainage 1597m - 1707m) is a critical structure controlling the water level upstream of it (Figure 4.5.25). The reach of watercourse downstream of this culvert has a large Manning's value and causes high water levels along this reach despite the restricted flow from the culvert.

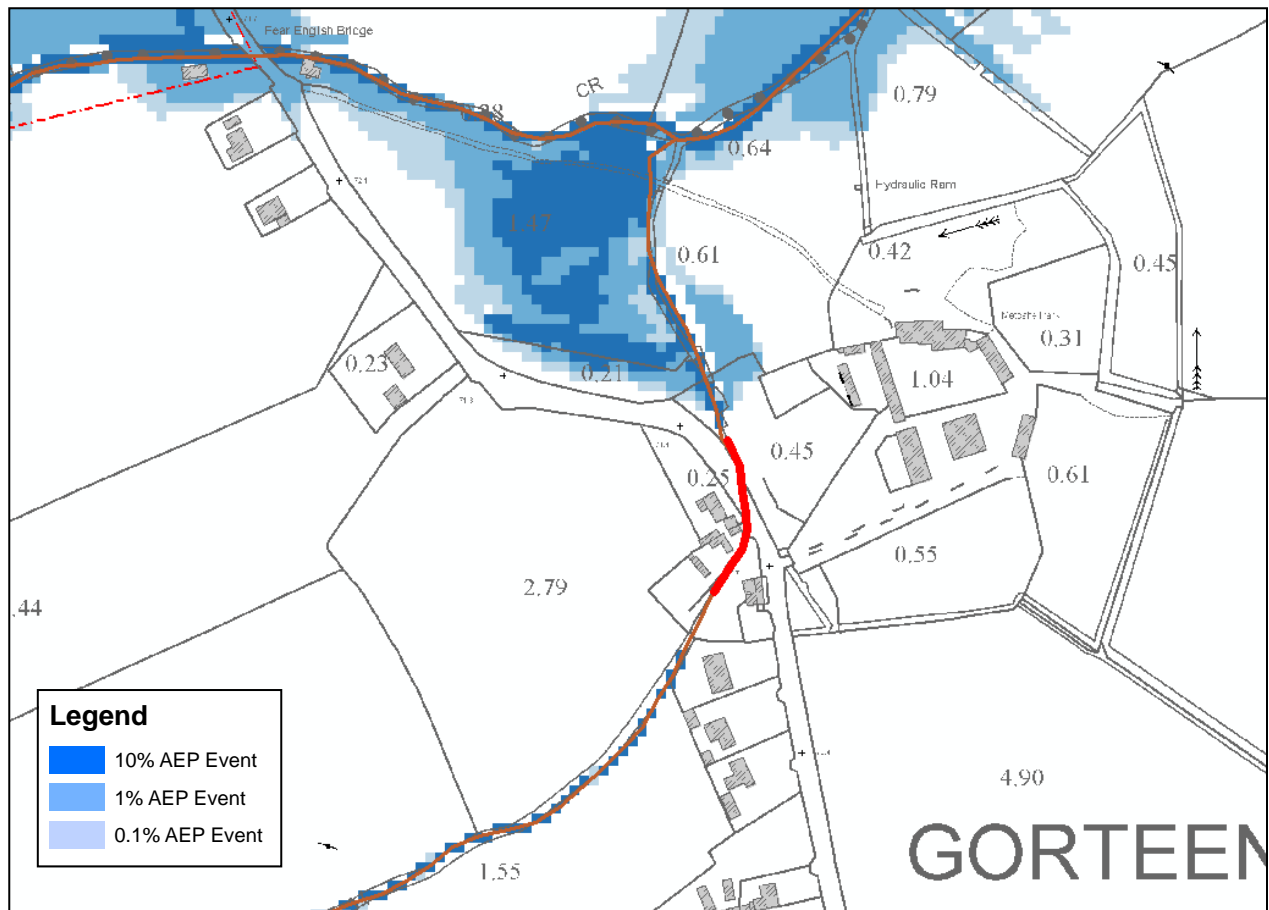


Figure 4.5.25: Influence of Critical Structure on the Split of the Fear English River (at chainage 1597m)

Out of bank flooding along the downstream reach of the Fear English River occurs during the 0.1% AEP event and floods houses in Dunferth Park as it travel overland towards the Blackwater River.

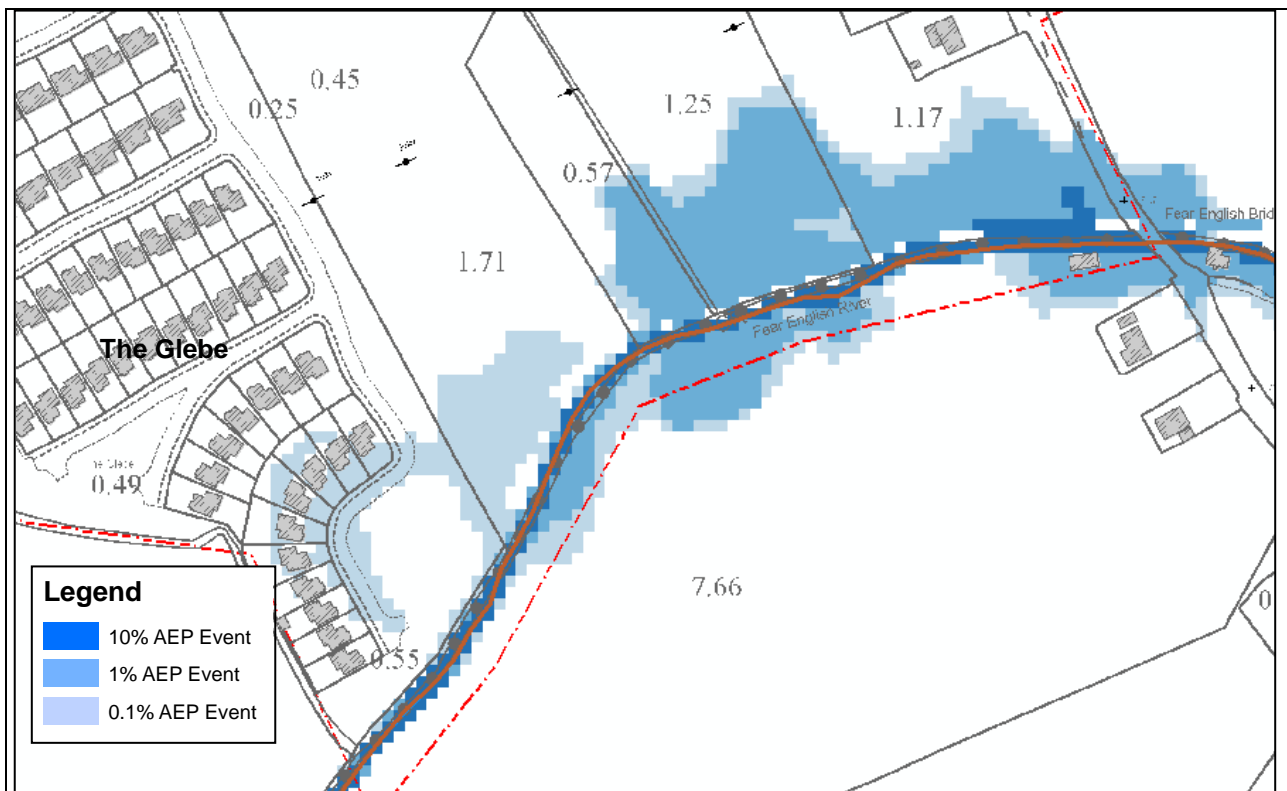


Figure 4.5.27: Houses in The Glebe at risk during a 0.1% AEP Event

While flooding occurs during the 1%AEP event Dunfieth Park and The Glebe are not affected. The Fear English River has adequate capacity at these locations. However one house upstream of Dunfieth Park is affected during the 1%AEP event as highlighted in Figure 4.5.28.

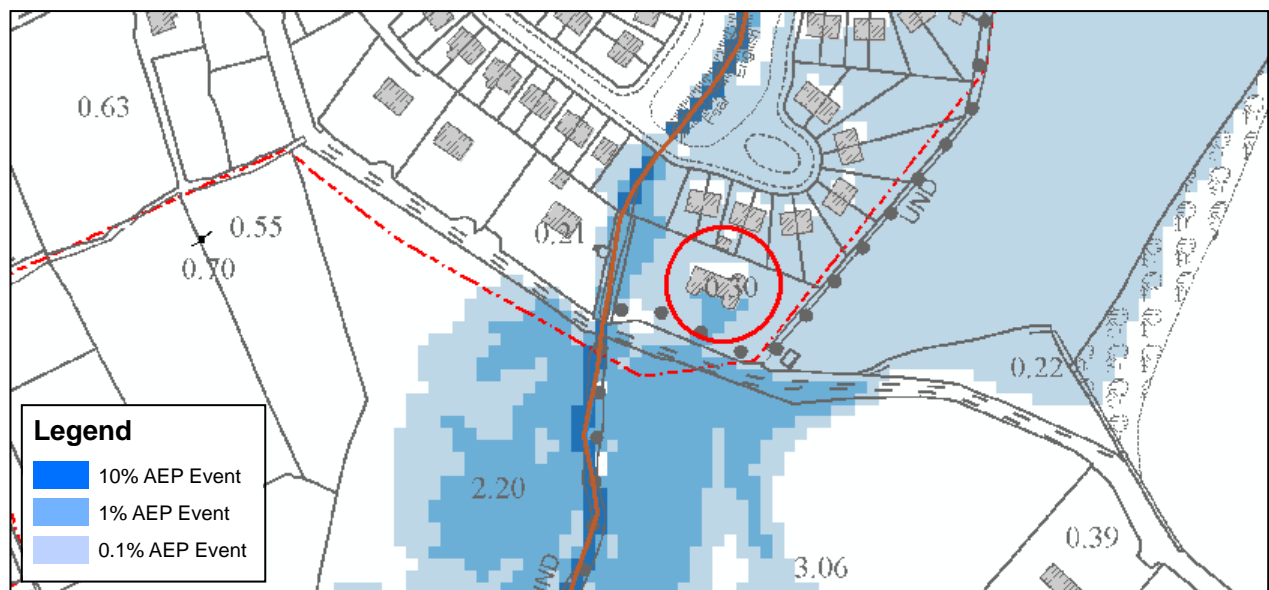


Figure 4.5.28: House at Risk during a 1% AEP Event

For further information on the extent, depths, frequency of flooding and the receptors affected please refer the Johnstown Bridge AFA flood hazard and flood risk maps.

(4) Hydraulic Model Deliverables:

Please see Appendix E for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Mark Wilson
Model Reviewed by:	Stephen Patterson
Model Approved by:	Andrew Jackson

4 HYDRAULIC MODEL DETAILS

4.6 LONGWOOD MODEL

Name	Local Authority	Unique ID	SSA	Status	Date
Longwood	Meath	70888	AFA	Final	08/05/2017

4.6.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600Rp0001_Flood Risk Review) highlighted Longwood as an Area for Further Assessment for fluvial flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA.

The Longwood model consists of the River Blackwater and a small watercourse called the Longwood Stream. The River Blackwater model starts just upstream of the AFA boundary and continues to the confluence with the River Boyne 7km downstream. The Longwood Stream model starts 1km upstream of the AFA flowing through the AFA before discharging to the River Blackwater.

The full reaches of both modelled watercourses lie within the Arterial Drainage Scheme (ADS) and while these watercourses may be subject to dredging this model represents the current status as of June 2012 when the cross section survey was carried out. There is one gauging station along the River Blackwater, the Castlerickard gauging station, which is over 5km downstream of the AFA and has flow and level data. The gauging station, Castlerickard (07003 – OPW) was found during the rating review to have some uncertainty in the rating at Q_{med} despite having an FSU classification of B for the period post arterial drainage scheme (1975 onwards).

The River Blackwater flows adjacent to the AFA boundary and it is along this length, including 0.5km of reach upstream and downstream, that the River Blackwater has been designated as a HPW and therefore modelled as 1D-2D using the MIKE suite of software. The remainder of the River Blackwater has been designated as MPW and has been modelled as 1D using the same suite of software. The Longwood stream's full length has been designated as HPW and has been modelled as 2D again using the MIKE suite of software.

Initially, as detailed in the hydrology report (IBE0600Rp0012_HA07_Hydrology Report), the Longwood model was to consist of the Longwood Stream in isolation, the Blackwater River being modelled as part the neighbouring Johnstown Bridge model. When assessed in more detail the decision to include the Blackwater River was taken as potential flooding from the Blackwater River to the AFA, either directly or via the Longwood Stream, is likely. Details of the revised model extent and schematisation are shown in

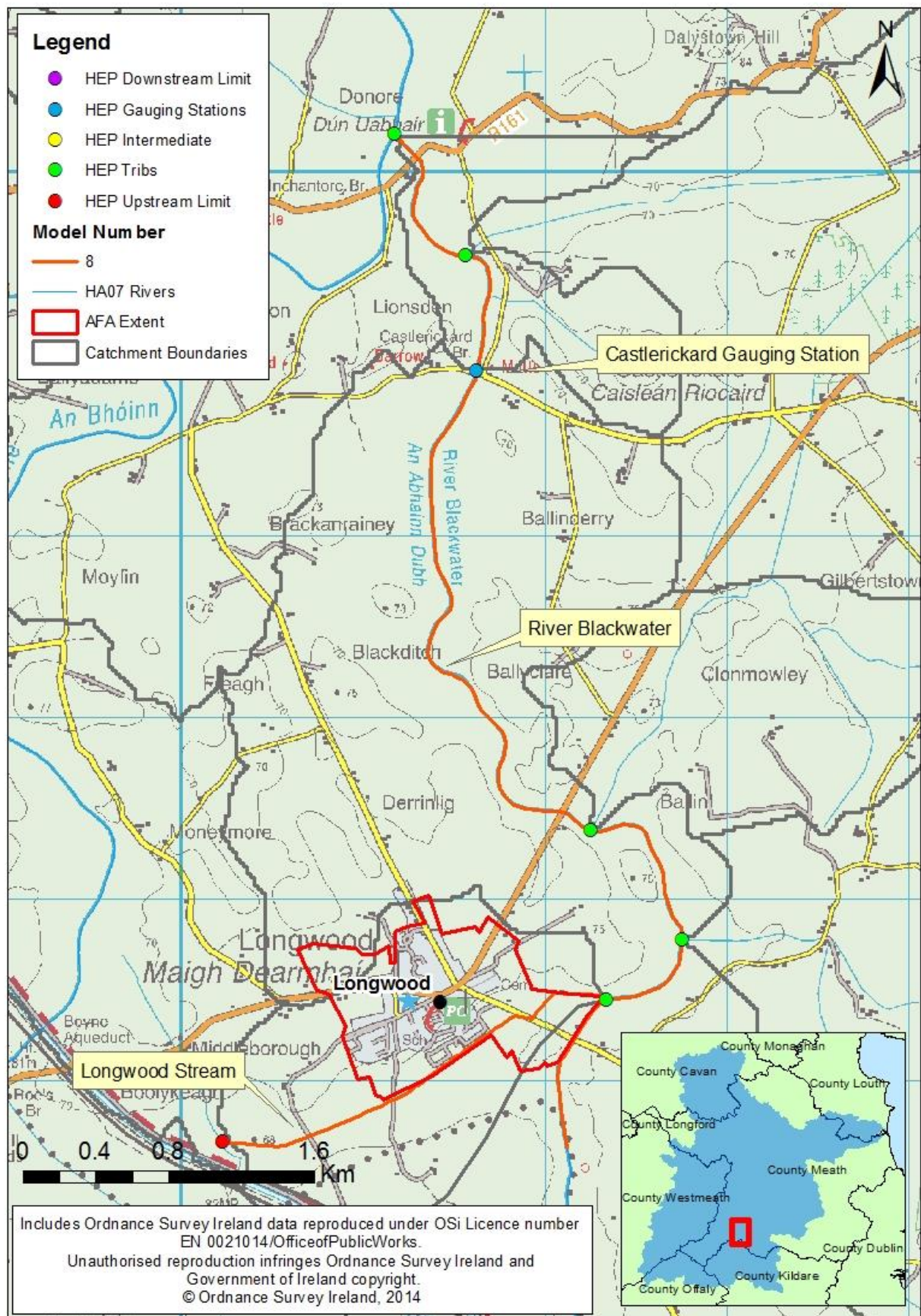


Figure 4.6.1 to Figure 4.6.4, inclusively.

The Johnstown Bridge model is located upstream of the Longwood model and models an overlapping

reach of the Blackwater River from Longwood to the confluence with the Rive Boyne. Whereas the Longwood model has modelled this reach in a combination of 1D-2D modelling and 1D modelling the Johnstown Bridge model has modelled it using 1D modelling only and is therefore considered to be less accurate. The presentation of the Johnstown Bridge flood extents therefore ends just upstream of the Longwood model and the remainder of the Blackwater River is presented in the Longwood model. The Johnstown Bridge model was also used to provide the upstream flow hydrograph in the Blackwater River for the Longwood model. An illustration of the two models is presented in Section 4.6.2 (1).

There are no further models downstream of the Longwood model on the River Blackwater, the next downstream model is the Trim model on the River Boyne, this model was used to provide the downstream boundary for both the Johnstown Bridge and Longwood Models.

(2) Model Reference:		HA07_LONG8
(3) AFAs included in the model:		Longwood
(4) Primary Watercourses / Water Bodies (including local names):		
Reach ID	Name	
0735	RIVER BLACKWATER	
0736	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	
(5) Software Type (and version):		
(a) 1D Domain: MIKE 11 (2011)	(b) 2D Domain: MIKE 21 rectangular mesh (2011)	(c) Other model elements: MIKE FLOOD (2011)

4.6.2 Hydraulic Model Schematisation

(1) Map of Model Extents:

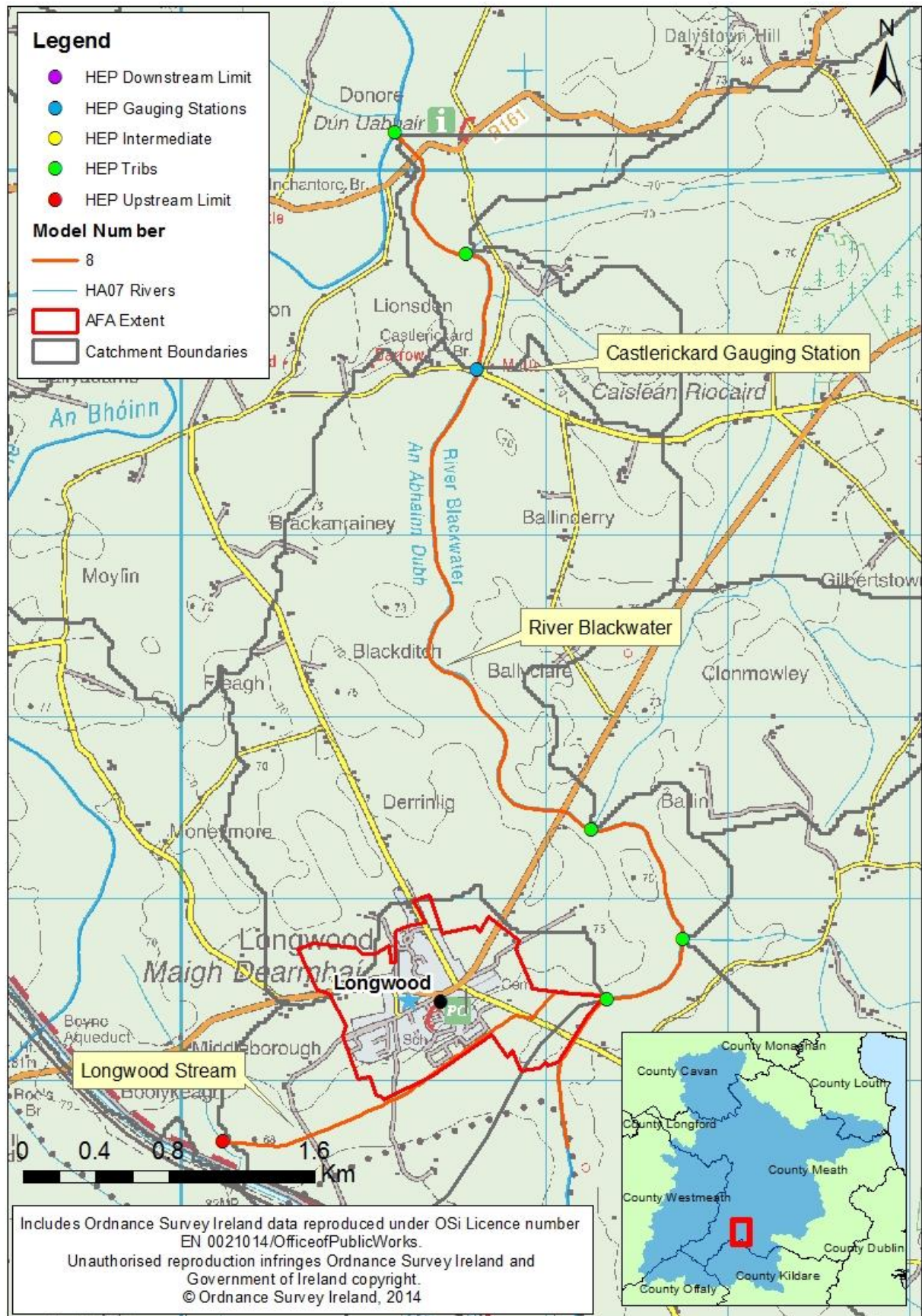


Figure 4.6.1 and Figure 4.6.2 illustrate the extent of the modelled catchment, river centre line, HEP locations and AFA extents as applicable. The Blackwater catchment contains 1 Upstream Limit HEP, 1

Downstream Limit HEP, 4 Tributary HEPs and 1 Gauging Station HEP.

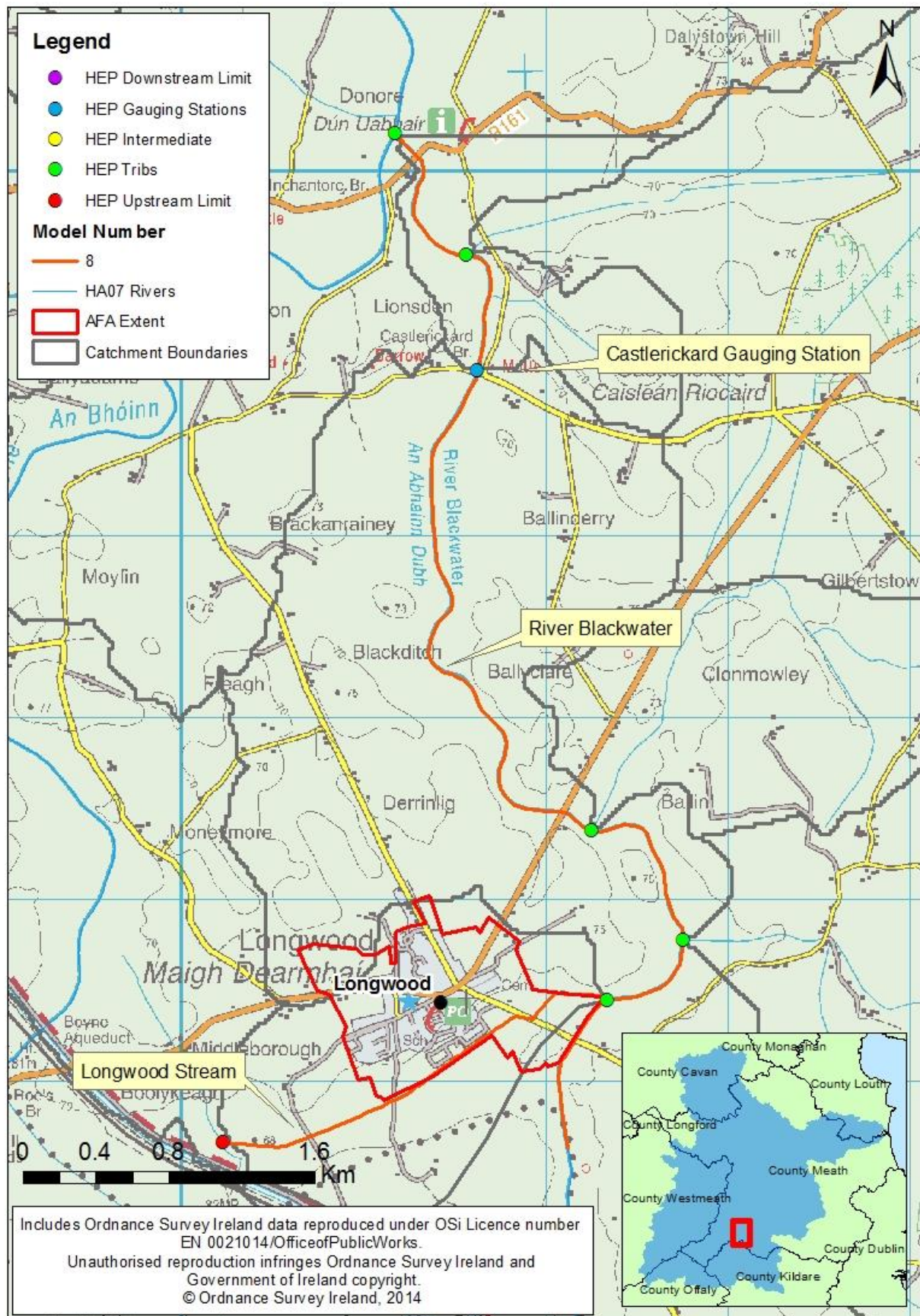


Figure 4.6.1: Longwood Model Overview



Figure 4.6.2: Longwood Model AFA

Figure 4.6.3 and Figure 4.6.4 show the location of the cross sections used in the model along with the location of the 2D links and the critical structures. A review was carried out to ensure agreement between the cross sections and 2D domain.

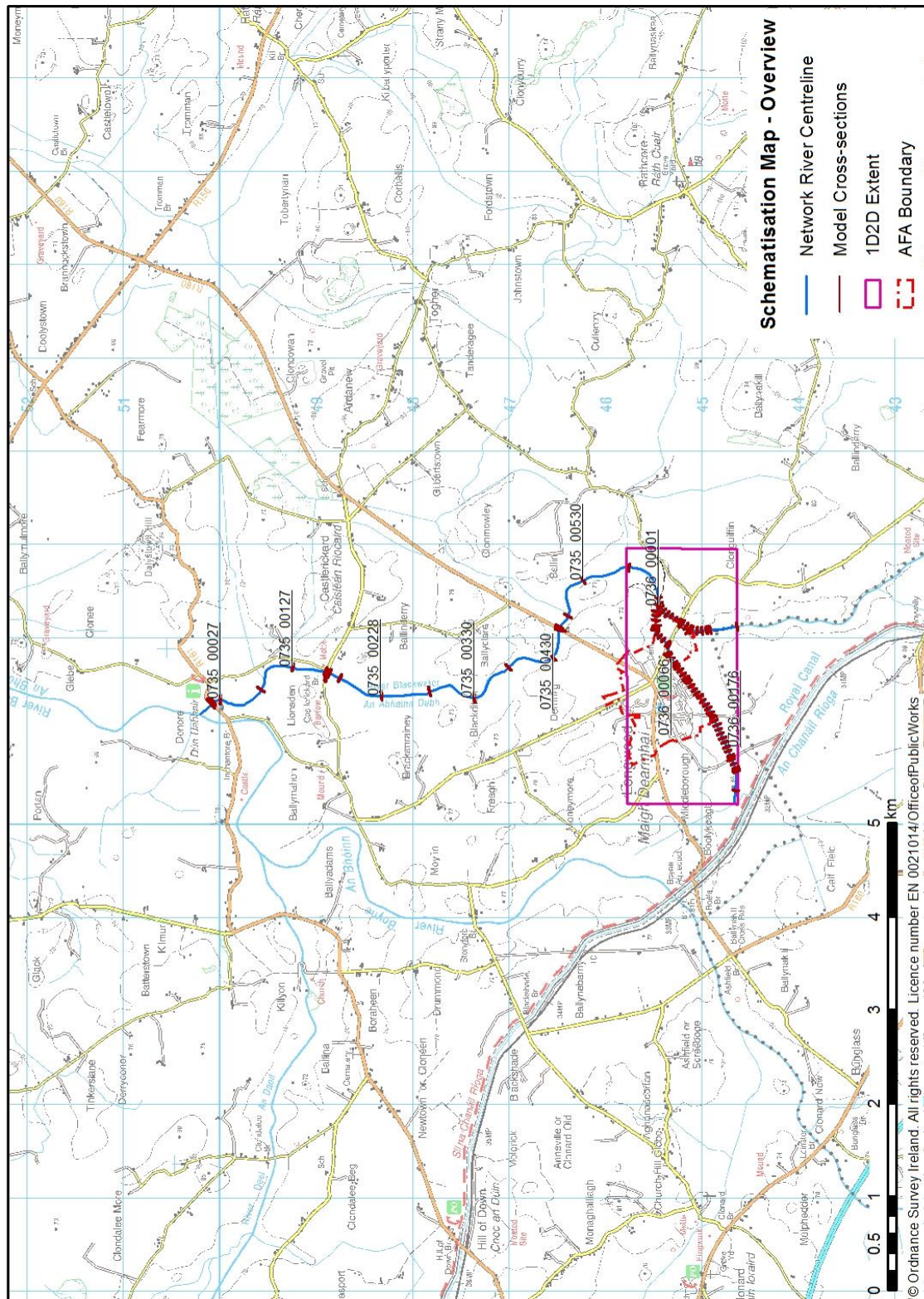


Figure 4.6.3: Overview of Model Schematisation

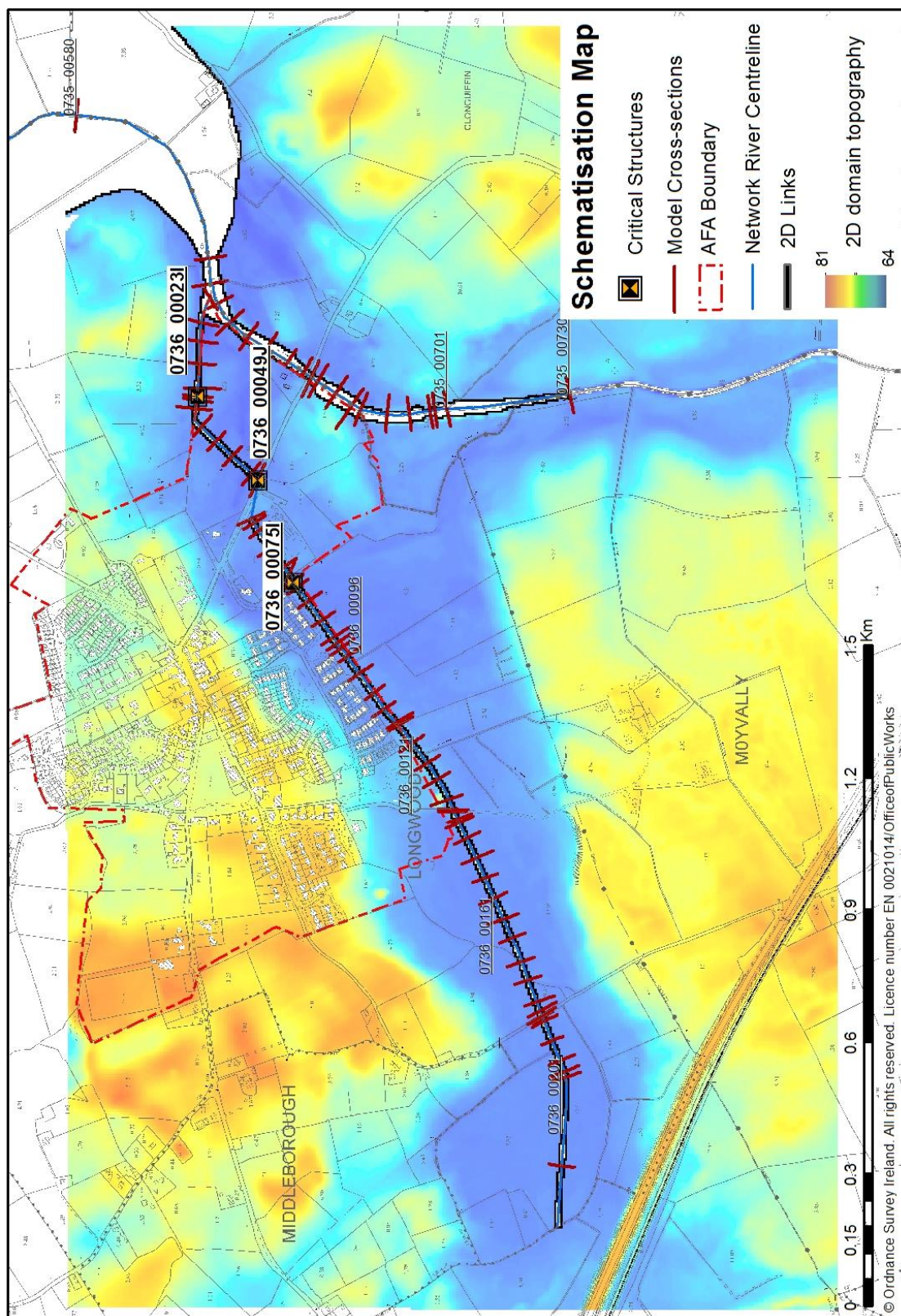
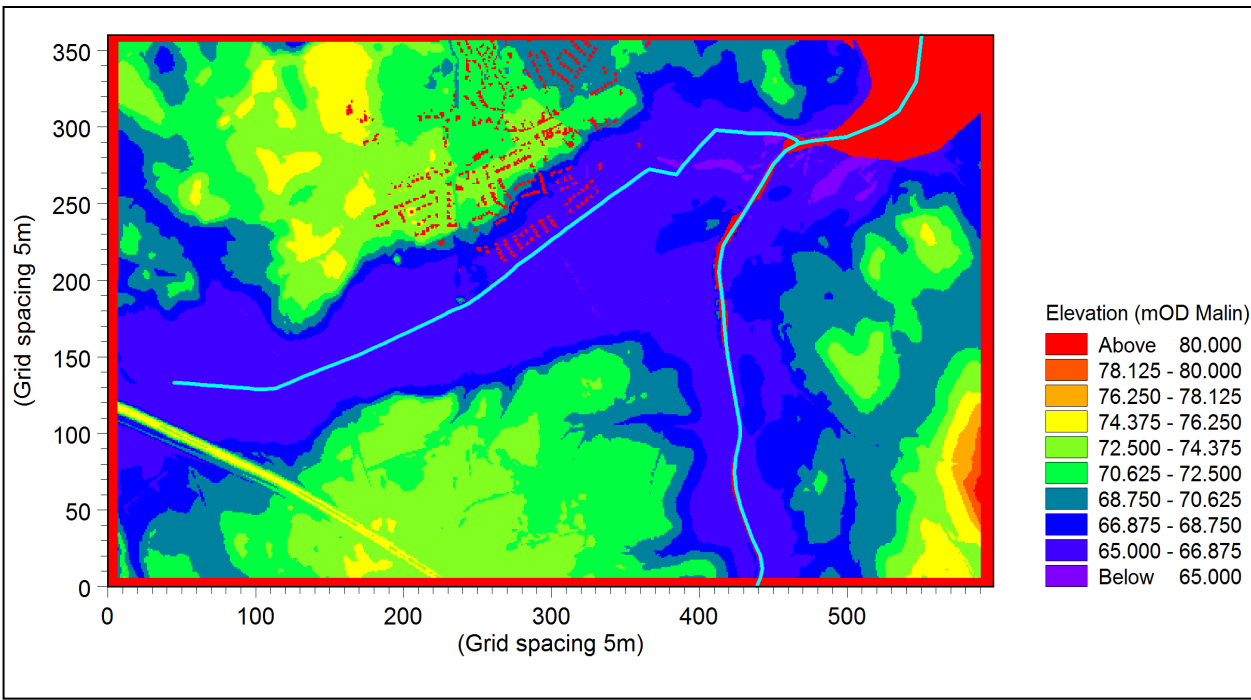


Figure 4.6.4: Longwood Model Schematisation AFA Overview

(2) x-y Coordinates of River (Upstream extent):															
<table><tr><th>River Name</th><th></th><th>x</th><th>y</th></tr><tr><td>0735</td><td>RIVER BLACKWATER</td><td>272081</td><td>244916</td></tr><tr><td>0736</td><td>LONGWOOD STREAM</td><td>270224</td><td>244665</td></tr></table>		River Name		x	y	0735	RIVER BLACKWATER	272081	244916	0736	LONGWOOD STREAM	270224	244665		
River Name		x	y												
0735	RIVER BLACKWATER	272081	244916												
0736	LONGWOOD STREAM	270224	244665												
(3) Total Modelled Watercourse Length:		9.39 (km)													
(4) 1D Domain only Watercourse Length:	5.8(km)	(5) 1D-2D Domain Watercourse Length:	3.59(km)												
(6) 2D Domain Mesh Type / Resolution / Area:		Rectangular / 5 metres / 5.41 km ²													
(7) 2D Domain Model Extent:															
<div></div>															
<p>Figure 4.6.5: Longwood Model 2D Model Domain</p> <p>This map illustrates the 2D domain modelled extents of Longwood and the general topography of the catchment.</p>															
(8) Survey Information															
(a) Survey Folder Structure:															
First Level Folder	Second Level Folder	Third Level Folder													
Murphy_E07_M05_WP2_0735_120625	V0_0735_GIS and Floodplain Photos	Flood_Plane_Photos_and_Shap efiles													
Longwood															
Murphy: Surveyor Name		Structure_Register													

E07: Eastern CFRAM Study Area, Hydrometric Area 7 M08: Model Number 8 0735: River Reference WP2 : Work Package 2 Version: Most up to date 120625– Date Issued (25 th JUN 2012)		Surveyed_Cross_Section_Lines
	V0_0735_Photos_Videos	0735_00016_DN
	V1_0735_Ascii	
	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	
(b) Survey Folder References:		
Reach ID	Name	File Reference
0735	RIVER BLACKWATER	Murphy_E07_M05_WP2_0735_120625
0736	LONGWOOD STREAM	Murphy_E07_M05_WP2_0736_120625
(9) Survey Issues:		
No survey queries were raised		

4.6.3 Hydraulic Model Construction

All structures within the 1D model that have potential to overtop, such as bridges and culverts, were given an overtopping weir representative of the associated parapet or deck. This allows for flood water to overtop a surcharged structure and avoids creating an artificially high backwater profile. Overtopping weirs were applied to all bridges and culverts in the Longwood model.	
(1) 1D Structures (in-channel along modelled watercourses):	See Appendix F Number of Bridges and Culverts: 23 Number of Weirs: 0



Figure 4.6.6: Photograph of Critical Structure on the Longwood Stream

A critical structure was identified on the Longwood Stream at chainage 1636m. While other access bridges are located upstream of this location, this culvert creates the greatest head loss and controls the water levels upstream and downstream. This remains the case for the duration of the 10% and 1% AEP flood events however during the 0.1% AEP flood event flood water overtops and bypasses the bridge causing the culvert downstream at chainage 1852m to become the dominant control structure.



Figure 4.6.7: Photograph of Critical Structure (inlet left, outlet right) on the Longwood Stream

A critical structure was identified on the Longwood Stream at chainage 1852m. The inlet is larger than the outlet with the critical geometry at the outlet of culvert. During the 10% and 1% AEP flood events the access bridge upstream at chainage 1636m restricts the flow thereby keeping the headloss and therefore water level difference across this culvert relatively small. During the 0.1% AEP flood event the upstream access bridge is bypassed allowing the flow to through this culvert to achieve a significant head loss that it dictates the water-levels upstream and downstream of it.

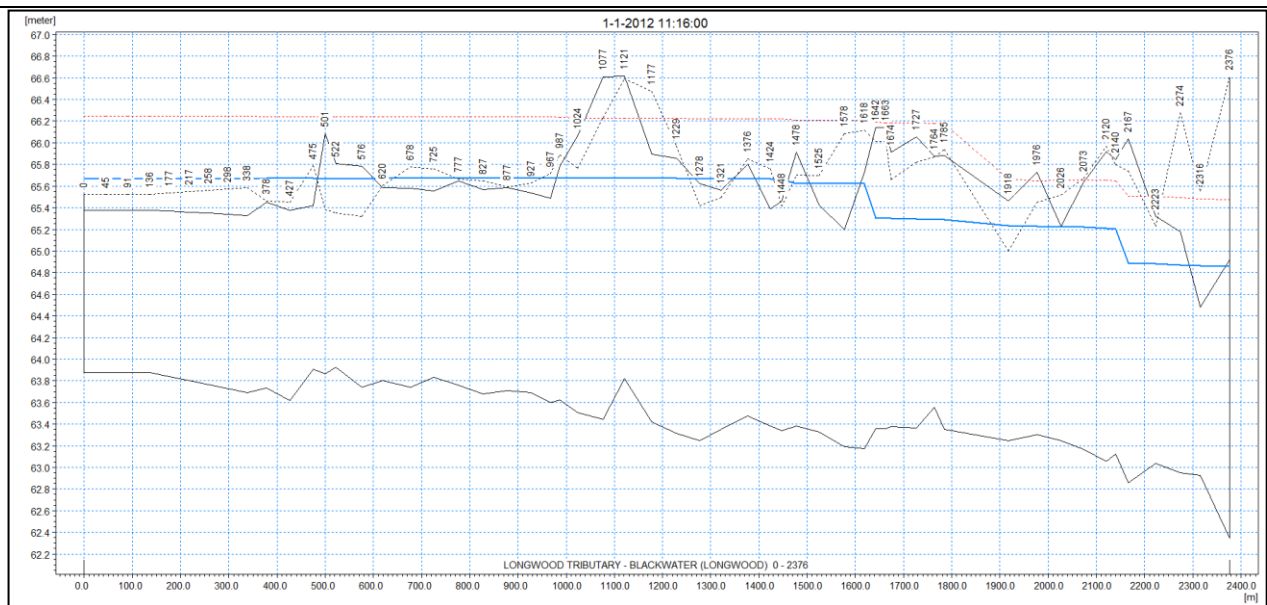


Figure 4.6.8 shows how the water levels are initially controlled by the access bridge at chainage 1636m and then by the culvert at chainage 1852m.

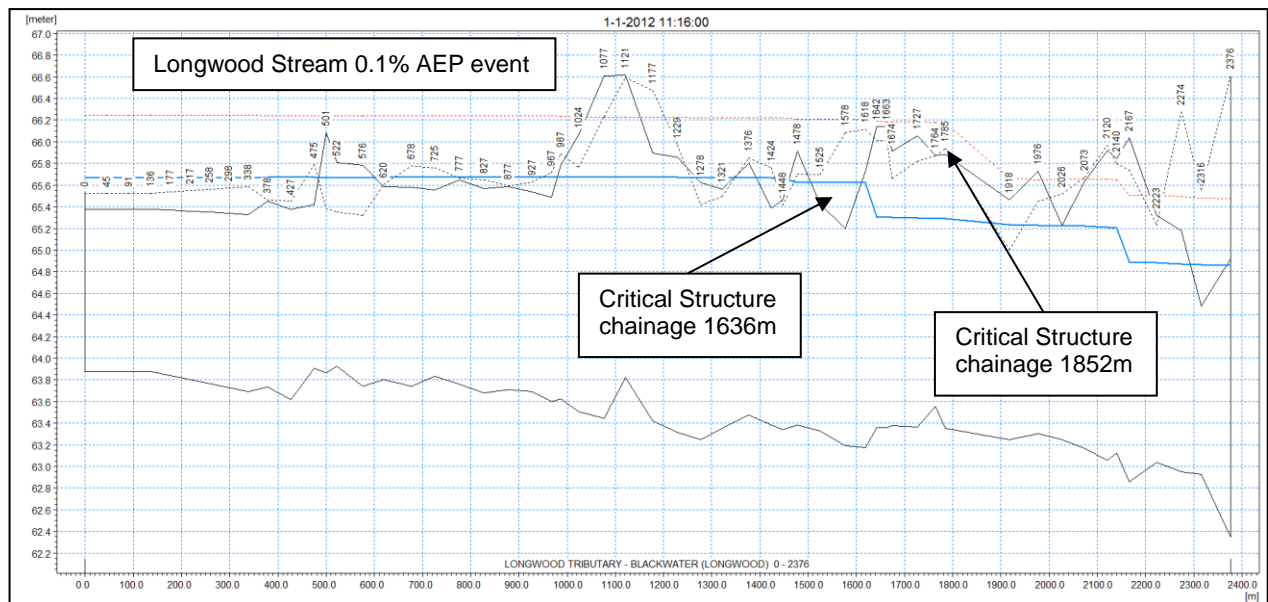


Figure 4.6.8: Water profile of the Longwood stream during a 0.1% AEP flood event highlighting the influence of critical structures

(2) 1D Structures in the 2D domain (beyond the modelled watercourses):	None
(3) 2D Model structures:	None

(4) Defences:

Type	Watercourse	Bank	Model Start Chainage (approx.)	Model End Chainage (approx.)
------	-------------	------	-----------------------------------	---------------------------------

No formal defences

(5) Model Boundaries - Inflows:

Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report - Section 4.8 and Appendix D). The boundary conditions implemented in the model are shown below.

Figure 4.6.9 Location and type of boundary conditions applied to the Longwood model

	Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
1	Distributed Source	Inflow	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	0	2375		Top up between 07_40000_U & 07_40000_1
2	Open	Inflow	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	0	0		07_40000_U
3	Open	Inflow	BLACKWATER	8705.4	0		0735_00701
4	Open	Q-h	BLACKWATER	15718	0		
5	Point Source	Inflow	BLACKWATER	9933.6	0		07_948_3
6	Point Source	Inflow	BLACKWATER	10887.79	0		07_1688_6
7	Point Source	Inflow	BLACKWATER	14811.98	0		07_1363_6
8	Distributed Source	Inflow	BLACKWATER	8705.4	14064		Top-up between 07_980_4_RPS & 07003_RPS
9	Distributed Source	Inflow	BLACKWATER	14064	15552		Top-up between 07003_RPS & 07_954_3

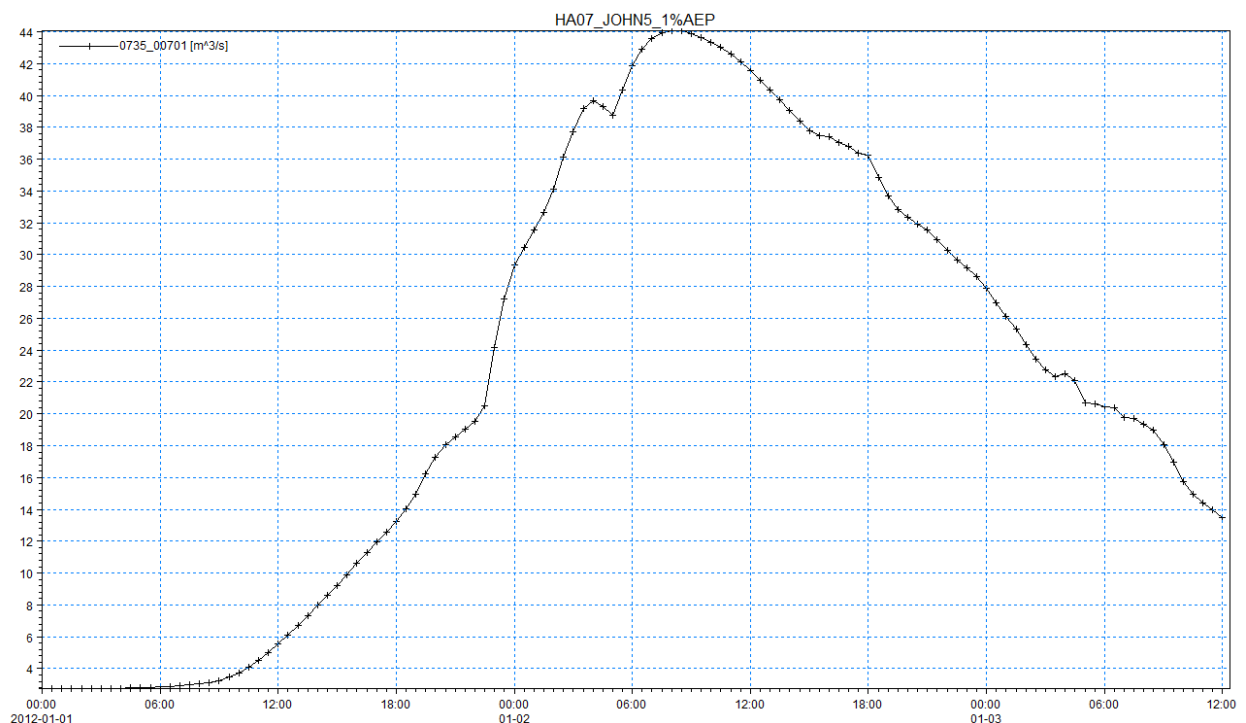


Figure 4.6.10: Inflow Hydrograph for the River Blackwater during a 1% AEP Event

Upstream of Longwood on the River Blackwater is Johnstown Bridge which has also been modelled as part of the CFRAM study. The Johnstown Bridge model includes the full reach of the River Blackwater to the confluence with the River Boyne and as a consequence there is an overlap in the two models. This allowed both models to be calibrated to the gauging station found in the lower reaches of the River Blackwater at Castlerickard.

A flow output from the Johnstown Bridge model was used at the same chainage as the upstream boundary in the Longwood model (chainage 8419m). The hydrograph above shows the upstream boundary of the River Blackwater.

As the modelled Longwood Stream starts near the head of its catchment a distributed flow was assigned along its length. However in order to obtain stable initial conditions a nominal steady flow of $0.1\text{m}^3/\text{s}$ was provided at the upstream boundary.

The hydrology was reviewed during the rating review at Castlerickard Gauging Station and was found to produce consistent results in the full Longwood model. There were therefore no other changes made to the hydrology. Please view Section 4.6.5(2) which discusses model updates for Final deliverables.

(6) Model Boundaries – Downstream Conditions:

The water level generated from the Trim model at the River Boyne/River Blackwater confluence were taken as the downstream boundary condition for the River Blackwater for each flood event, i.e. the 0.1% AEP event water levels from the Trim model were used to define the 0.1% AEP flood levels in the Longwood Model. Figure 4.6.11 shows the downstream boundary water level based on the River Boyne 0.1% AEP flood event. The downstream extent of the Longwood model is located at the chainage 15718m of the River Blackwater and is located 3.6km from the Longwood Stream confluence. Therefore, the influence of the River Boyne on water levels at Longwood is considered to be minimal.

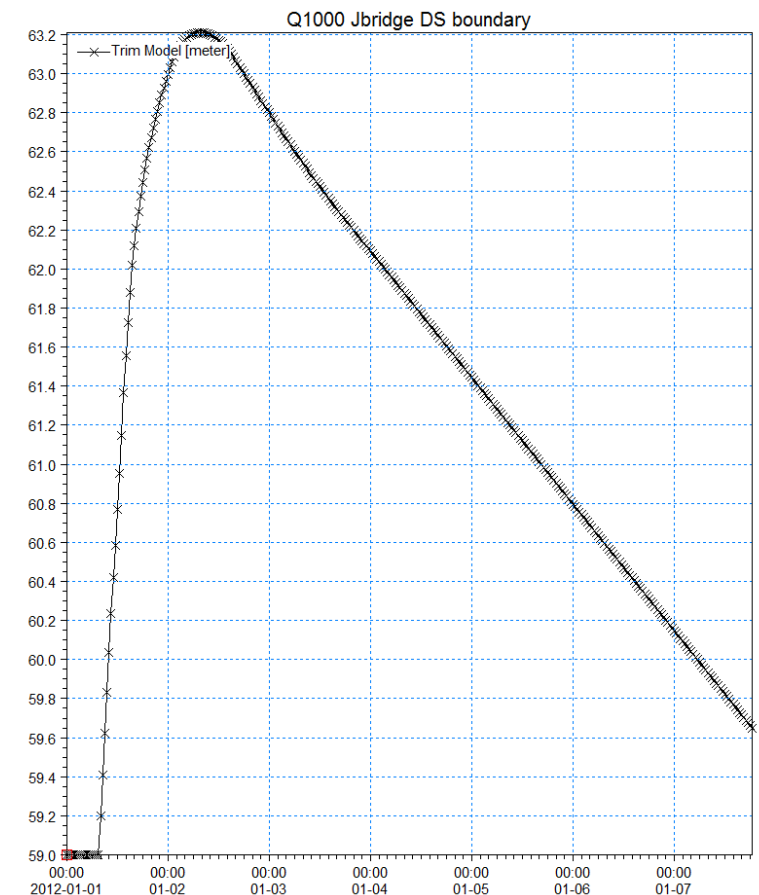
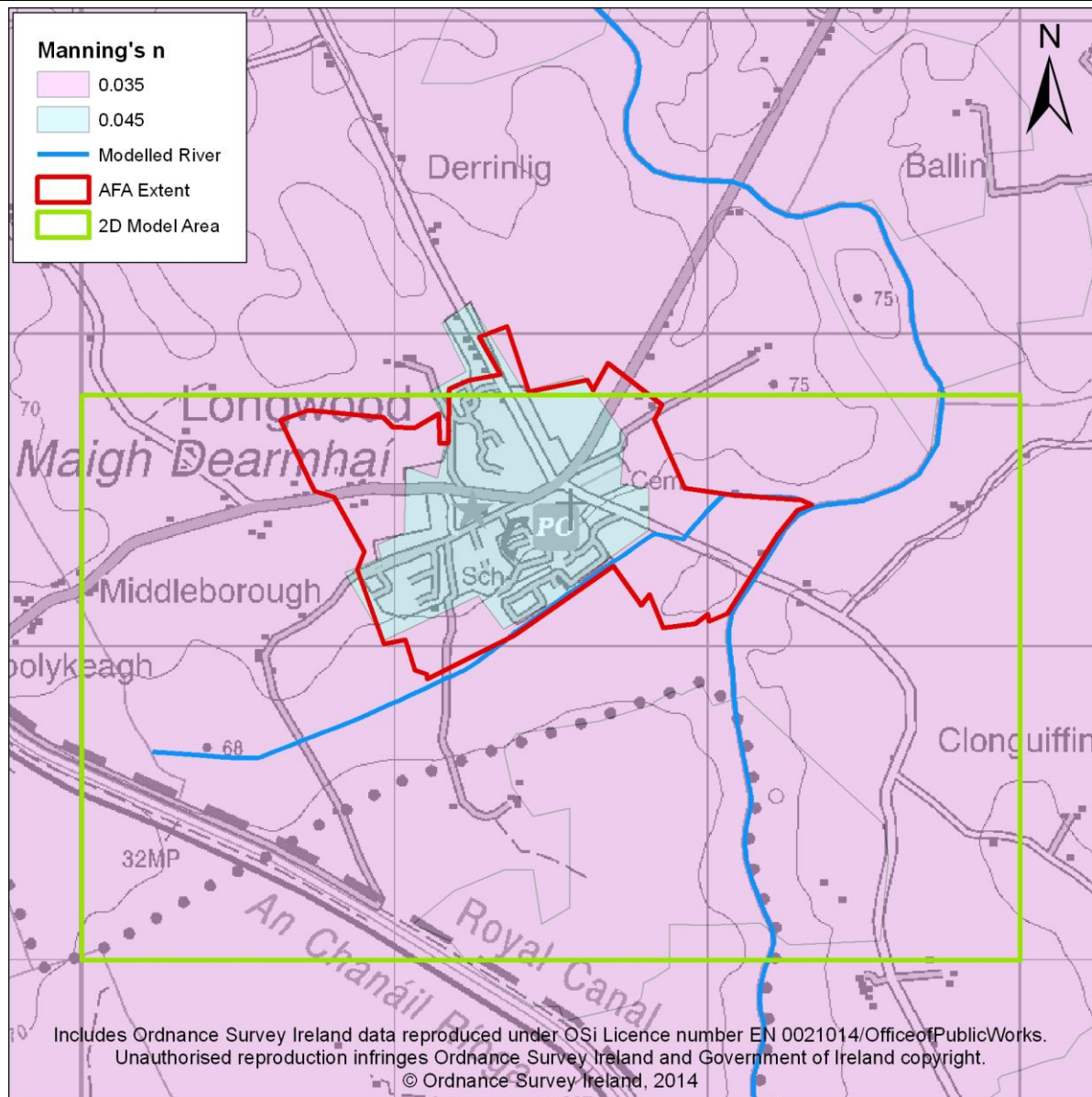


Figure 4.6.11: Downstream Boundary Water Level (based on the River Boyne 0.1% AEP flood event)

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.040	Maximum 'n' value: 0.070
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.040	Maximum 'n' value: 0.070
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.035 (Inverse of Manning's 'M')	Maximum 'n' value: 0.045 (Inverse of Manning's 'M')

**Figure 4.6.12: Floodplain Roughness Values in 2D domain**

This map illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset. The surrounding rural area to Longwood is assigned a Manning's n of 0.035. Longwood is assigned a Manning's n value of 0.045.

(d) Examples of In-Bank Roughness Coefficients

Figure 4.6.13: Example of roughness coefficients on the Blackwater River at cross section 0735_00016

Manning's n value: 0.040

Clean, winding with riffles



Figure 4.6.14: Example of roughness coefficients on the Longwood Stream at cross section 0736_00189

Manning's n value: 0.070

Sluggish reach with weeds



Figure 4.6.15: Example of roughness coefficients on the Blackwater River at cross section 0735_00681

Manning's n value: 0.070 (on LHB)

Dense vegetation on left hand bank

4.6.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact 1% AEP hydraulic model within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a high increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.6.16. This outcome indicates that the Longwood model demonstrates a high sensitivity to changes in roughness parameters. Properties affected are located at Brackinrainey Wood, Brackinrainey Manor and The Courtyard areas of Longwood, especially those located within a close proximity to the Longwood Stream, at the southern extent of the AFA. This increase of the flood extents has a high impact upon receptors located within the AFA as 76 new receptors are affected. In comparison, no receptors are affected during the design event.

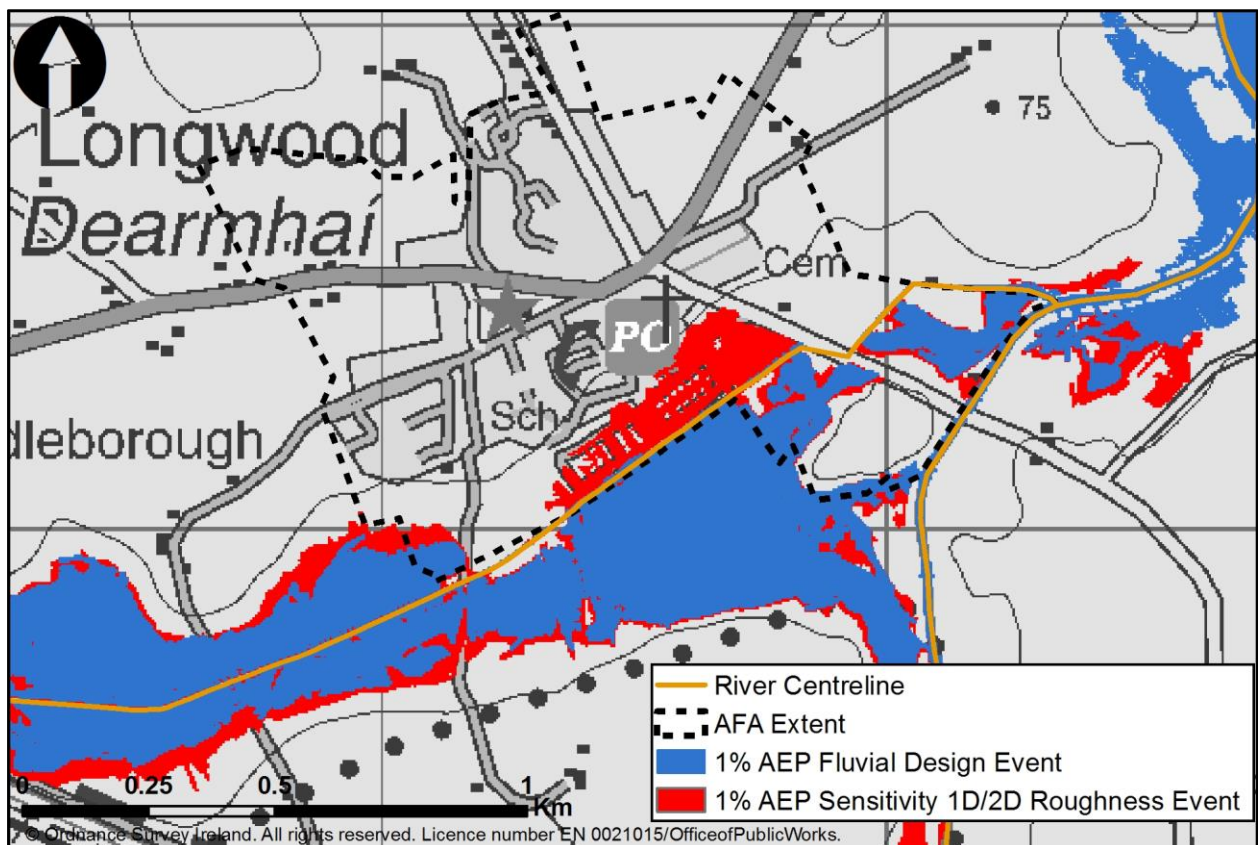


Figure 4.6.16: Comparison between 1% AEP Design Event and 1% AEP Sensitivity 1D/2D Roughness Event

- b) Downstream boundary increase – it has been determined for Longwood that the downstream water level boundary has no impact on the AFA. Therefore, a downstream water level boundary sensitivity run was not required.

- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Longwood model is assessed as having high uncertainty/sensitivity (refer to the Hydrology Report IBE0600Rp0012 for further detail); factors of 1.36 and 1.45 have been applied to design flows for the sensitivity simulation. Figure 4.6.17 shows that the Longwood model has a high sensitivity to increased inflow parameters; this is reflected by the increase of flood extent. Increases in flood extents affect the Brackinrainey Wood and Courtyard areas of Longwood. These changes have a significant impact upon receptors located within the AFA as approximately 22 additional buildings are affected. This is a high impact as no receptors are affected during the design event.

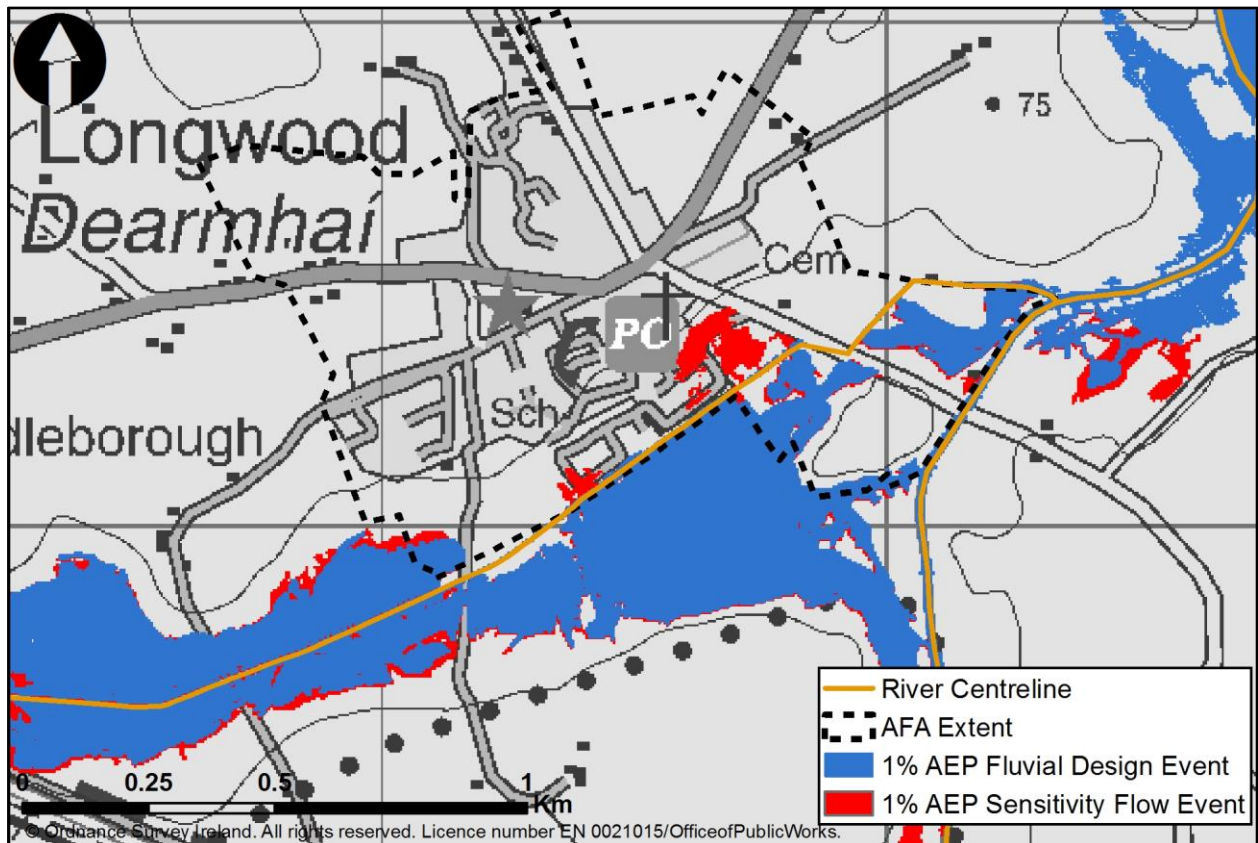


Figure 4.6.17: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event 1

- d) Flow volume – A sensitivity adjustment factor of 2 has been applied to flood duration to assess the response of the model. Flood durations developed from single or from catchment descriptors with uncertainty at flood flows are assessed. Figure 4.6.18 shows that the Longwood model demonstrates a high sensitivity to flow volume parameters adjustment, resulting in a high increase to flood extents. This increase has a high impact upon AFA receptors, as 50 new properties are affected. These properties are located at Brackinrainey Wood and Brackinrainey Manor areas of Longwood, particularly towards the southern extent of the AFA boundary and within a close proximity to the Longwood watercourse.

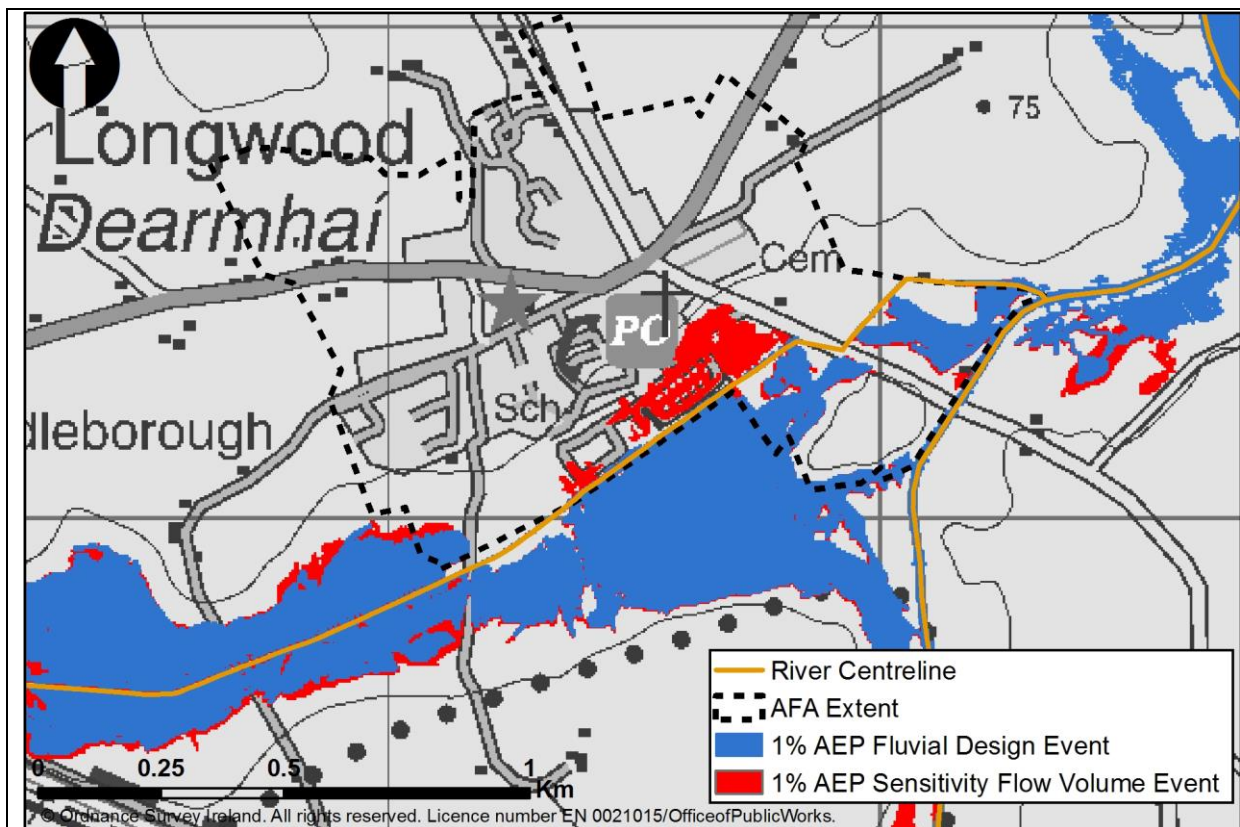
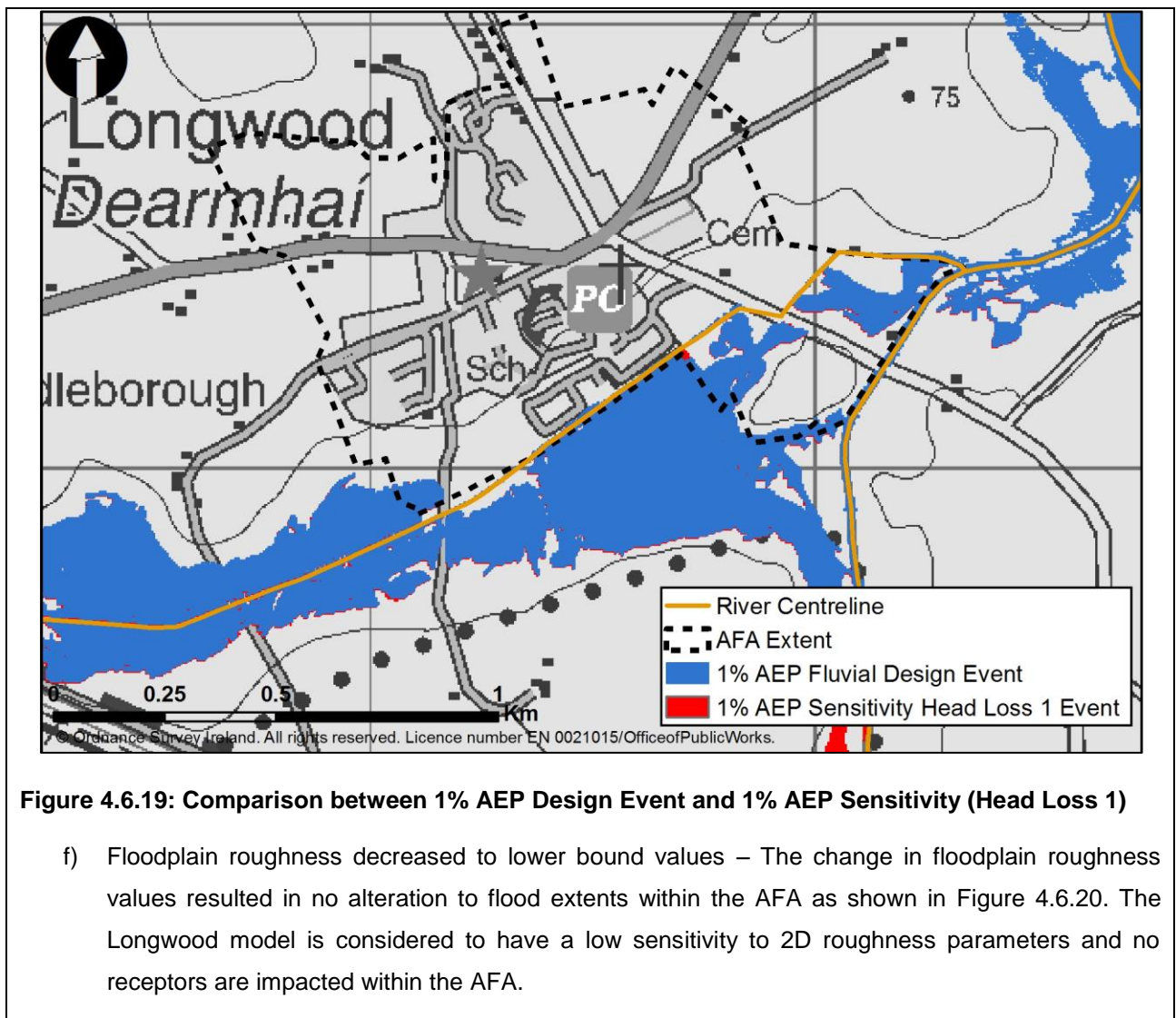


Figure 4.6.18: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Volume Event

- e) Afflux/Head loss at key structures –A model simulation has been conducted to assess the sensitivity of flood extents to adjusting the head loss coefficient a key structure. The structure 0736_00075I located on the Longwood Tributary watercourse was assessed. As shown in Figure 4.6.19 the Longwood model has shown it has a low sensitivity to parameter changes as there was little change in flood extents and no additional impact to receptors.



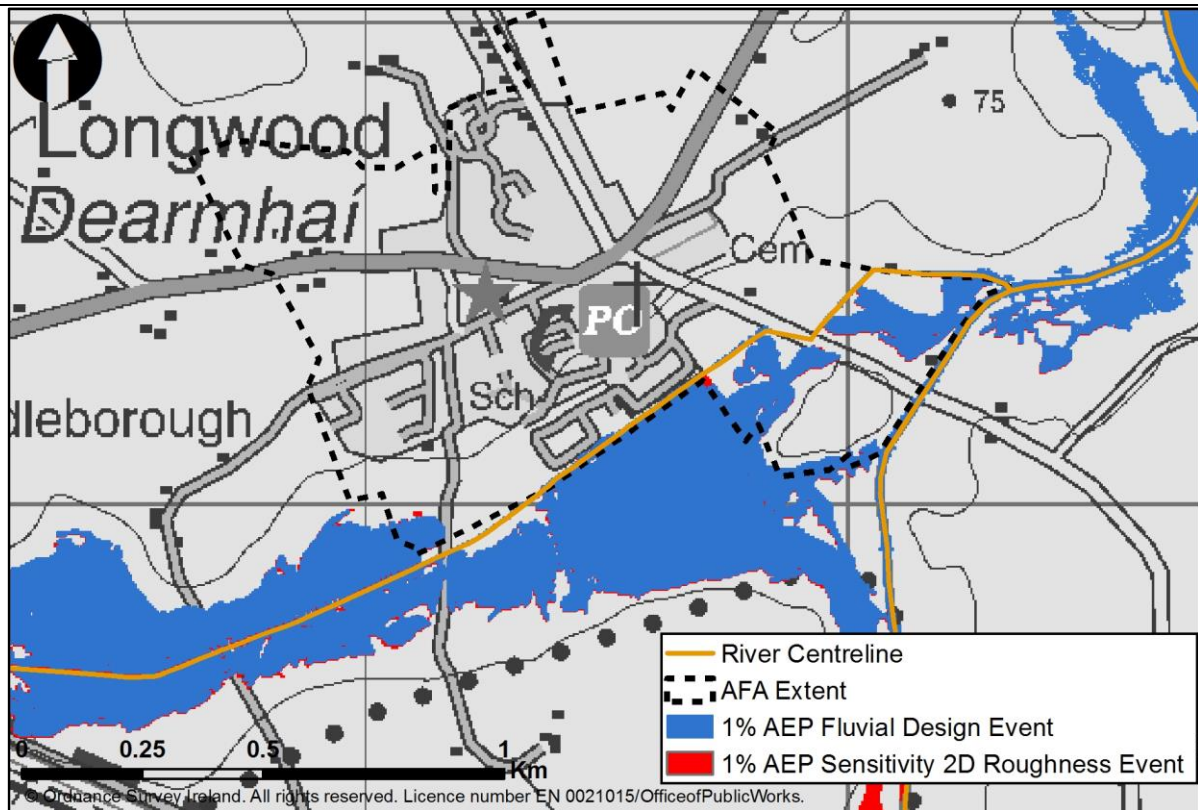


Figure 4.6.20: Comparison between 1% AEP Design Event and 1% AEP Sensitivity 2D Roughness Event

Table 4.6.1 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Longwood model. Of these parameters, the model indicates a high sensitivity to increase to increase model inflows. The Longwood model is assessed as having a high uncertainty/sensitivity as the Longwood Stream is ungauged. The Longwood model also indicates a high sensitivity to flow volume and 1D/2D model roughness. The model is less sensitive to the other parameters, with the resulting analysis identifying relatively minor increases in flood extents.

Table 4.6.1: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	High	High
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	High	High
1% AEP Sensitivity Flow Volume Event	High	High
1% AEP Sensitivity Head Loss 1 Event	Low	Low
1% AEP Sensitivity 2D Roughness Event	Low	Low

4.6.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (From IBE0600Rp0004_HA07 Inception Report unless otherwise specified):				
(a) AUG 2008.		In Longwood, historical information indicates flooding of low lying lands adjacent to Ashfield Bridge. This historical information at Ashfield Bridge, while in the vicinity of Longwood is located on the River Boyne and does not influence the Longwood model.		
(b) NOV 2002.		No detailed information was found for this event.		
(c) JAN 1995.		No detailed information was found for this event.		
Summary of Calibration The lack of historical flooding information at Longwood gives little quantitative data to calibrate the model to the larger flood events. However the model was calibrated to the spot gaugings at Castlerickard and the roughness values of the model adjusted accordingly see Section 4.6.5 (4) for details. A comparison between estimated and modelled flows at HEP points was carried out and the details of which are presented in Section 4.6.5 (5). A mass balance analysis was carried out to assess the difference between the discharge into the model and the volume of water stored with the discharge out of the model. Results showed a difference of 0.29%. This is within the acceptable limits as stated in the Environment Agency's Fluvial Design Guide. Whilst anecdotal information and available data has been used to the best extent possible, overall there is little or poor data to calibrate the model to and observation of more events would be necessary to reduce the uncertainty in model results.				
(2) Post Public Consultation Updates: Following informal public consultation and formal S.I. public consultation periods in 2015, analysis of the results of the rating reviews and their impact on the hydrological analysis for the UoM07 watercourses was finalised. This resulted in the hydrology and mapping outputs being updated, however there were no changes made to hydraulics. A revised set of flood hazard and risk mapping has been issued as Final to reflect this change.				
(3) Standard of Protection of Existing Formal Defences:				
Defence Reference	Type	Watercourse	Bank	Modelled Standard of Protection (AEP)
No Formal Defences				

(4) Gauging Stations:

There is one gauging station within the model extent, this is the Castlerickard (07003) water level and flow gauge.

Castlerickard (07003)

An OPW water level and flow gauge (07003) is located at Castlerickard on the Blackwater at chainage 14072m. There are 65 spot water level and flow gaugings recorded for the site from 18th June 1975 to 25th July 2001. This gauge has been given a B rating classification for the period from 1970 (i.e. flows can be determined up to the Q_{med} with confidence)

This gauging station was subject to a rating review (reported in IBE0600Rp0012_HA07 Hydrology Report_F01). The Longwood model's Q-h relationship was compared with that of the rating review model and the gauging station spot gaugings. The roughness values in the channel and at bridges were increased in order to agree with the initial rating review which showed good correlation with the spot gaugings as shown in Figure 4.6.21. Typically Manning's n values at bridges were increased to 0.035.

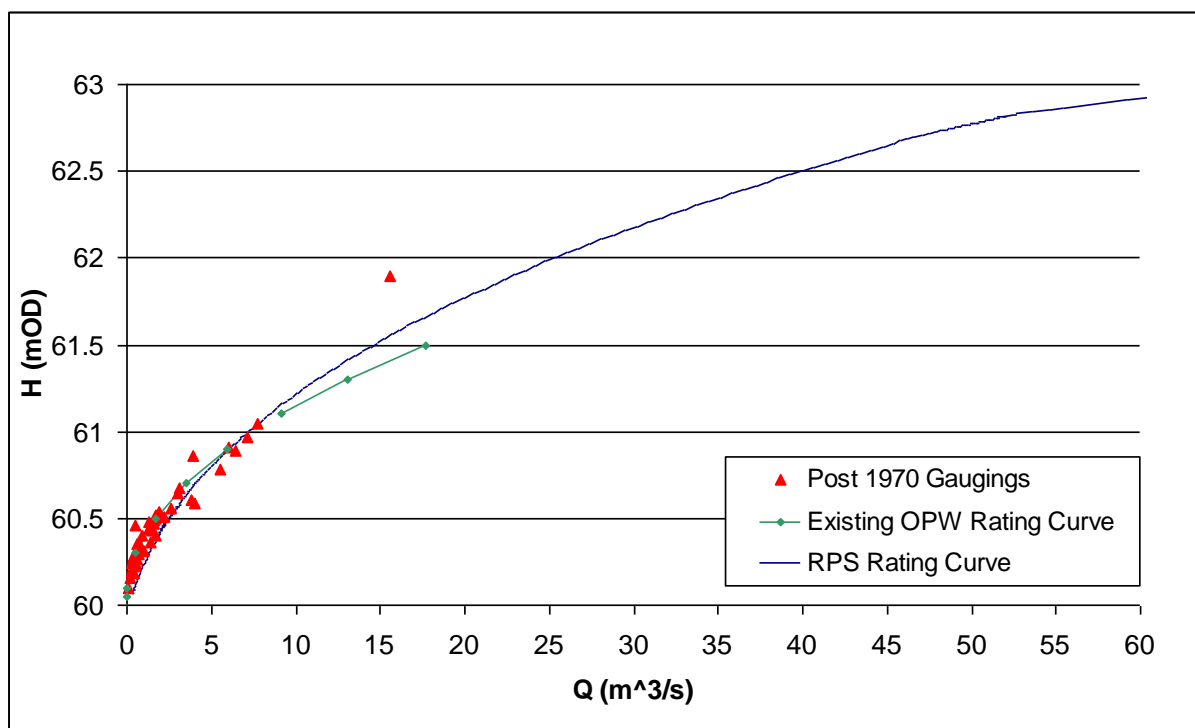


Figure 4.6.21: QH Relationship Comparison (between Johnstown Bridge model and OPW rating curve at Castlerickard Gauging Station)

(5) Comparison of Flows:

Table 4.6.2 provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

Table 4.6.2: Modelled Flows and Check Flows

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m3/s)	Model Flow (m3/s)	Diff (%)
LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD) 68.245	10%	-	0.14	-
	1%	-	0.21	-
	0.1%	-	1.41	-
LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD) 2346.19	10%	1.34	1.02	-23.73
	1%	2.51	1.55	-38.41
	0.1%	4.54	3.89	-14.42
BLACKWATER 8715.2	10%	24.67	24.67	0.00
	1%	36.84	36.84	0.00
	0.1%	53.99	53.98	0.02
BLACKWATER 14098	10%	33.03	33.62	-1.78
	1%	49.52	45.47	+8.19
	0.1%	71.43	58.92	+17.52

The table above provides details of flow in the model at every HEP inflow, check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

The modelled flows correlate well with the check flows at the upstream extent of the Blackwater River. The modelled flows at the downstream end of the Blackwater River were found to be consistently lower than the estimated flows by 12-17%. A review of the hydrological estimates has shown that the sum of the inputs is more than the estimated flow at the downstream HEP check point 07_954_3. There is a notable attenuation effect within the model and while the hydrological estimates have accounted for some attenuation it is not considered that it would capture the attenuation effects of the specific floodplain topography such as water stored just upstream of the Longwood Stream confluence which takes a peak flow of 10cumecs out of the Blackwater River (see Figure 4.6.22 below). Given that the attenuation increases as the flood event magnitude increases it is in line with expectations that the percentage difference between the check flows and model flows increases also.

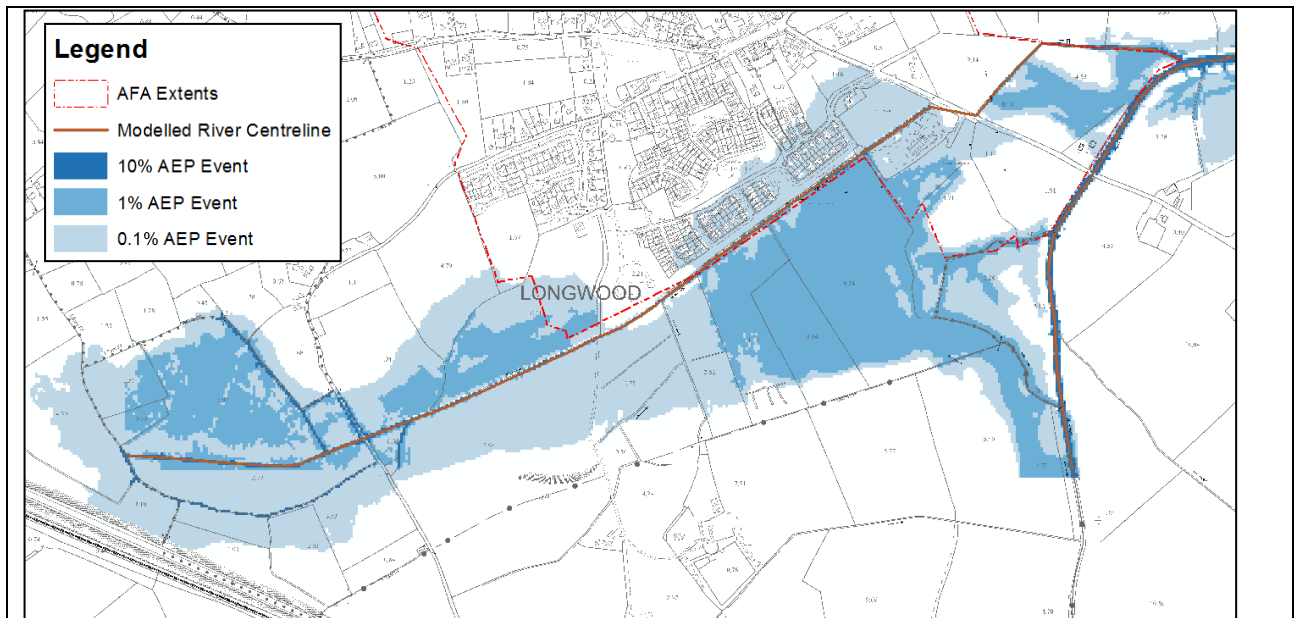


Figure 4.6.22: Interaction of Flood Extents of the Longwood Stream and River Blackwater

The flow applied to the Longwood Stream is a distributed flow along its entire reach. There is therefore no check flow to compare to the model flow at the upstream boundary. At the downstream boundary there is an increasing percentage difference between the check and model flows as the event being considered increases.

A complex hydraulic mechanism takes place along the Longwood Stream which makes identifying the cause of this discrepancy difficult. A percentage difference of -23.73% is recorded for the 10% AEP event. During this flood event little out of bank flooding occurs eliminating the possibility of lost flow due to floodplain attenuation. A model run was carried out whereby a low flow was applied to the Blackwater River. It was found that the downstream extent at chainage 2346.19m, had a peak flow of $1.34\text{m}^3/\text{s}$, giving a percentage difference of 1.5%. It can therefore be deduced that the percentage difference during the 10% AEP flood event is due to the negative flow from the Blackwater River travelling up the Longwood Stream.

A larger difference is present for the 1% and 0.1% AEP flood events. This is due to the backwater effects of the Blackwater River along with out of bank flooding which does not return to the Longwood Stream as shown in the Figure 4.6.22 above.

(6) Other Information:

The Longwood Stream was included in the Boyne arterial drainage scheme and the channel adjacent to the residential estate (Brackinrainey Manor) has been deepened and receives regular maintenance. Reports from OPW regional staff indicate the channel along this reach of the Longwood Stream to be close to top of bank during significant flood events (size/AEP unknown). The model results for the 10%AEP event at the same location shows a freeboard of approximately 200mm - 300mm, supporting OPW regional staff observations.

OPW regional staff also advised that they have never known the Longwood Stream to flood adjacent to

Brackinrainey Manor. This also is consistent with model result where no out of bank flooding occurs at this location during the 1%AEP event.

The watercourses in the Longwood model were assessed and modified to provide improved drainage to land owners under the ADS which involved delineating the benefiting land. A walkover survey was undertaken by land valuers to identify flood prone and marshy land that would benefit by the improved drainage. A good indication can therefore be obtained from the ADS benefiting lands as to the extent of a river's floodplain. A review was carried out for the Blackwater River and the Longwood Stream.

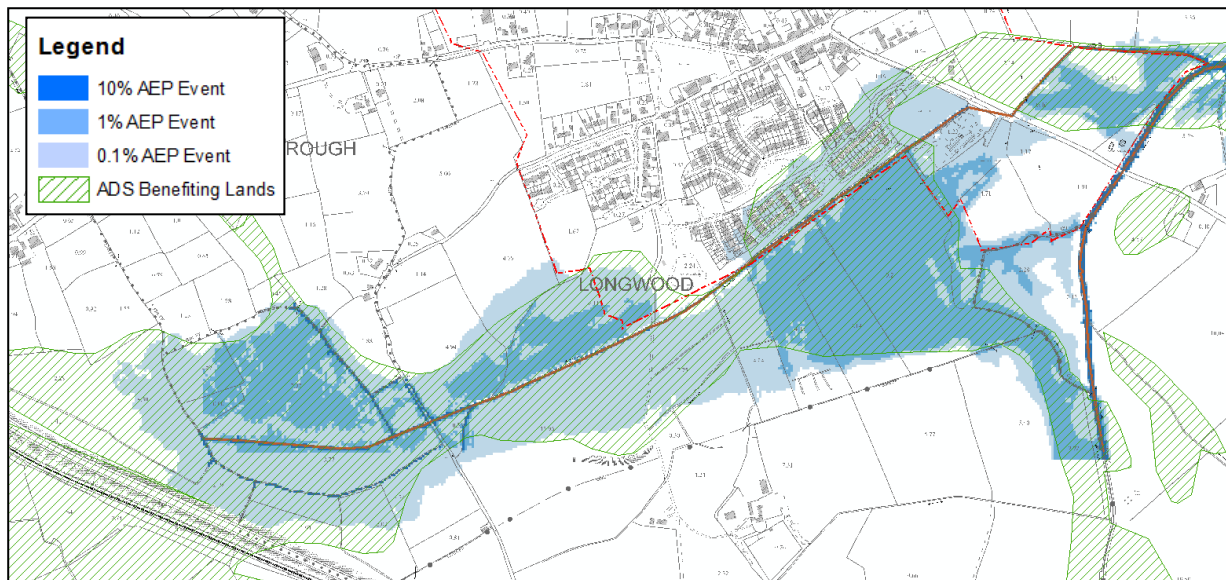


Figure 4.6.23: Comparison between ADS Benefitting Lands and Modelled Flood Extents (at the upper reach of the Longwood model)

Generally the model flood extents agree with the ADS benefiting lands around the Longwood area for both the Blackwater River and the Longwood Stream as shown in Figure 4.6.23. Extents differ where the model does not take account of certain watercourses. It should also be noted that the delineation of the ADS benefiting land includes land improved by drainage and not solely flood prone land.

As the model moves into the MPW, 1D modelling, the flood extents cover a smaller area than the ADS benefiting lands as shown in Figure 4.6.24. Extents differ where the model does not take account of certain unmodelled watercourses in addition it should be noted that the delineation of ADS benefiting land includes land improved by drainage and not solely flood prone land.

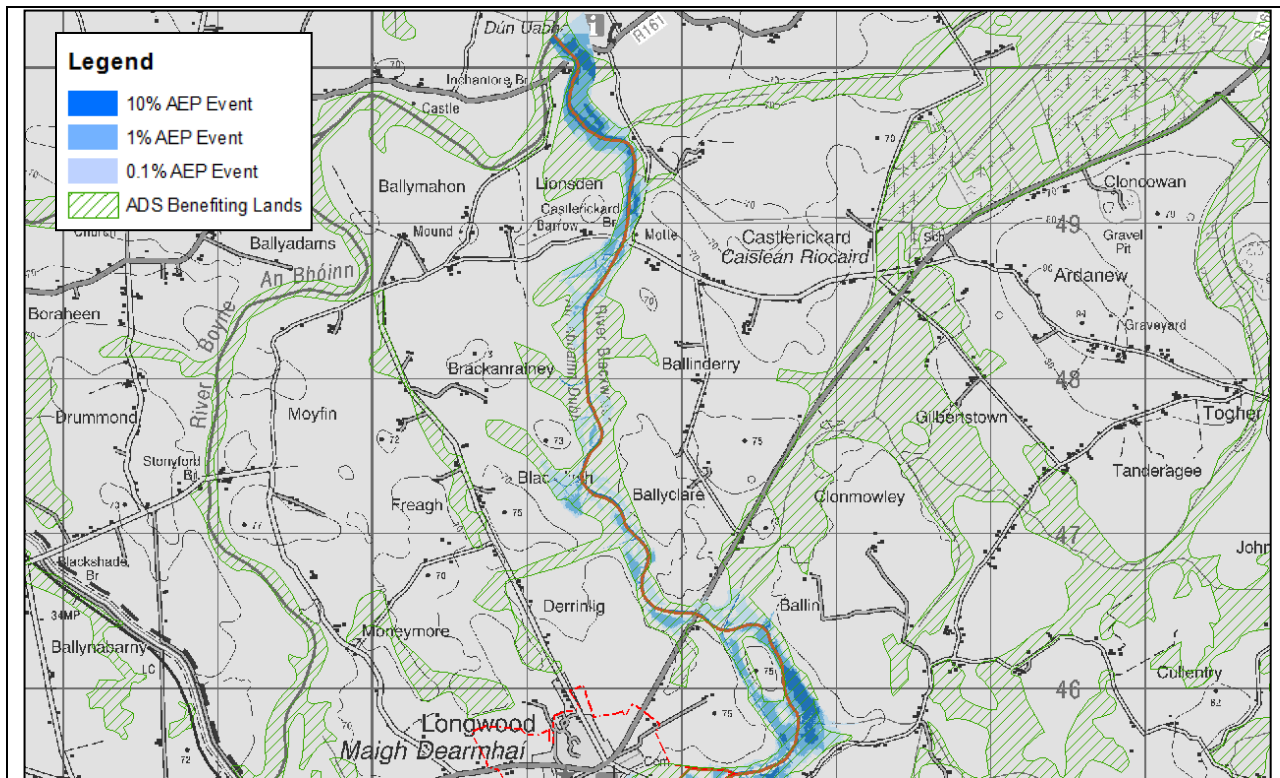


Figure 4.6.24: Comparison between ADS Benefitting Lands and Modelled Flood Extents (along the MPW reach of the Longwood model)

4.6.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

(a) A review of the roughness coefficients was carried out and extended into the out of bank areas along the 1D section of the River Blackwater. The roughness values were updated according to the calibration of the model at Castlerickard gauging station. This in general increased Manning's n from 0.030 to 0.040.

(b) The culvert located in the Longwood Stream from chainage 1805.9m to 1897.6m consists of a large culvert inlet which changes to a smaller culvert at the outlet. The geometry of the smaller culvert dictates the head loss through the culvert and the head water upstream of it and was therefore used to represent the whole culvert. This created a uniform geometry throughout the culvert which improved the stability of the model. Model runs with both the larger, original inlet and amended smaller inlet were carried out and the headwater levels compared. It was found that the headwater did not change from one modelling method to the other showing that the culvert is controlled by the outlet. This is typical of long, shallow culverts.

(c) The last two bridges on the River Blackwater (chainage 14067m & 15537m) have multiple additional arches on the floodplain. During extreme flood events out of bank flooding occurs and water would flow through these arches. As these bridges lie within the 1D section of the model the main channel and floodplain are represented by a single cross section and the bridge represented by a number of culverts in

parallel. A sensitivity test was carried out to ensure the model was representing the arches. The water level was reduced when the additional structures were added showing that the additional structures were allowing water to flow through them.

(e) The cross sections on the MPW (chainage 14,434m – chainage 15,718m) were extended beyond the limit of the topographical survey using the NDHM. A review was carried out post cross section extension to compare the topographical survey level with the NDHM. It was found that for many cross sections a discontinuity was occurring between the two survey datasets. Each cross section was reviewed individually and adjusted so the NDHM tied in with the topography survey. The assumption being that the topographical survey is the more accurate. Where the NDHM data could be offset horizontally to tie in with the topographical survey this was done in the first instance. Secondly where the NDHM seemed to be vertically offset the entire cross section extension to the left or right of the original cross sections was adjusted accordingly to tie it in. Lastly where the discrepancy is minor or the previous two methods are deemed inappropriate the NDHM data was smoothed to tie in with the topographical data.

(f) At the confluence between the Longwood tributary and the River Blackwater the bed level drops by approximately 0.8m. While this can sometimes cause instabilities in a model the Longwood model did not experience any. No amendments were therefore required.

(2) Hydraulic Model Limitations and Parameters:

(a) A 2 second time-step for both the MIKE 11 and MIKE 21 models has been selected in order to achieve a successful model simulation for all return periods.

(b) The lateral links along the upper reach of the Longwood Stream required an exponential smoothing factor of 0.2.

Hydraulic Model Parameters:

MIKE 11

Timestep (seconds)	2
Wave Approximation	High Order Fully Dynamic
Delta	0.5

MIKE 21

Timestep (seconds)	2
Drying / Flooding depths (metres)	0.2 / 0.3
Eddy Viscosity (and type)	0.25 (Velocity Based)

MIKE FLOOD

Link Exponential Smoothing Factor (where non-default value used)	0.2 at Longwood Stream ch0 - 486m
Lateral Length Depth Tolerance (m) (where non-default value used)	0.1

(3) Design Event Runs & Hydraulic Model Handover Notes:

At the onset of a 1% and 0.1% AEP flood event out of bank flooding occurs along the upper and lower reaches of the Longwood Stream with a minimal amount of flooding occurring along the middle where Brackinrainey Manor and Saint Nicholas National School are located. The water levels for all events are affected by a series of access bridges and culverts spread along the full length of the Longwood Stream as shown in Figure 4.6.25. The most dominant of which is located at the downstream extent of Brackinrainey Manor and upstream of Saint Nicholas National School at chainage 1636m.

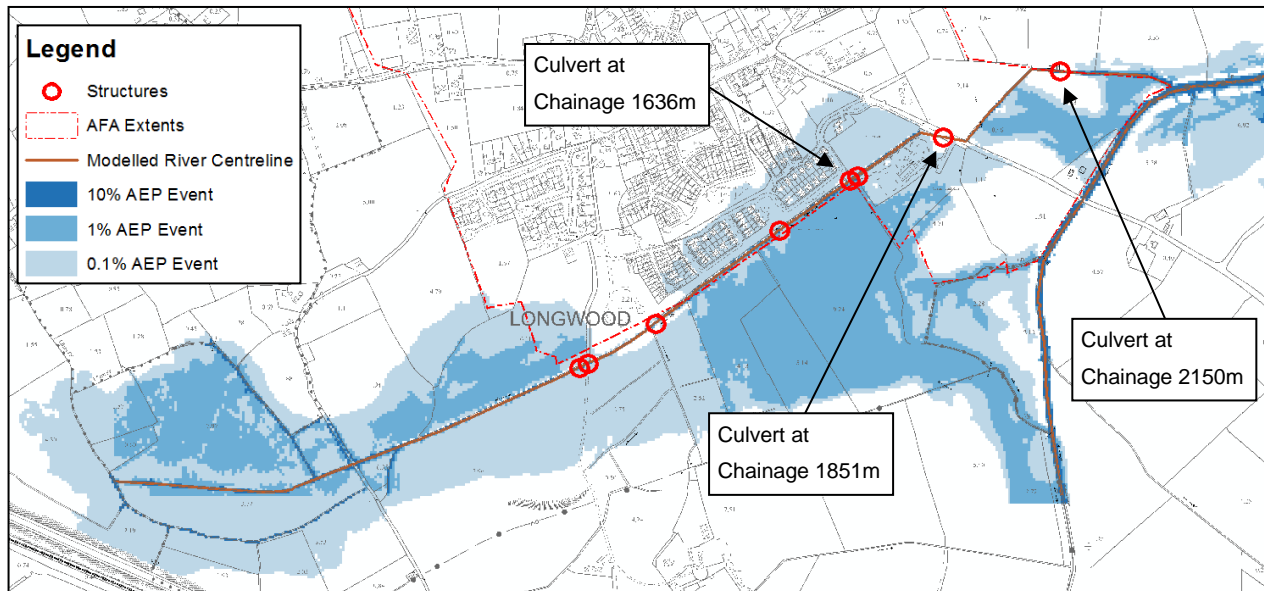


Figure 4.6.25: Location of Structures along the Longwood Stream in Relation to the Model Flood Extents

As the 1% and 0.1% AEP floods continue out of bank flooding occurs along the upper reach of the modelled Blackwater River. The flood water travels overland to join the Longwood Stream flood waters. Most of the access bridges overtop and/or are bypassed at this stage including the culvert at ch1636m during the 0.1% AEP flood event. In doing so the most dominant structure at controlling water levels changes to the long culvert located at chainage 1851m which travels under the L4020 road adjacent to Saint Nicholas National School. This occurs for the 0.1% AEP event only. During the 1% AEP flood event the culvert at chainage 1636m remains the dominant control structure.

It is only during the 0.1%AEP flood event that flooding to Brackinrainey Manor occurs. A small amount of out of bank flooding occurs at the upstream extent of the estate however the majority of flooding comes from downstream when the culvert at chainage 1851m surcharges and causes out of bank flooding along the left hand bank. The flood water flows into the estate and affects approximately 80 houses.

Downstream of the L4020 Road flooding occurs to the surrounding fields during the 1% and 0.1% AEP flood events. This is due to an undersized access bridge at chainage 2150m causing water to backup and cause out of bank flooding. Downstream of this culvert out of bank flooding occurs due to the water levels in the Blackwater River causing the water in the Longwood Stream to back up.

(4) Hydraulic Model Deliverables:

Please see Appendix F for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Mark Wilson
Model Reviewed by:	Stephen Patterson
Model Approved by:	Andrew Jackson

4 HYDRAULIC MODEL DETAILS

4.7 MORNINGTON

Name	Local Authority	Unique ID	SSA	Status	Date
Mornington	Meath	70880	AFA	Final	08/05/2017

4.7.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600Rp0001_Flood Risk Review_F02) highlighted Mornington as an Area for Further Assessment for fluvial and coastal flooding based on a review of historic flooding and the extents of flood risk determined during the PFRA. The Mornington River catchment emanates in the higher ground around the southern edge of Drogheda and approximately 4km west of Bettystown. The catchment drains higher ground in an easterly direction via a number of tributaries and then turns northwards when it reaches flatter ground approximately 300m inland of the sand dunes at Bettystown before eventually draining to the Lower Boyne Estuary approximately 1km downstream of the area known as Lady's Finger. At least three unnamed tributaries join up to form the Mornington River at the aforementioned flatter portion of the catchment and another drainage ditch (referred to as reach c2_1) draining the lower ground between the main river and the sand dunes to the south of Mornington. The Mornington River as it passes through the AFA is characterised by very flat reaches with flood defence embankments and walls running parallel to large portions of the reaches. At its downstream reaches the Mornington River is heavily tidally influenced and a flap valve structure is located close to Lady's Finger along the lower reaches of the Mornington River to prevent coastal flood waters from propagating further upstream.

In light of the recent Flood Alleviation Scheme undertaken at Mornington (the flood defences discussed above and constructed in 2010), initial design fluvial flow inputs were extracted from the report 'Mornington District Surface Water & Flood Protection Scheme' (Kirk McClure Morton, Jan 2004) and as such were not subject to the same hydrological analysis detailed in the HA07 Hydrology Report. The derivation of the flows for the total catchment and sub-catchments were based on delineation of the catchment area and hydrological properties and the application of the statistically based FSR methodology for un-gauged catchment design flow estimation. Prior to modelling RPS undertook to review these estimates against estimates derived using the FSU methods, released since the Mornington report was finalised.

Table 4.7.1 shows a comparison of the estimated fluvial design flows for the entire catchment.

Table 4.7.1: Estimated Total Mornington River Catchment Design Flows

Node ID_CFRAMS	AREA (km ²)	Q _{bar} / Q _{med}	Flows for AEP							
			50% (2)	20% (5)	10% (10)	5% (20)	2% (50)	1% (100)	0.5% (200)	0.1% (1000)
Previous Study – FSR	9.71	2.11	2.00	2.53	2.88	3.26	3.75	4.13	-	-
Eastern CFRAM Study – FSU	9.71	2.09	2.09	3.04	3.77	4.58	5.84	6.99	8.34	12.50

It can be shown from the table above that both methods are in good agreement at the index flood flow (approximately 50% AEP). As the frequency of events decreases however the discrepancy between the design peak flows based on the two methodologies becomes wider with the FSU derived flows for the 1% event nearly 70% higher. The discrepancy can be shown to be as a result of the growth curve which has been applied to the index flood flow in both cases. As discussed within the HA07 Hydrology Report the FSR methodology advises the use of a Regional Growth Curve for Ireland based on the available national pool of data at the time FSR was undertaken. This growth curve is largely based on large river data which is now known to display flatter growth curve behaviour, particularly in the Eastern side of Ireland. The Eastern CFRAM Study has applied an FSU based approach whereby growth curves are derived for individual catchments based on a tailored pooling group made up of gauging station years from hydrologically similar catchments. In the case of the Mornington catchment which is a small catchment within HA07 growth curve behaviour is considered to be much steeper when an analysis based on hydrologically similar catchments is undertaken. It is considered that the FSU approach is a more refined approach and as such the FSU based flows, utilising the existing catchment delineation, have been taken forward for use in the Mornington model. The generally higher flows discussed have been distributed proportionally across the model as per the catchment delineation undertaken for the original Study.

The Mornington River has been identified as a HPW and entire modelled watercourse is 1D/2D. The Mornington River and its three tributaries within the AFA have been modelled using the Infoworks ICM suite of software such that they could be integrated into the Lower Boyne Estuary model which utilised ICM in order to represent steep culverting within the system. The Mornington model reaches are part of the larger Drogheda / Mornington / Baltray model but for the purposes of reporting this section of the hydraulic report (4.7) refers only to the portions of the model which are relevant to Mornington AFA. This includes all of the Mornington River and its tributaries and the Lower Boyne Estuary. For reporting on the Drogheda and Baltray portions of the model see Section 4.3.

The Mornington River lies within the UoM08 and HA08 boundary despite the fact that it discharges to the Boyne Estuary. This may be as a result of historic mapping showing the Mornington River discharging directly to the sea at the coast rather than to the Boyne Estuary. For the purpose of the Eastern CFRAM Study it is considered appropriate that it is included within the UoM07 / HA07 reporting structure, particularly as the remainder of UoM08 was addressed under the pilot FEM FRAMS project. It is also recommended that any future review of these boundaries results in the Mornington catchment being included within HA07 given its outfall within the Boyne Estuary.

(2) Model Reference:		HA07_DROG7_F02	
(3) AFAs included in the model:		Drogheda / Mornington / Baltray	
(4) Primary Watercourses / Water Bodies (including local names):			
Reach ID	Name		
C2	MORNINGTON RIVER		
C2_1	MORNINGTON TRIB		
C2_3	MORNINGTON TRIB		
C2_4	MORNINGTON TRIB		
(5) Software Type (and version):			
(a) 1D Domain:	(b) 2D Domain:	(c) Other model elements:	
Infoworks ICM (Version 4.5)	Infoworks ICM Flexible Mesh		

4.7.2 Hydraulic Model Schematisation

(1) Map of Model Extents:
<p>Figure 4.7.1 illustrates the extent of the total Drogheda / Mornington / Baltray model and the extent of the model applicable to the Mornington AFA is shown in Figure 4.7.2. The extents, modelled river centre lines and HEPs are shown. The HEPs shown in relation to the Mornington AFA have been adapted from the previous Study 'Mornington District Surface Water & Flood Protection Scheme' (Kirk McClure Morton, Jan 2004) with adjustments to the original flows based on the differences outlined in Table 4.7.1 applied.</p> <p>.</p>

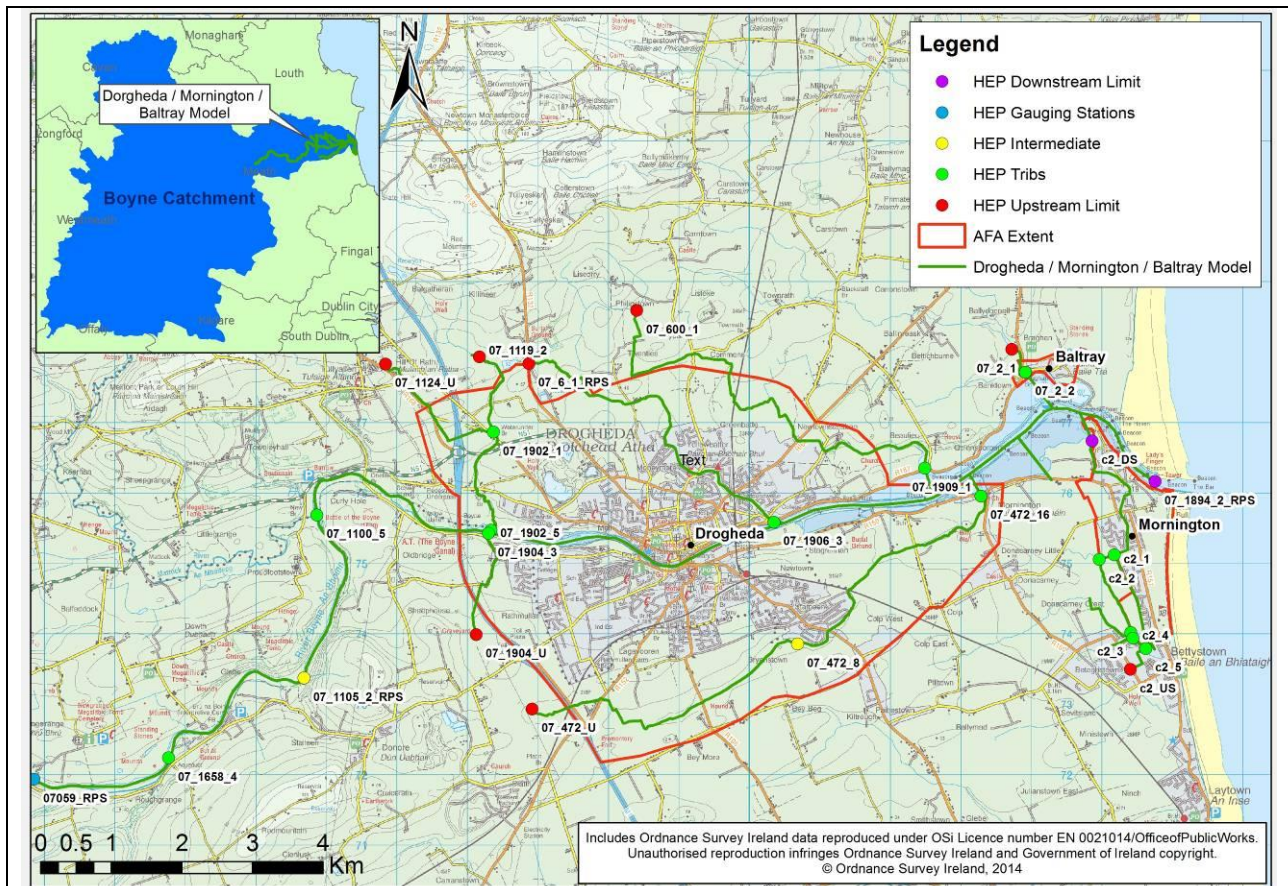


Figure 4.7.1: Drogheda / Mornington / Baltray Model Overview

The Mornington AFA contains seven HEPs, 1no. Upstream Limit, 1no. Downstream Limit and 5no. Tributary HEPs. Note that Upstream Limit HEPs on the modelled tributaries were not considered to be required as the tributaries represent catchments less than 1km^2 . As such a simplified conservative approach has been applied where the total tributary catchment flow has been applied at the upstream extent (except where otherwise discussed in this report).

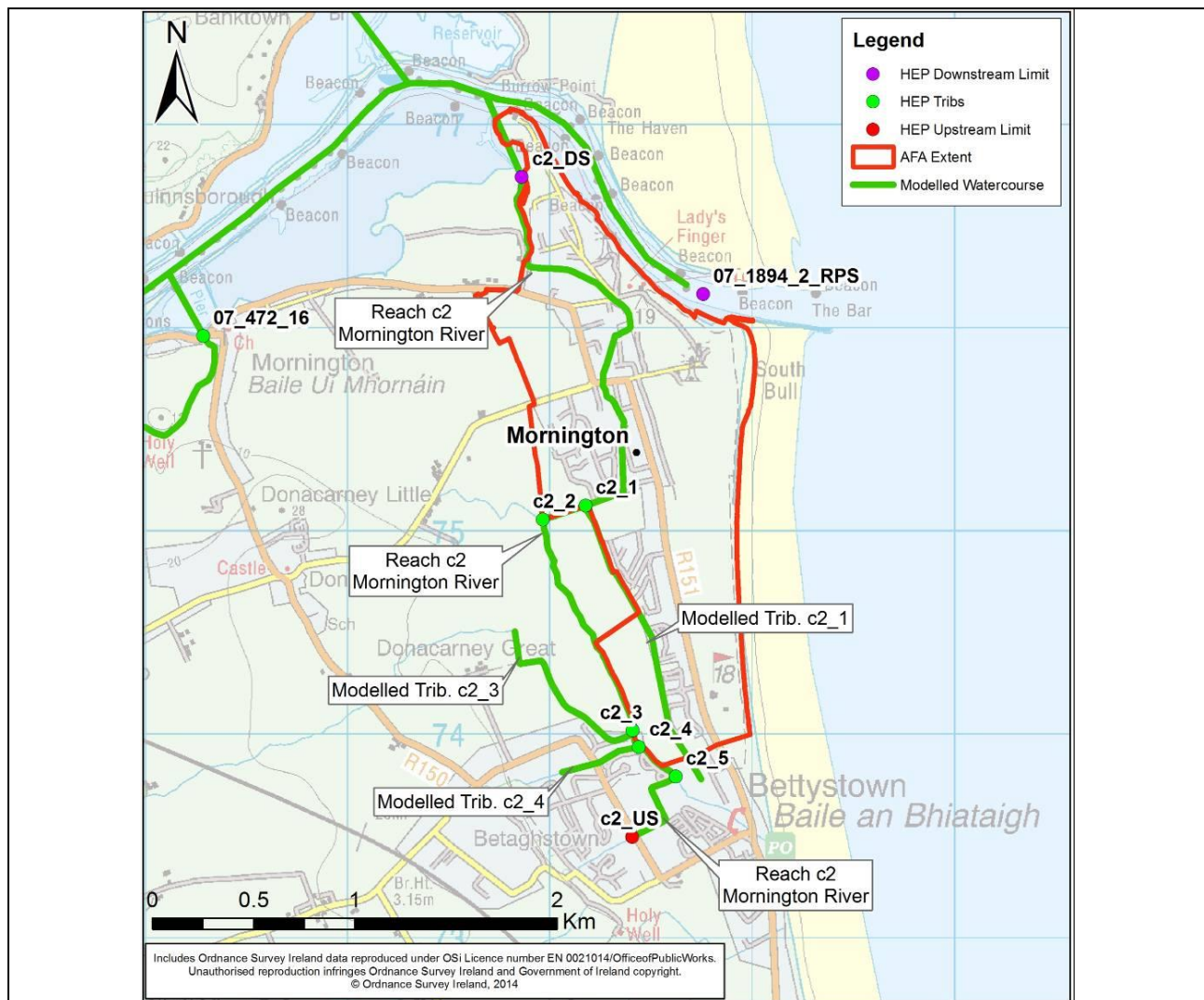


Figure 4.7.2: Mornington AFA Model Extents

2) x-y Coordinates of River (Upstream extent):

Reach ID	River Name	x	y
C2	MORNINGTON RIVER	315406	273498
C2_1	MORNINGTON TRIB	315746	273782
C2_3	MORNINGTON TRIB	314825	274507
C2_4	MORNINGTON TRIB	315060	273815

(3) Total Modelled Watercourse Length:

C2	4.25km
C2_1	1.46km
C2_3	0.93km
C2_4	0.45km

(4) 1D Domain only Watercourse Length:

0 km

(5) 1D-2D Domain

Watercourse Length:

C2	4.25km
C2_1	1.46km
C2_3	0.93km
C2_4	0.45km

(6) 2D Domain Mesh Type / Resolution / Area:	Infoworks ICM Flexible Mesh / 1 - 25m ² / 7.8km ²
<p data-bbox="178 280 550 313">(7) 2D Domain Model Extent:</p> <p data-bbox="178 347 1465 515">The 2D domain of the model extends to over 2km upstream of the M1 Boyne Bridge at Drogheda to the coast. In relation to the Mornington AFA the 2D domain covers the entire Lower Boyne Estuary and covers the entire modelled extents of the Mornington River from the Boyne Estuary to Bettystown as shown in Figure 4.7.3.</p> <p data-bbox="178 548 1465 772">Figure 4.7.3 shows the model 2D domain that was constructed from LiDAR data; following a visual inspection an error in the LiDAR data was identified to the west of Mornington. This was not considered to have any significant impact upon flooding within the AFA as it is outside the extents and only affects higher ground. This was checked and verified following the completion of initial extreme AEP modelled flood events, consequently no topographic adjustment to the 2D model was required.</p>	

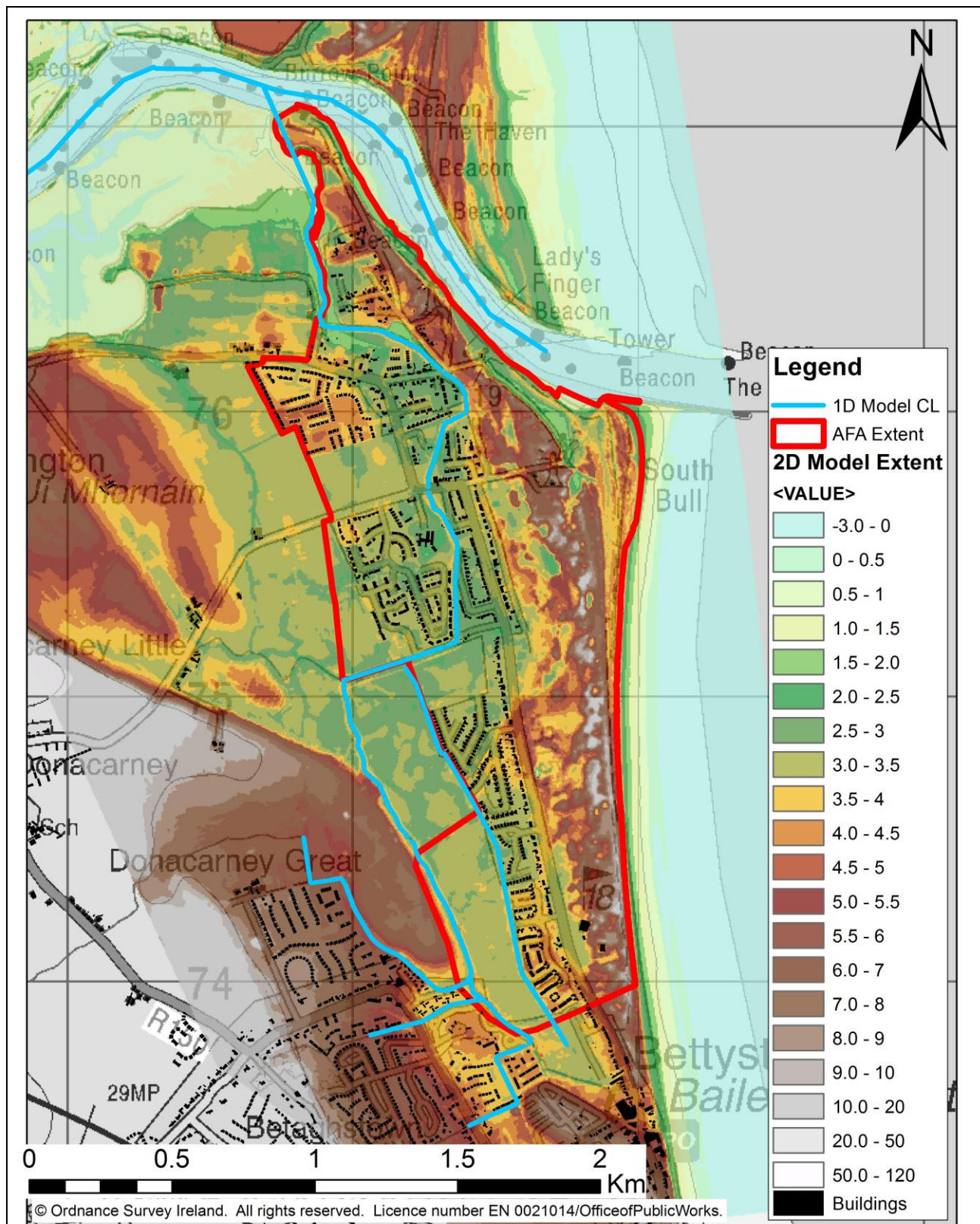


Figure 4.7.3: Mornington 2D Model Extents

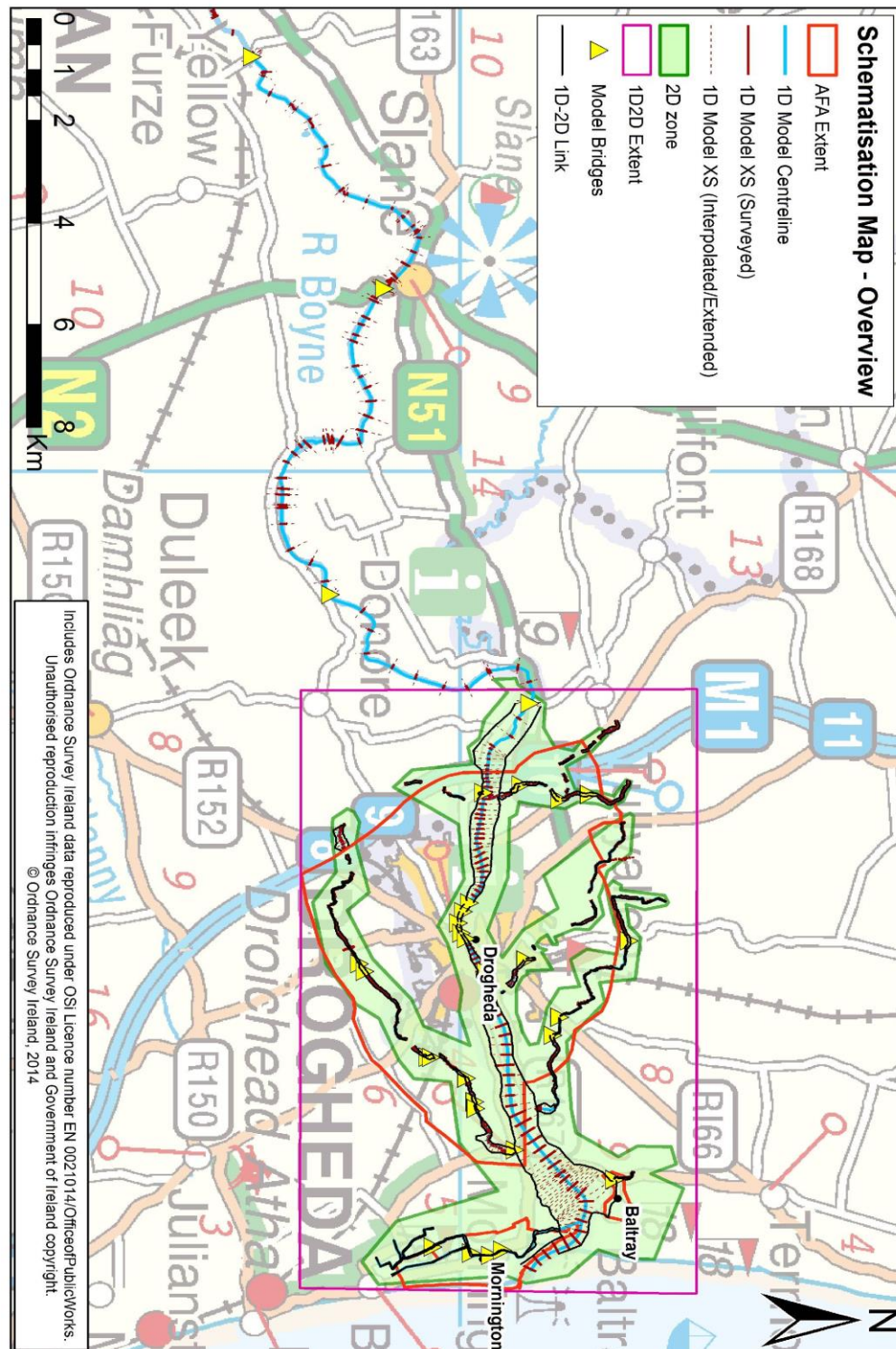


Figure 4.7.4: Drogheda / Mornington / Baltray Model Schematisation Overview

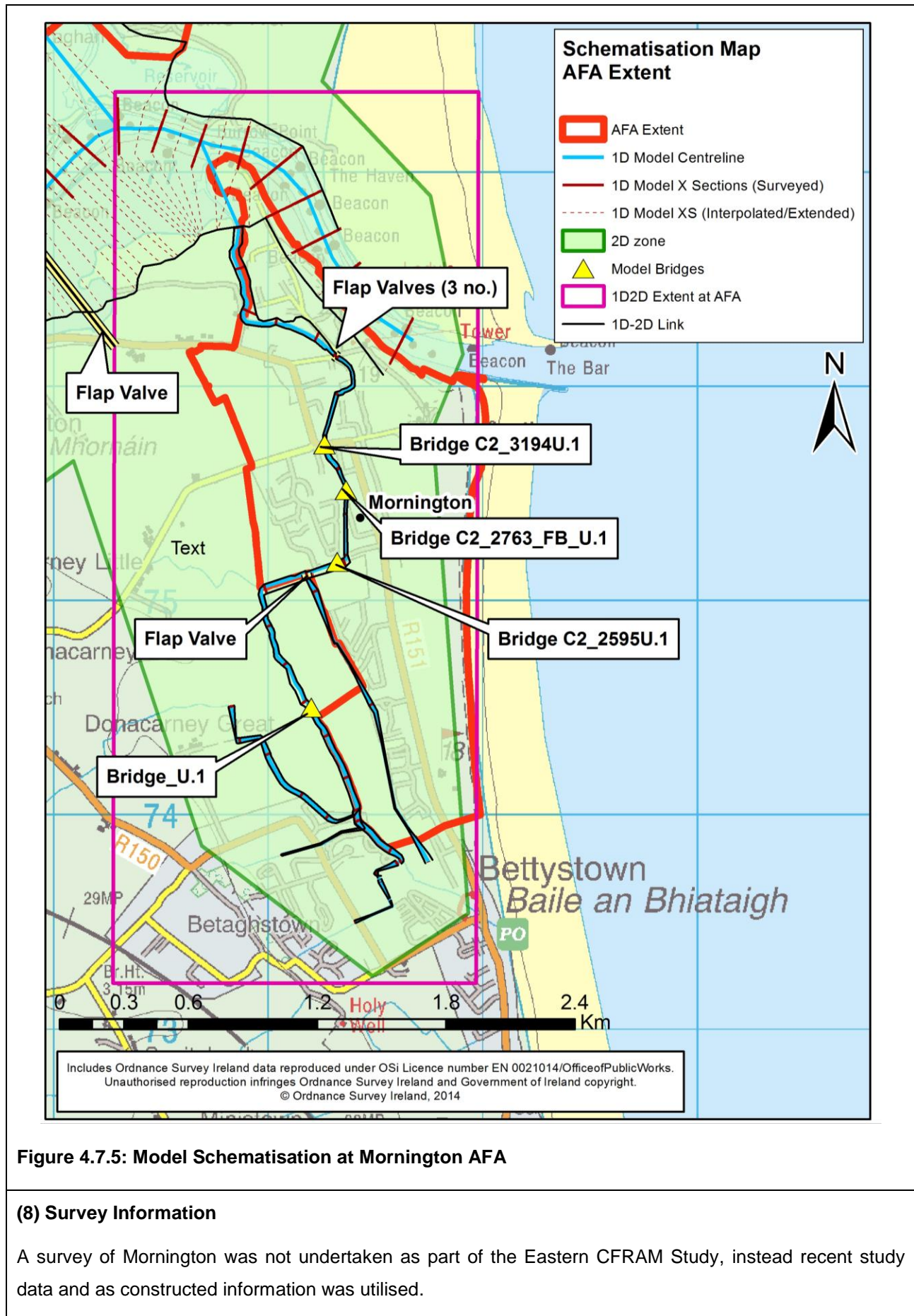


Figure 4.7.5: Model Schematisation at Mornington AFA

(8) Survey Information

A survey of Mornington was not undertaken as part of the Eastern CFRAM Study, instead recent study data and as constructed information was utilised.

(a) As Built Drawing References:**LAYOUTS**

IBE0325 201_1 LAYOUT 1 OF 3

IBE0325 201_2 LAYOUT 2 OF 3

IBE0325 201_3 LAYOUT 3 OF 3

DRAINAGE

IBE0325 601 ST MARYS PROPOSED OVERFLOW DRAINAGE LAYOUT

IBE0325 301 DRAINAGE LAYOUT

STRUCTURES

HOLLOW BLOCK WORK WALL SKETCH

IBE0325 RW02-A LADYS FINGER RETAINING WALL DETAILS

IBE0325 RW03-A RETAINING WALL AT SALT MARSHES

IBE0325 RW04-A RETAINING WALL AT SALT MARSHES REBAR DETAILS

IBE0325 RW05-A RC WALL L6-L9 TYPE 1

IBE0325 BE01-B BRIDGE EXTENSION AT HARRYS SHOP

IBE0325 BE02-A RC DETAILS

IBE0325 BP01-C LADYS FINGER BRIDGE PARAPET DETAILS

IBE0325 BP02-C LADYS FINGER BRIDGE PARAPET RC DETAILS

IBE0325 BP10-A HARRYS SHOP BRIDGE UP-STREAM PARAPET GA & RC DETAILS

IBE0325 FB01-B FOOTBRIDGE RELOCATION BASE DETAIL

IBE0325 FB02-A FOOTBRIDGE ABUTMENT REBAR

IBE0325 FB03-A FOOTBRIDGE DECK REBAR

(c) LiDAR Survey Reference:

ISA-47 Drogheda (2m x 2m grid)

(9) Survey Issues:

It is assumed that the information provided for model construction as listed below provides all the relevant information to develop an accurate representation of the Mornington River HPW through the AFA:

- Existing 'Mornington District Surface Water and Flood Protection' scheme model
- 'Mornington District Surface Water and Flood Protection' scheme 'as constructed' drawings
- CFRAM Study LiDAR

It was noted that the development called Northlands was constructed since the Mornington scheme was completed and as such the watercourse and hence 1D section through this area was considered to have changed (partially culverted). Although this area is located outside the AFA extents details of the culverts have been collected since the draft mapping was produced and added to the model to ensure that this area is accurately represented.

The surveyed cross sections along the MPW reach of the River Boyne did not extend a sufficient distance beyond top of bank to cover the entire floodplain. All surveyed cross sections along this reach were augmented using the NDHM dataset to enable full coverage of the MPW floodplain to be included in the 1D domain of the model. The transition between the surveyed cross section dataset and the NDHM was manually edited to ensure a smooth transition between the datasets within the ICM software. No additional edits were made to the data extracted from the NDHM.

4.7.3 Hydraulic Model Construction**(1) 1D Structures (in-channel along modelled watercourses):**

See Appendix G

Number of Bridges and Culverts: 10

Number of Weirs: 0

The structures along the modelled reach of the Mornington River were included in the existing model. Parameters for these structures were checked against existing photographs and site walkover survey photographs to ensure model parameters were within reasonable bounds. Further details are included in Chapter 3.5.1. A discussion on the way structures have been modelled is included in Chapter 3.4.3 and an overview and AFA specific map are shown in Figure 4.7.4 and Figure 4.7.5

A number of structures constrict flow, in the most extreme events the flood flow is generally constrained within the defences. Where the defences are breached within the AFA for the most extreme events, flooding is dominated by a backwater effect emanating from the downstream coastal / fluvial boundary dictated by flood levels in the Boyne Estuary. Therefore, two structures are considered critical to flooding within the AFA, including the bridge located at Harry's Shop and the flap valve arrangement at the bridge leading over to Lady's Finger. The locations of these critical structures are shown in Figure 4.7.6.

A number of other structures which are not critical to flooding are heavily surcharged during the most extreme flood flow events such that the out of bank / deck flow is greater than the flow through the structure. This is the case for the field access bridge to the west of Foxbury (upstream node: Bridge_U)

and the three culverts in the vicinity of Northlands (upstream node: North_MH_U, 01BETT00220J.In and 01BETT00210I.In)

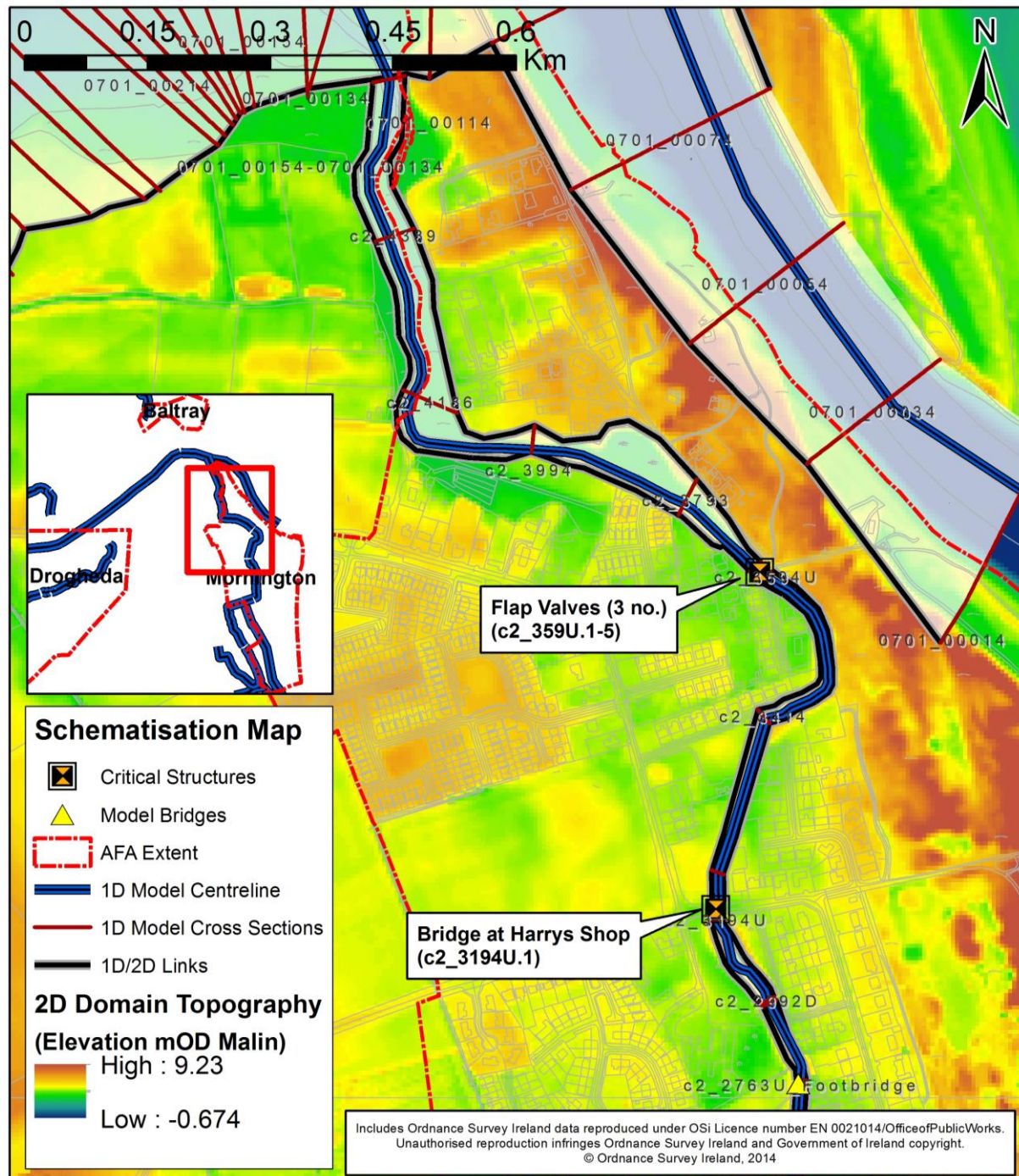


Figure 4.7.6: Schematisation of Critical Structures within the Model

Figure 4.7.7 below shows critical structure (c2_3194U.1, chainage 2918m) this bridge allows the R151 to cross over the Mornington River; it is located adjacent to Harry's (Centra) shop. Modelling results show that the relatively small opening and reduced hydraulic capacity result in a significant afflux at this structure during the 1% and 0.1% AEP events.

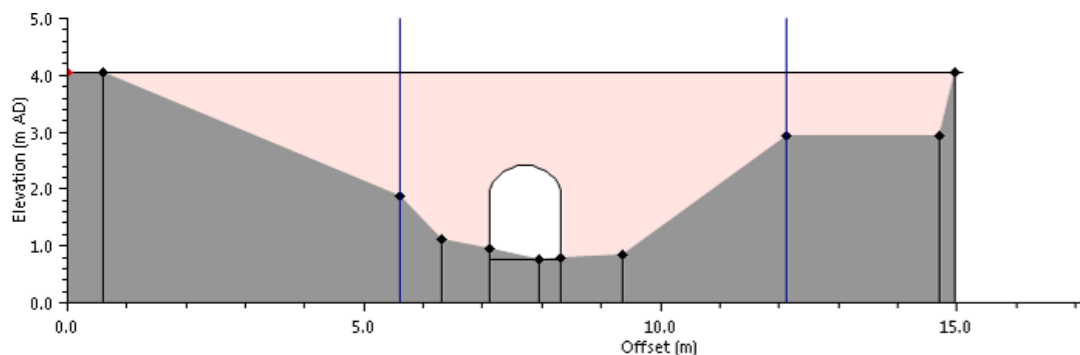


Figure 4.7.7: Critical Structure Identified on the Mornington River (at chainage 2918m ID: c2_3194U.1 adjacent to Harry's Shop)

Figure 4.7.8 below show a flap valve structure at Lady's Finger (chainage 3423m ID: c2_359U.1 to 5). This critical structure is located on the downstream reach of the Mornington River. Hydraulic modelling has illustrated that this particular structure prevents coastal flood water from propagating from the Boyne Estuary up the Mornington River for all design event scenarios. The structure is a stone triple arch bridge with tilting disc type tidal valves located on the downstream face of each of the arches. The invert of the flap valves is approximately 0.9m OD Malin which is below the average high tidal level. During all events the fluvial discharge from the Mornington River is controlled by the coastal water level. The inclusion of the flap valves within the model prevents the propagation of flood waters into the Mornington River channel and displacing a large volume of flood storage within the channel and between the flood defence embankments. Recent correspondence from the local authority indicates that the flap valves may have performed poorly in a recent fluvial event restricting flow out of the Mornington River at low tidal level. For the purposes of design runs headloss values of 1.0 have been retained which is considered appropriate for cast iron disc type valves. The performance of these flap valves is considered further in Section 4.7.4

to ascertain the impact of a poor performance of these valves.



Figure 4.7.8: Critical Structure (identified on the Mornington River at chainage 3423m Lady's Finger, upstream face pre flood alleviation works (top) and downstream face (right) post works)

In addition to the critical structures discussed, there are a number of footbridges located along the modelled reaches of the Mornington River (see Figure 4.7.5). These are not considered critical to flooding

as they do not provide significant constriction to flood flow. From a review of the hydraulic model it is clear that the flow constriction which may occur at the levels of the bridge decks is not a significant contributor to flooding as the dominant flood mechanism is the total water level resulting from fluvial flows building up behind the flap valved outlet at Lady's Finger which itself is controlled by tidally dominated total water levels in the Boyne Estuary.

In addition to the footbridges there is a flap valved outlet at the downstream extent of reach c2_1, the drainage ditch that drains the portion of low lying land between the Mornington River and the coastline. The structure is included within the model but is not considered critical to flooding as it is located above the mean high water tidal level and approximately 1.2km upstream from the flap valved outlet at Lady's Finger. This reach receives very little flow and flooding from the reach is only apparent in the 0.1% AEP modelled fluvial scenario when large areas of the Mornington River system breach the flood embankments due to water levels within the system.

All of the tributary watercourses (c2_1, c2_3 and c2_4) which have been included in the model represent catchments which are less than 1km² and, with the exception of the flap valve on reach c2_1, structures have not been surveyed on these watercourses but the model reaches have been retained from the existing model for improved detail in flood mapping and better representation of inflows to the Mornington River.

(2) 1D Structures in the 2D domain (beyond the modelled watercourses):

None

(3) 2D Model structures:

None

(4) Defences: Table 4.7.2, presents a list of the modelled Mornington flood defences.

Table 4.7.2: Mornington schedule of flood defences

Type	Watercourse	Bank	Model Start Chainage (approx.)	Model End Chainage (approx.)
Embankment	C2 (Mornington River)	RIGHT	481	2330
Wall	C2 (Mornington River)	RIGHT	2330	2697
Embankment	C2 (Mornington River)	RIGHT	2697	2719
Wall	C2 (Mornington River)	RIGHT	2719	2796
Embankment	C2 (Mornington River)	RIGHT	2796	2908
Wall	C2 (Mornington River)	RIGHT	2924	3162
Embankment	C2 (Mornington River)	RIGHT	3162	3252
Embankment	C2 (Mornington River)	RIGHT	3403	3458
Embankment	C2 (Mornington River)	RIGHT	3648	4220
Embankment	C2 (Mornington River)	LEFT	1714	2131
Wall	C2 (Mornington River)	LEFT	2131	2645
Embankment	C2 (Mornington River)	LEFT	2645	2925
Embankment	C2 (Mornington River)	LEFT	2945	3330
Wall	C2 (Mornington River)	LEFT	3330	3428
Wall	C2 (Mornington River)	LEFT	3447	3494
Embankment	C2 (Mornington River)	LEFT	3494	3901
Embankment	C2 (Mornington River)	Perpendicular to watercourse	3918	3918
Flap Valve	C2_1	N/a	1457	1457
Flap Valve	C2_2	N/a	630	630

(5) Model Boundaries - Inflows:

Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report_F01- Section 4.7 and Section 4.7.5(5)). The main inflow hydrograph to the Mornington River implemented in the model at node c2_185 is shown in Figure 4.7.9.

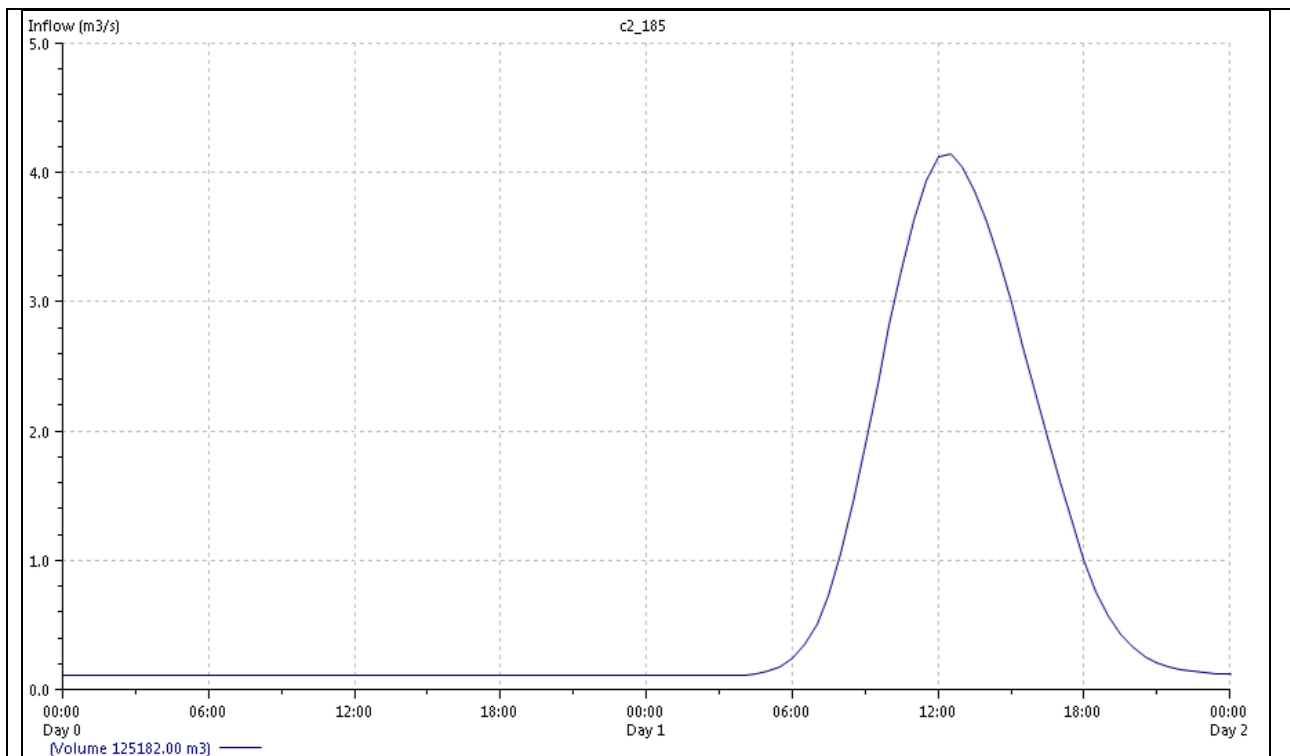


Figure 4.7.9: Upstream 1% AEP Inflow Hydrograph to Mornington River at c2_185

The upstream boundary of the Mornington River is located at node ID c2_185 in the model; a point inflow was applied at this node to account for flow entering the Mornington River upstream of this location. A point flow was applied at Node ID c2_2190a to account for inflow from a small tributary not included in the model due to its catchment size. Point flows were also applied at the upstream extent of two of the modelled watercourses c2_3 and c2_4 based on the total flow in the sub-catchment to the HEP (as shown in Figure 4.7.2). This conservative approach was considered appropriate given the small catchments (less than 1km²).

For the watercourse c2_1, 25% of the flow was applied as a point flow at the upstream extent with the remaining 75% applied laterally to the reach. This modelling approach was applied when it became clear from initial model runs that applying the total catchment flow at the upstream limit was overly conservative. This catchment is elongated and the vast majority of the catchment is thought to drain to the watercourse downstream of the upstream boundary. Discussions with OPW indicated that there are connections between this tributary and the Mornington River. There is no survey data of these connections or further information on the interaction of these two watercourses. Even with the flow dispersed more proportionally along the length of the reach c2_1 flood extents could not be validated along this undefended reach. A review of the catchment delineation indicates that there is great uncertainty as to where the flat areas between this drain and the Mornington River drain to. In light of the uncertainty in this very small catchment (less than 1km²) it was considered appropriate that a proportion of the catchment flow from the tributary was dispersed to the Mornington River.

All of the inflow hydrographs were delayed by 28 hours such that peak flows coincided with the peak flooding conditions in the Boyne Estuary as a result of the extreme coastal water level boundary discussed in Section 4.7.3 (6).

(6) Model Boundaries – Downstream Conditions:

The modelled downstream boundary condition is taken from the ICPSS node NE_09 at the mouth of the Boyne Estuary where it meets the Irish Sea such that the effects of extreme coastal water levels in the Boyne Estuary can be considered within the Mornington AFA. A range of extreme coastal water level boundaries from 50% AEP to 0.1% AEP have been considered.

The extreme coastal water level boundary has been developed using a tidal cycle halfway between an astronomical mean high water and a mean high water spring tide at Dublin Port. A typical 48 hour surge profile has then been applied to achieve a peak water level at the appropriate ICPSS extreme water level for NE_09 node. The fluvial design hydrographs have been shifted in time such that the peak occurs simultaneously with peak water level.

The likelihood of joint occurrence of extreme fluvial events and extreme coastal events is discussed in the Hydrology Report and it is not considered that there is evidence of significant joint occurrence. However boundary conditions for the less dominant events are maintained at 50% AEP for fluvial and coastal dominated model runs in line with a precautionary approach.

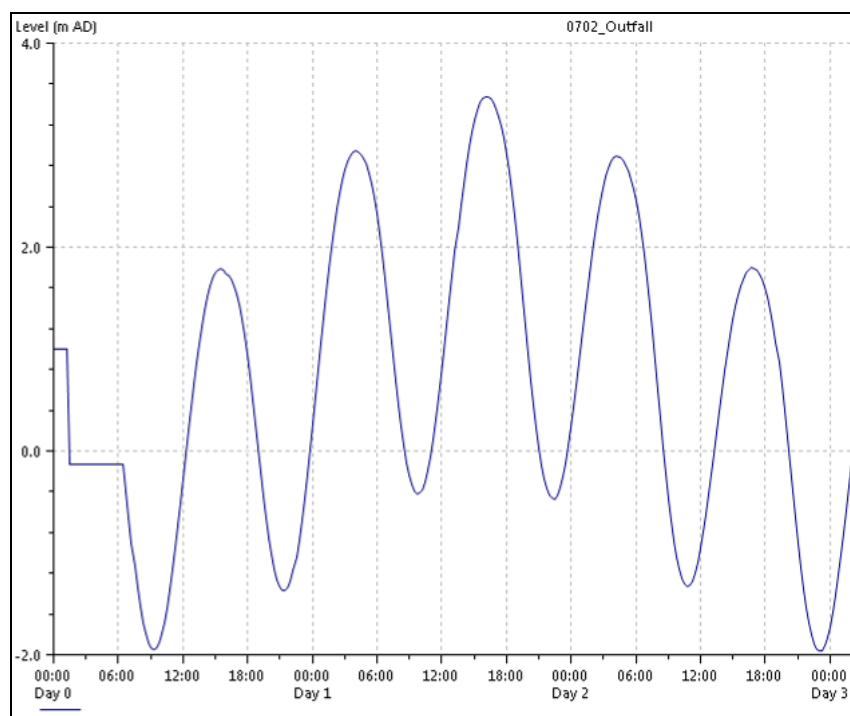


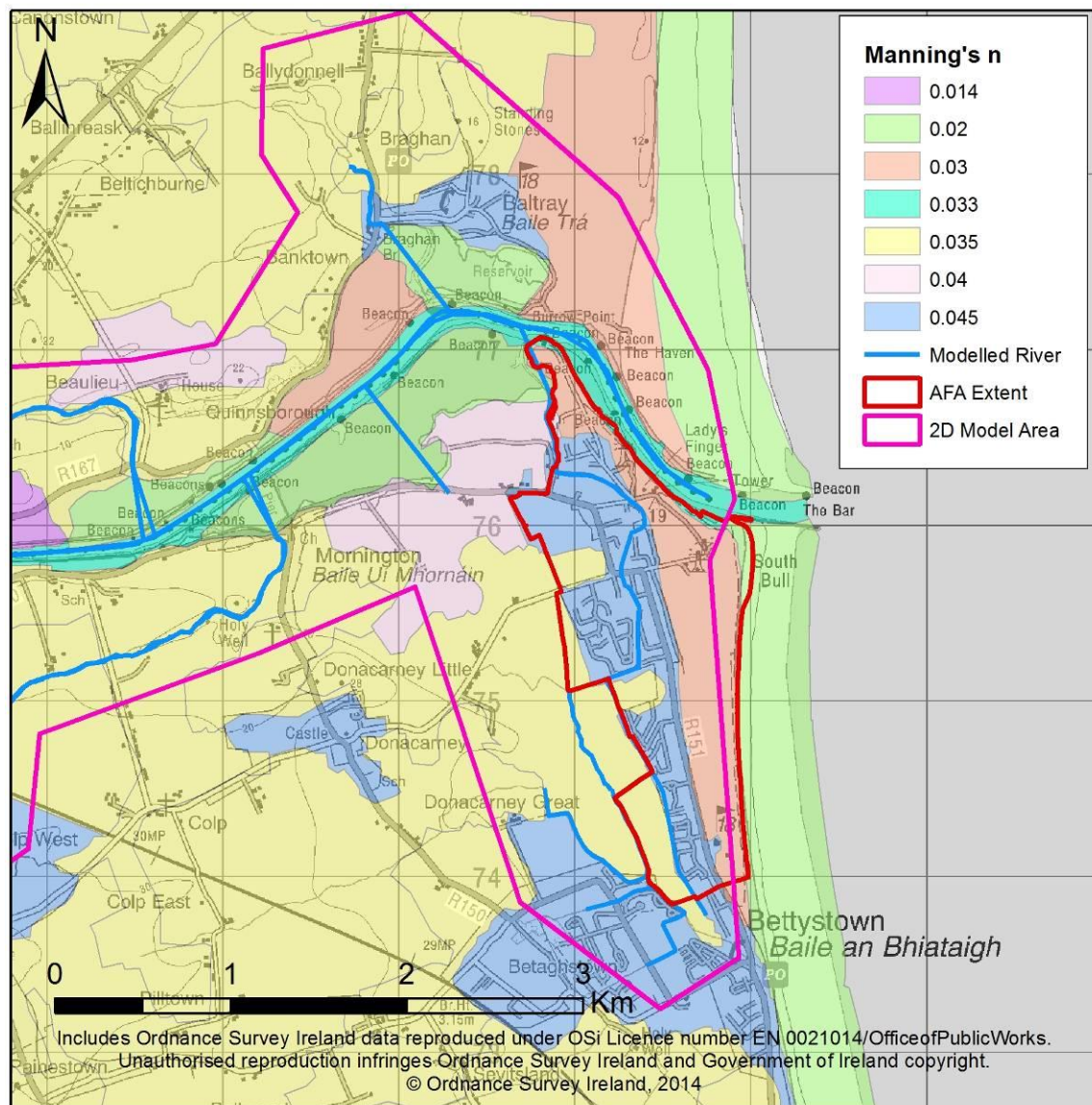
Figure 4.7.10: 0.5% AEP Coastal Water Level Boundary (applied at 0701_Outflow)

The coastal boundary has been applied at the model node '0701_Outflow' representing a 1D boundary at the mouth of the Boyne Estuary where it meets the Irish Sea. It is not considered that the boundary needs to be applied to a 2D boundary at this location as the channel is well defined with high ground to either side.

It is also considered that there is no flood risk to Mornington directly from the open coastline as a constant high level (above 4m) is provided between the beach and the Mornington AFA.

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.055
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.055
(c) MPW/HPW Out-of-Bank (2D)	Minimum 'n' value: 0.020 (Inverse of Manning's 'M')	Maximum 'n' value: 0.045 (Inverse of Manning's 'M')

**Figure 4.7.11: Map of 2D Roughness (Manning's n)**

This map illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the Corine Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset. Any values seaward of the high water were taken as 0.033 unless otherwise specified.

(d) Examples of In-Bank Roughness Coefficients

XS_ID:c2_3194 – (Harry's Shop)

Manning's $n = 0.035$

Natural stream – clean winding



XS_ID: c2_7754d – (d.s. of Northlands)

Manning's $n = 0.055$

Natural stream – sluggish reaches, weedy deep pools

Figure 4.7.12: Examples of In-bank roughness values (Manning's n)

4.7.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact to the present day 1% AEP hydraulic model simulation within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a low increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.7.13. This outcome indicates that the Mornington model demonstrates a low sensitivity to changes in roughness parameters. This slight change to the flood extents has had no additional impact to properties located within the AFA.

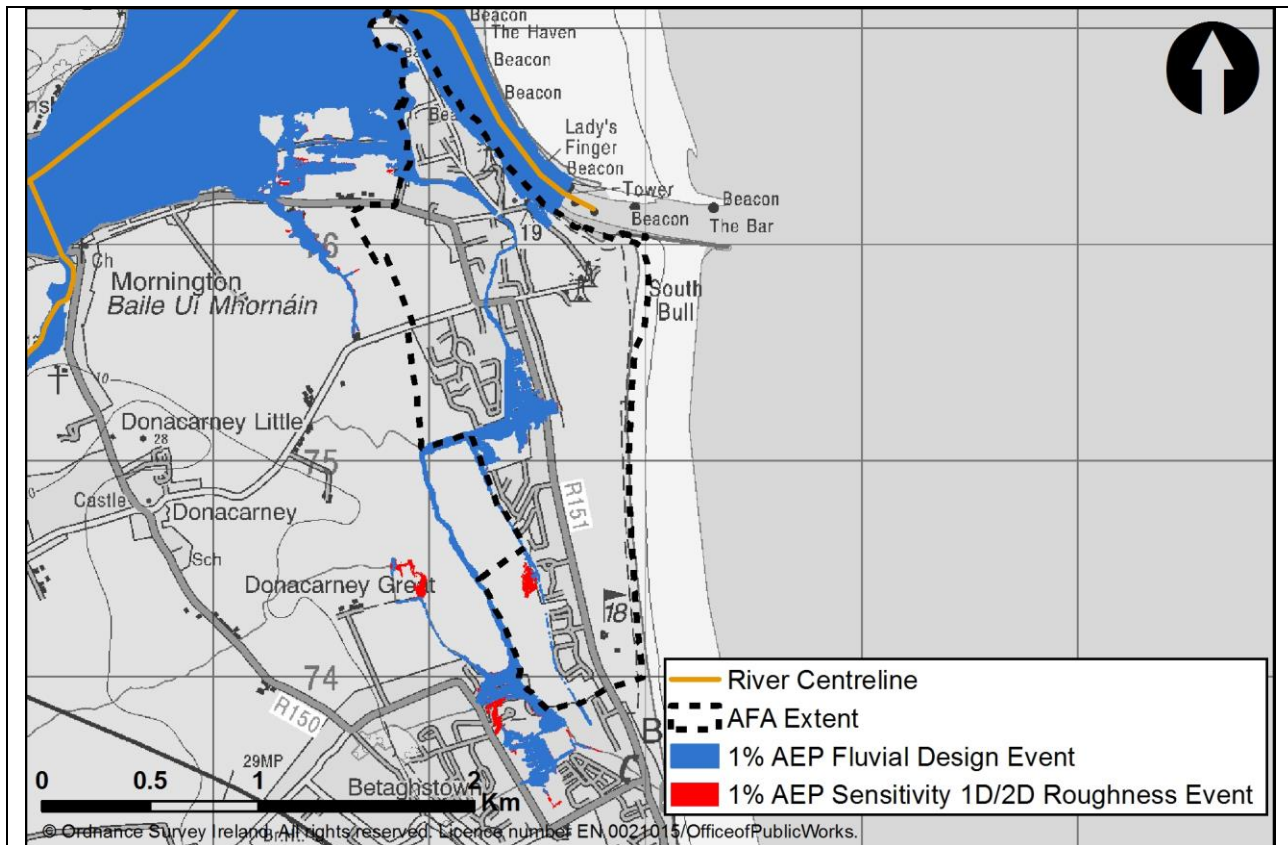


Figure 4.7.13: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in 1D/2D Roughness

- b) Downstream boundary increase – The downstream coastal boundary located at the Boyne Estuary was increased to the water level generated from the 0.5% AEP mid-range future design scenario (peak water level 4 m OD). Changing the boundary condition increases the peak water levels at the downstream extent of the model. Figure 4.7.14 shows that the model at Mornington demonstrates a moderate sensitivity to increasing the downstream boundary. This change has no further impact upon receptors as no additional properties are affected. There is an increase of the flood extents within an area of Mornington that remains undeveloped, close to the Mornington Estate.

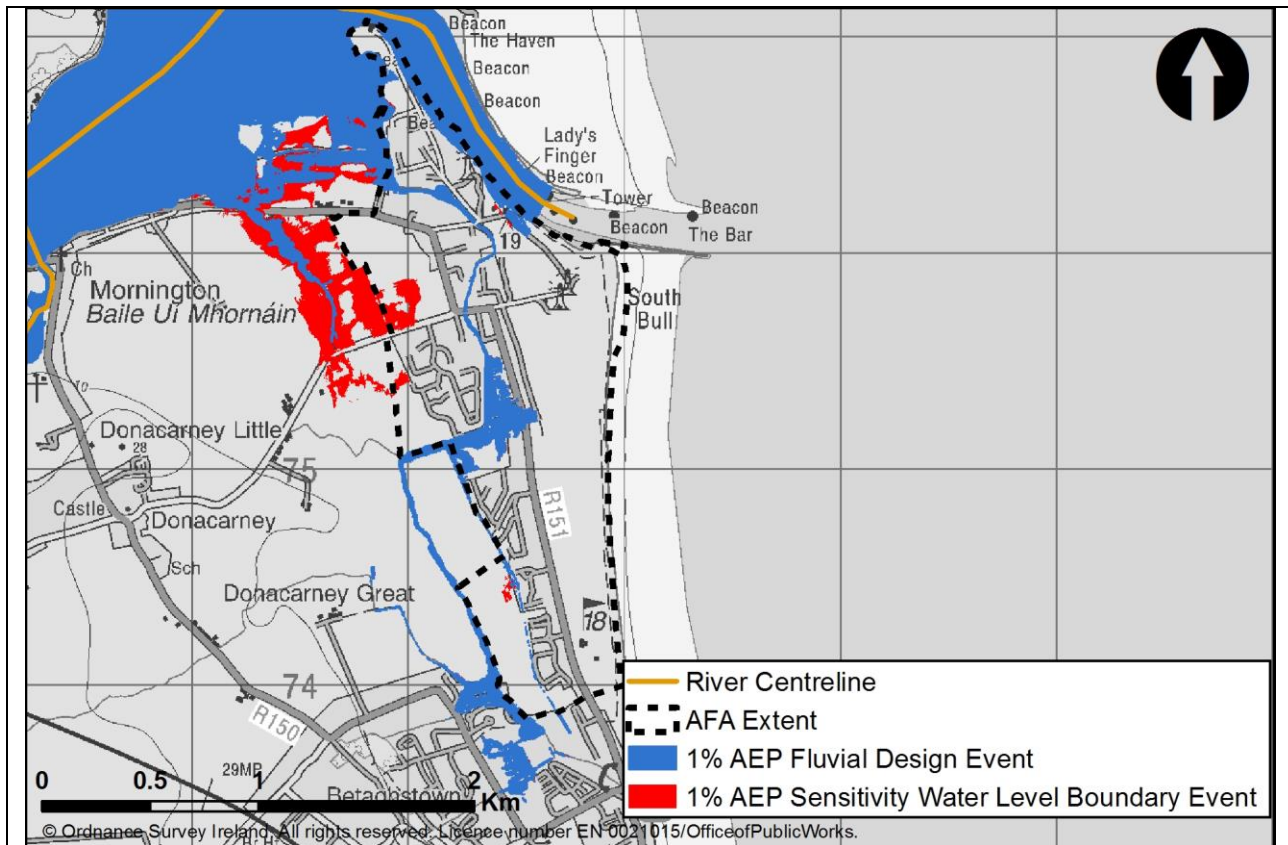


Figure 4.7.14: Comparison between 1% AEP Design Event & 1% AEP Sensitivity Water Level Boundary Event

- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Mornington model is assessed as having low uncertainty/sensitivity (refer to the Hydrology Report IBE0600Rp0012 for further detail); a factor of 1.68 is applied to design flows for the sensitivity simulation. Figure 4.7.18 shows that the Mornington model has a high sensitivity to increased inflow parameters; this is reflected by the increase of flood extent. Significant increases in flood extents can be found throughout the AFA. Residential areas affected include Foxbury, Fairways and the Mornington Estate. These changes have a high impact on receptors as approximately 154 additional buildings are affected. This is a relative 376% increase when compared to the 1% AEP design results.

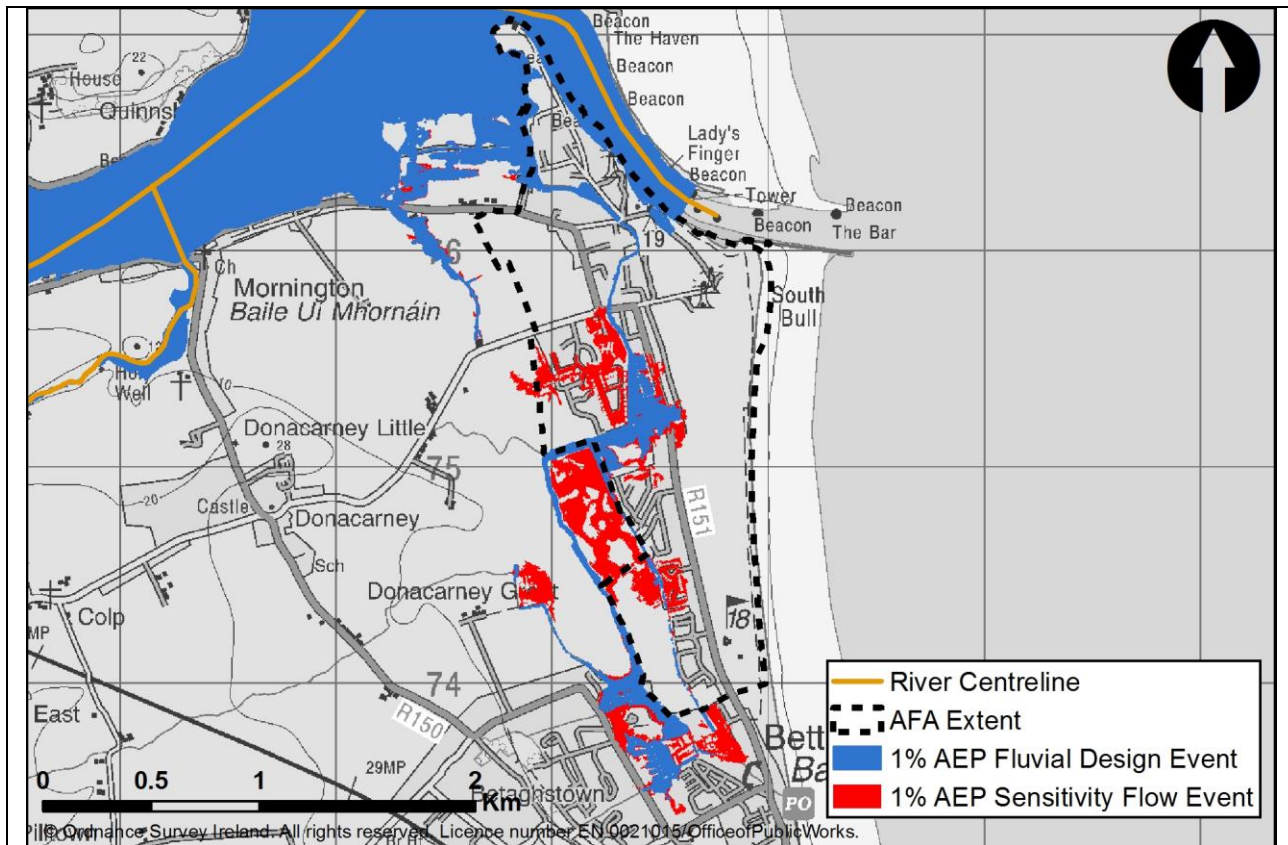


Figure 4.7.15: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event

- d) Floodplain roughness decreased to lower bound values – The change in floodplain roughness values resulted in no alteration to flood extents within the AFA as shown in Figure 4.7.16. The Mornington model is considered to have a low sensitivity to 2D roughness parameters. When compared to the design event no additional receptors are impacted within the AFA.

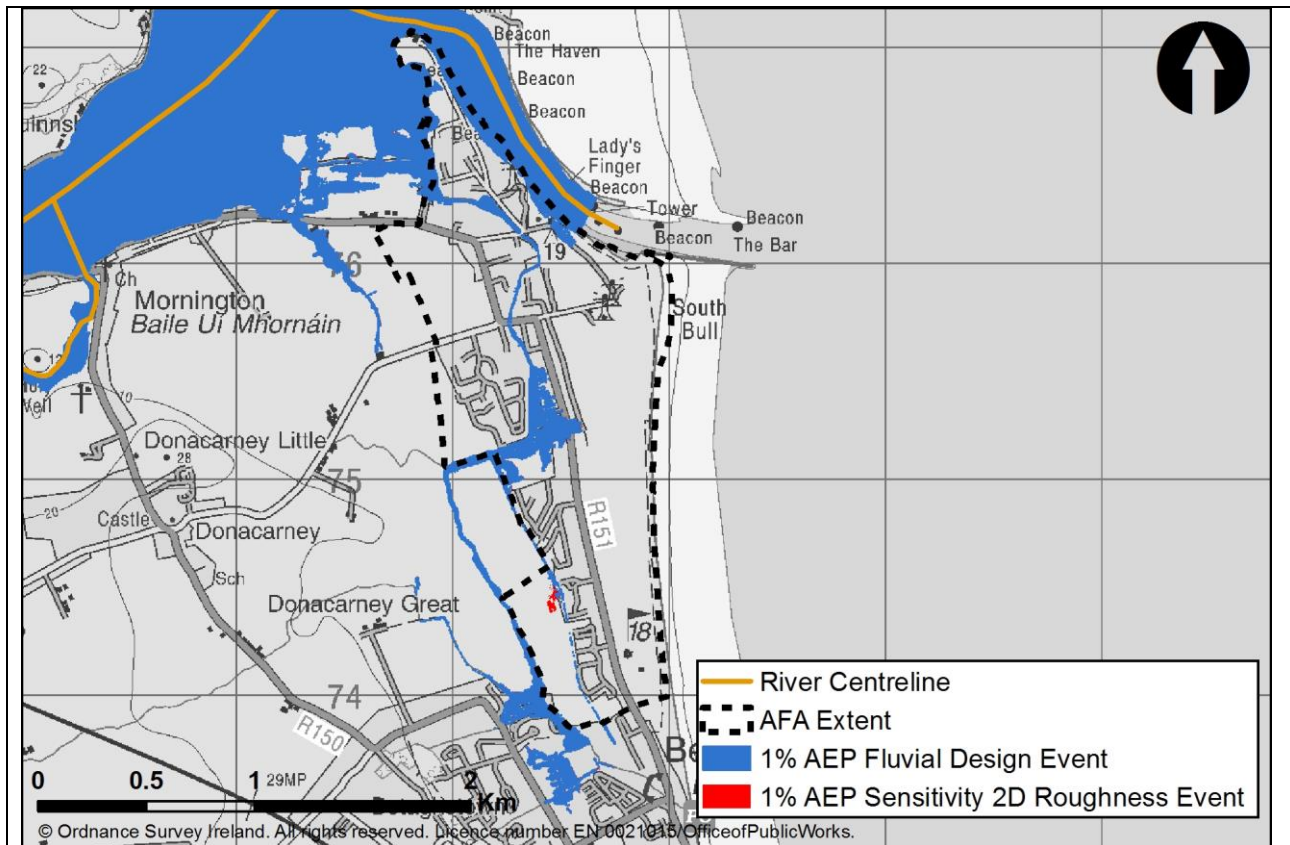


Figure 4.7.16: Comparison between 1% AEP Design Event and 1% AEP Sensitivity 2D Roughness Event

Table 4.7.3 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Mornington area of the Drogheda / Mornington / Baltray model. Of the parameters assessed, the model demonstrates a high sensitivity to inflow. Table 8.1 in the Hydrology Report states that there is low uncertainty associated with the hydrological inputs for this model, principally due to the availability of good quality gauge information. The model demonstrates lesser sensitivity to the other parameters, with the resulting analysis identifying low to moderate increases in flood extents and low impact to receptors within the AFA

Table 4.7.3: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	Low	-
1% AEP Sensitivity Water Level Boundary Event	Moderate	-
1% AEP Sensitivity Flow Event	High	High
1% AEP Sensitivity 2D Roughness Event	Low	Low

4.7.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods	
(a) OCT 2011.	<p>Flood event data collected as part of Eastern CFRAM Study flood event response shows that the Northlands housing development was inundated on the night of the 24th and 25th October 2011 by the Mornington River. Maximum flood depths of 0.36m were recorded for the flood event and a flood extent outline was recorded.</p> <p>There is no hydrometric gauge data in or in the vicinity of the Mornington River catchment which could be considered representative in terms of catchment response. There are no hourly rain gauges within the catchment with the nearest rain gauge for which data is available located at Dublin Airport, approximately 30km to the south. A review of the hourly data at Dublin Airport found that 67mm fell over a nine hour period on afternoon and evening of the 24th October. When analysed against the FSU DDF model can be shown to be a rainfall frequency of 1% AEP. The next closest hourly gauge is at Killowen 42km to the north. At this station 40mm was found to have fallen over a 14 hour period which equates to a 50% AEP event. Although rainfall frequency does not exactly reflect fluvial flood frequency, given the catchment is a relatively small catchment it is considered the rainfall event is likely to have been of the critical duration for flooding in the catchment; based on an FSR estimate using catchment descriptors the critical duration rainfall event for the catchment was estimated to be 9 or 10 hours. As such it is estimated that the fluvial return period may have been anywhere between a 50% and a 1% AEP. Assigning a more precise AEP within this range is difficult given the lack of catchment specific data. There were a number of reports of flooding in and around Drogheda but the event centre can be shown meteorologically and based on the severity of the events to be in and around Dublin.</p> <p>Given the detail of the flood event data recorded in relation to the event at Northlands but the uncertainty surrounding the frequency of the event it is considered that the event can only provide partial validation of the model. The development levels are captured within the 2D zone through the LiDAR however the 1D model represents the channel as open through Northlands whereas there are two culverted portions under the roads and gardens at either end of the development. Nevertheless detailed outputs from the model were reviewed against the event data in an attempt to calibrate the model.</p> <p>A comparison between the recorded event flood extents at Northlands and the modelled extents is shown in Figure 4.7.17.</p>

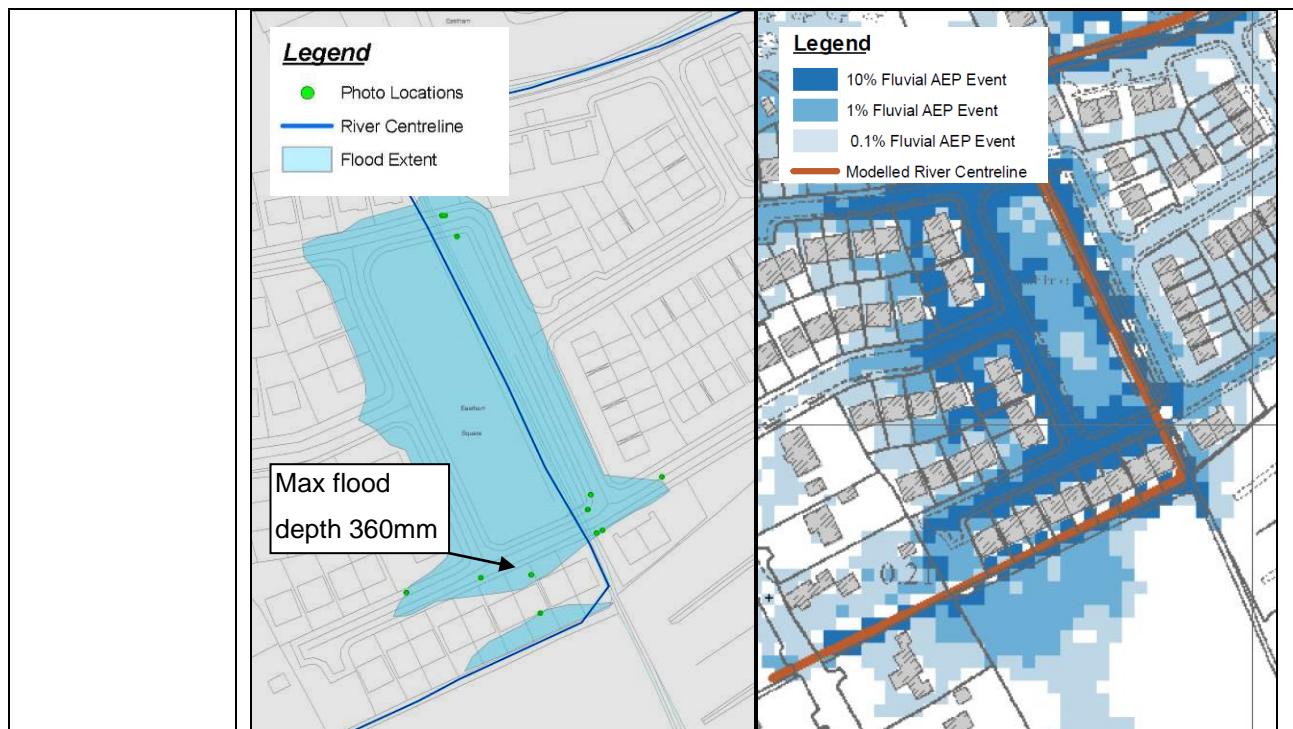


Figure 4.7.17: Comparison of Recorded Oct 2011 Flood Extent (left) and Modelled Flood Extents (right)

The maximum recorded flood depth of 360mm was recorded at the location shown in Figure 4.7.17 above. Maximum modelled flood depths for the design event scenarios are 360mm exactly for the 10% AEP and 537mm for the 1% AEP for the same location. Given the event is unlikely to have been as extreme as a 1% AEP event outside of Dublin, the fact the 10% AEP modelled and event recorded water depths are the same provides partial validation of the model. This agreement did not warrant further adjustment of the model parameters above those which were initially estimated based on observations, despite the fact the culverts are not represented as such within the model.

No further reports of flooding in the Mornington catchment were received for the event. This is consistent with modelled events of magnitudes up to 1% AEP which indicate no properties flooding within the AFA.

(b) FEB 2002.

The historical data indicated that flooding occurred in Drogheda, Baltray, Edenderry and Mornington in February 2002. This event was caused by heavy rainfall causing the River Boyne to overtop its banks, with high tides and strong winds adding to the flooding in Drogheda, Baltray and Mornington.

In Mornington, the tide level reached 3.36mOD. Analysis of tidal records in a report entitled "Mornington District Surface Water and Flood Protection Scheme" showed that the flood event in Mornington had an AEP of approximately 1%, while the corresponding river flow was found to be less than Q_{med} (approaching 100% AEP, 1 year return period). Existing defence embankments downstream of Lady's Finger were

	<p>overtopped by a margin of approximately 0.25m during the February 2002 event. Areas including Crook Lane, Tower Road, St. Mary's and Seaview Park were inundated and approximately 30 houses were flooded in these areas, with water depths of up to 0.6m. Up to 80 additional houses were protected with sandbags. In order to prevent further flooding from the river backing up during the subsequent high tide, units of the fire brigade and the civil defence were deployed in over-pumping the Mornington River past tidal defences.</p> <p>A flood alleviation scheme has been carried out in Mornington. The model shows that during a 1% event in the River Boyne and Mornington River, no out of bank flooding is occurring at Lady's Finger or in Mornington and Bettystown. Given the changes in this location the historical data cannot be used to calibrate the defended model however this data was used to calibrate the existing model upon which the defended model was built to a tolerance of 150mm (see "Mornington District Surface Water and Flood Protection Scheme", KMM, January 2004).</p>
(c) NOV 2000.	<p>Information was found for a flood event which occurred in Drogheda, Navan, Trim, Baltray, Edenderry and Mornington in November 2000. Floods were caused by heavy rain and storm force winds.</p> <p>In Mornington, it was estimated in a report by Kirk McClure Morton ("Mornington District Surface Water and Flood Protection Scheme", January 2004) that a 5% AEP flow occurred in the Mornington River, with an estimated discharge during this event of $2.98\text{m}^3/\text{s}$, while the tide level was within the normal tidal range (the estimated tidal level on this date was in the region of 1.6-1.7mOD Malin at River Boyne - Bar, where a mean high water spring tide is 1.84mOD Malin). The worst affected area was low lying agricultural land adjacent to the Golf Links Road on the main channel of the Mornington River. Further areas, included Fairway Lawns and the Caravan Park north of Bettystown, were at risk due to flooding of tributaries. Given the changes in this location the historical data cannot be used to calibrate the defended model however this data was used to validate the existing (undefended) model upon which the defended model was built (see "Mornington District Surface Water and Flood Protection Scheme", KMM, January 2004). Differences between in the predicted water levels ranged from 20mm lower to up to 140mm higher at the location of Harry's Shop.</p> <p>All of the areas discussed as having flooded directly from the Mornington River are now shown to be within the benefitting area following the new analysis (up to the 1% AEP event). Fairways Lawns adjacent to the tributary c2_1 and which remains undefended following the flood protection scheme constructed in 2010 is still shown to be at slight risk (gardens to the rear of two properties) from the 1% and 10% AEP event. The caravan park at Bettystown is not shown to be at risk however the recent development at Northlands adjacent to the caravan park is and this may indicate that topographical changes as a result of the Northlands development may have changed</p>

	the flood extents in this area. In light of this the defended area and the 10% and 1% AEP extents can be shown to be broadly validated by the historical evidence in relation to the event.
(d) DEC 1978.	Information was found for a flood event which occurred in Drogheda, Navan, Trim, Mornington and Baltray in December 1978. The flooding transpired after a period of heavy rainfall on the Boyne catchment, which approximated to the 50% AEP 2 day rainfall for the area. High tides/low barometric pressure also contributed to the flooding in Drogheda, Mornington and Baltray. The only details available regarding this event in these AFAs is that the Boyne Mill clothing factory in Drogheda was flooded, with the water level reaching doorsteps in Baltray and Mornington. There is insufficient detail from the event to provide validation of the latest mapped extents.
(e) DEC 1954.	A flood event was found to have occurred in Drogheda, Mornington, Navan and Trim in December of 1954. However, the press article on http://www.floodmaps.ie in which the flooding in Drogheda and Mornington was reported was not available for download, and no further details on this flood event in Drogheda and Mornington could be found. . There is insufficient detail from the event to provide validation of the latest mapped extents.

Summary of Calibration

Some historical information is available for the Mornington area which showed the Mornington River to be a source of flood risk to the surrounding area historically. A flood alleviation scheme has been carried out since this historical flooding to address these flood risk issues and the pre flood protection scheme model upon which the Study model has been built was shown to be calibrated / validated to two of these events, November 2000 and February 2002. The other two historic events, December 1978 and December 1954, have insufficient detail for calibration or verification of the model. Flooding occurred post flood protection works on the evening of the 24th / 25th October 2011 in the Mornington catchment and although it is difficult to put a definite frequency on the event, the resolution of the flood event data available is high. This data partially validates the upper reaches of the model but long term hydrometric or rainfall data is not available within the catchment such that the frequency of the event can be estimated with the certainty needed upon which to base model calibration.

A comparison between check flows at HEPs and modelled flows has indicated that returning peak flows are well anchored to the hydrological estimates for the 10% and 1% AEP fluvial events. In the 0.1% AEP event there are a number of locations where there is a large discrepancy. However given this is the event at which the standard of protection of the defences is exceeded and there is a large amount of floodplain flow and interaction between individual catchment flood flow routes, this is not considered a reflection of the model being poorly anchored to the hydrological estimates.

There are no significant water level instabilities within the Mornington River or lower Boyne Estuary reaches of the model. It was noted in the 0.1% AEP fluvial model however that in-channel and out of bank flow hydrographs at the lower reaches of the tributaries c2_3 and c2_4 appear quite unstable. A review of

the water level hydrograph found that the shape is smooth and it is considered that the instability in the flow hydrograph is due to the large cross bank flows from the Mornington River across the bottom reaches of both of these tributaries. Combining in-bank and out of bank flow hydrographs results in a partly smoother hydrograph and it is thought that instability may relate to the model being limited to transfer of flow across the bank in one direction at a time per time step. This is not considered to have a significant impact on uncertainty of peak flood levels or extents and is fairly typical of reaches with a large cross bank flow.

(2) Post Public Consultation Updates:

Following comments received during informal public consultation and formal S.I. public consultation periods in 2015, some consultees indicated that flooding occurred in the residential street named St. Nicholas Village / The Dunes. Flooding was said to have occurred within the street and came very close to entering properties in November 2014. Flood waters had to be pumped out of the area even after flood levels in the Mornington River had dropped. A photo showing the flooding at the end of 'The Dunes' cul-de-sac was provided shortly after the informal public consultation event as shown in Figure 4.7.18, below.



Figure 4.7.18: Flood Event of November 2014 at The Dunes

As discussed in relation to model calibration there is very little observed data upon which to estimate the frequency of flood events in the Mornington River. Consultees at the informal public consultation did however indicate that there is an ongoing flood risk in the area and that it may be partly pluvial in nature.

Consultees indicated that the flooding arose from rising water levels in the area rather than overtopping of the embankment along the Mornington River adjacent to the Dunes.

RPS reviewed the model in light of the information provided in relation to the November 2014 flood event. It was found that the drain referred to in this report as tributary C2_1 has the potential to flood St. Nicholas Village / The Dunes depending on the flows which are entered into the model and that there is a high degree of uncertainty in relation to the design flows within this tributary given its small catchment size (less than 0.5km²). The design inflows entered into this tributary were reduced at draft stage as there was no known history of this significant flood mechanism. RPS reverted to FSU based estimates of flood flow for this tributary in light of the information provided by consultees and found that flood waters build up in the model at the confluence of the tributary and the Mornington River as the flap valve outfall to the Mornington River is shut due to high water levels. Flood flows build up around the confluence spreading along the rear of the Mornington River flood defences eventually leading to flooding in The Dunes.

These changes resulted in increased flooding from the tributary as shown below in Figure 4.7.19. The 1% AEP event flood extents in the updated maps spread eastwards from the Tributary C2_1 before filling the area at St. Nicholas Village / The Dunes. Residential properties are effected in this area in the 5% AEP event and greater. This is considered consistent with the responses from consultees. The model was updated and check flows recalculated with a revised set of flood hazard and risk mapping issued as Final to reflect this change. Flood depths on the road at the location shown in Figure 4.7.18 range from approximately 300mm in the modelled 5% AEP event to 400mm in the 1% AEP event.

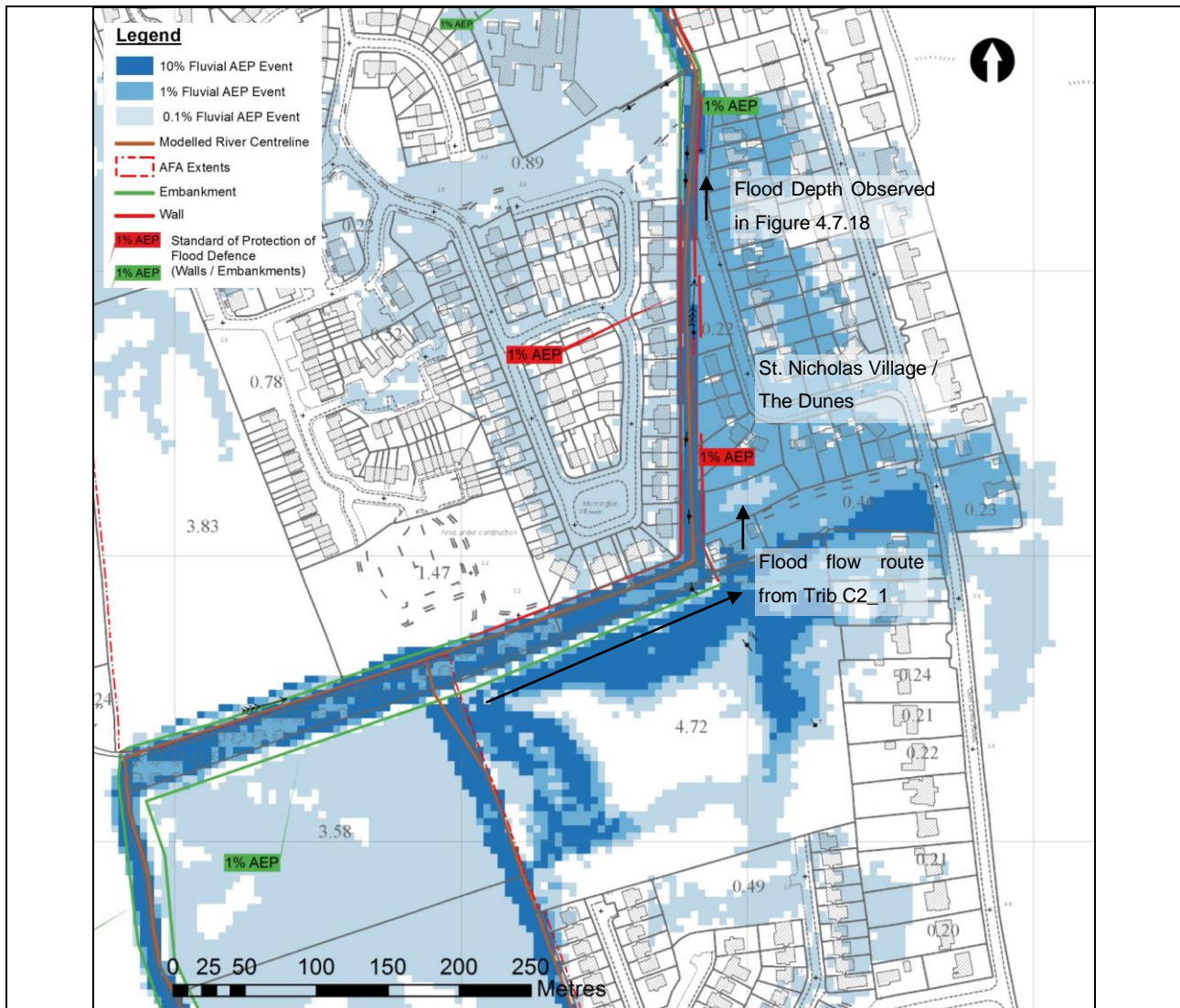


Figure 4.7.19: Revised Flood Extents Following Public Consultation

(3) Standard of Protection of Existing Formal Defences:

Defence Ref.	Type	Watercourse	Bank	Length / Diameter	Designed Standard of Protection (AEP) (Fluvial)
1	Earthen Embankment	C2 (Mornington River)	RIGHT	1943m	1%
2	R.C. Wall	C2 (Mornington River)	RIGHT	343m	1%
3	Earthen Embankment	C2 (Mornington River)	RIGHT	29m	1%
4	R.C. Wall	C2 (Mornington River)	RIGHT	83m	1%
5	Earthen Embankment	C2 (Mornington River)	RIGHT	111m	1%
6	R.C. Wall	C2 (Mornington River)	RIGHT	239m	1%
7	Earthen Embankment	C2 (Mornington River)	RIGHT	110m	1%
8	Earthen Embankment	C2 (Mornington River)	RIGHT	44m	1%
9	Earthen	C2 (Mornington River)	RIGHT	556m	1%

	Embankment				
10	Earthen Embankment	C2 (Mornington River)	LEFT	623m	1%
11	R.C. Wall	C2 (Mornington River)	LEFT	406m	1%
12	Earthen Embankment	C2 (Mornington River)	LEFT	305m	1%
13	Earthen Embankment	C2 (Mornington River)	LEFT	244m	1%
14	R.C. Wall	C2 (Mornington River)	LEFT	103m	1%
15	R.C. Wall	C2 (Mornington River)	LEFT	48m	1%
16	Earthen Embankment	C2 (Mornington River)	LEFT	427m	1%
17	Earthen Embankment	C2 (Mornington River)	Perpendicular to watercourse	110m	1%
18	Plastic Flap Valve	C2_1 (Mornington Tributary)	N/A	900mmØ	N/A
19	Plastic Flap Valve	C2_2 (Mornington Tributary)	N/A	450mmØ	N/A
20	Cast Iron Flap Valve	C2 (Mornington River)	N/A	3 x 900mmØ	N/A
The defences listed above are shown in Figure 4.7.20.					

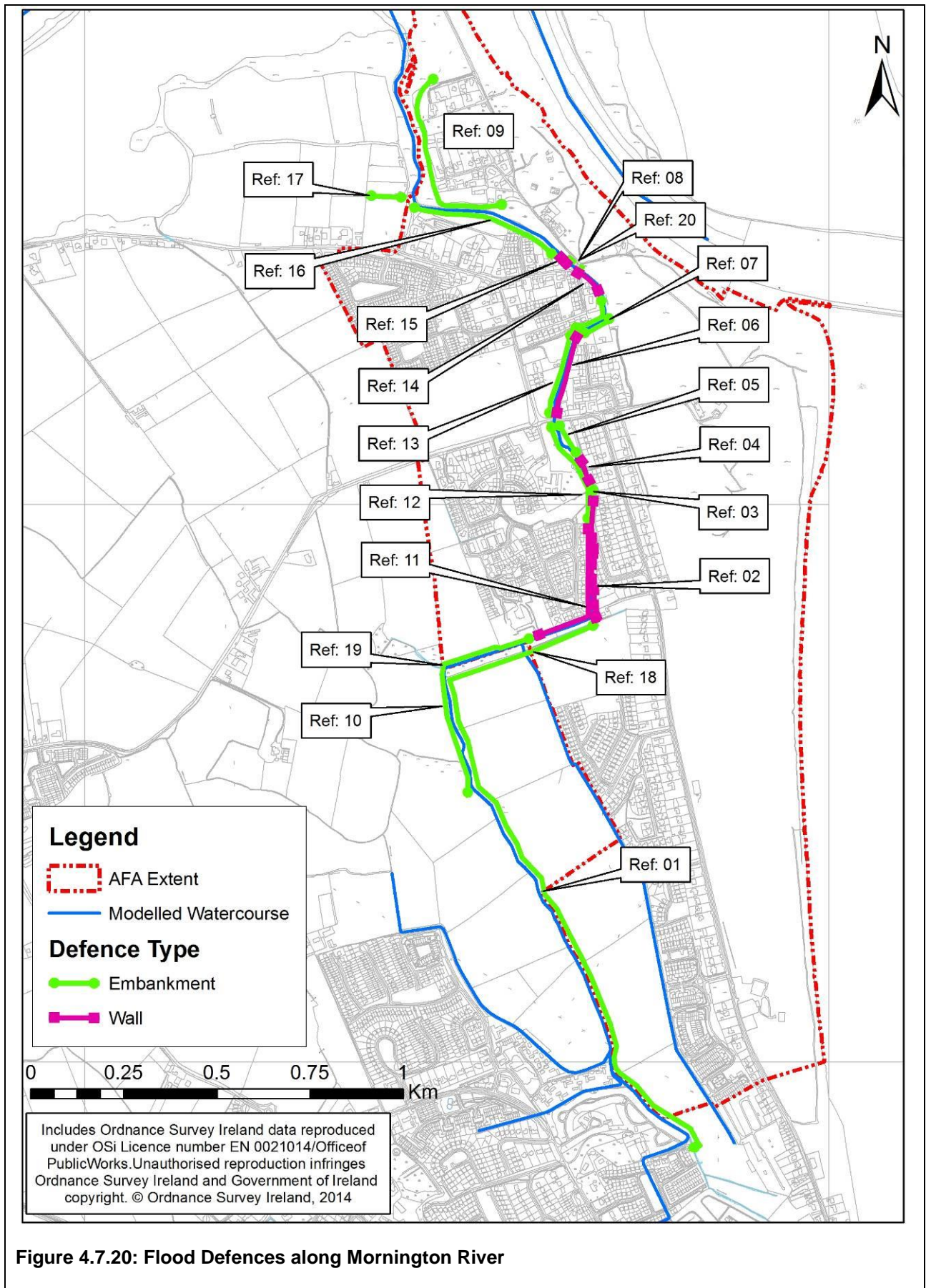




Figure 4.7.21: Defence Reference:15 (Lady's Finger)



Figure 4.7.22: Defence Reference:06 (adjacent to Harry's Shop)

(4) Gauging Stations:

There are no gauging stations on the modelled reaches.

(5) Comparison of Flows:

Table 4.7.4 provides a comparison between modelled flows and the hydrologically derived flows for the tributary check and downstream check point HEP flows. At the top of the Mornington River (c2_US) the upstream inflow has been compared to the modelled flow entering the AFA extents.

Table 4.7.4: Modelled Flows and Check Flows

River Name & HEP	AEP	Peak Water Flows		
		Check Flow (m ³ /s)	Model Flow (m ³ /s)	Diff (%)
Mornington River c2_US	10%	2.15	2.18	1%
	1%	3.99	3.82	-4%
	0.1%	7.14	6.49	-9%
Mornington River Trib c2_1	10%	0.17	0.29	72%
	1%	0.32	0.60	88%
	0.1%	0.58	1.59	174%
Mornington River Trib c2_3	10%	1.26	1.24	-2%
	1%	2.33	2.5	7%
	0.1%	4.17	3.9	-6%
Mornington River Trib c2_4	10%	0.11	0.11	0%
	1%	0.2	0.25	25%
	0.1%	0.36	1.37	281%
Mornington River c2_DS	10%	3.77	3.75	-1%
	1%	6.99	7.11	2%
	0.1%	12.5	7.47	-40%

It can be seen from the table above that for the 10% AEP events the model is very well anchored to the hydrological estimates at all locations where the difference is less than 2% at all locations apart from at the downstream extent of tributary c2_1 where it is as much as 72%. At this location the modelled flow represents the flow through the flap valve at the downstream extent of this tributary. A review of the modelled flood shows that when the inflow to this reach peaks there is zero flow through the flap valve as it is closed during high water levels in the Mornington River at the downstream side of the valve. The peak flows observed in the table represent the draining out of the flood water which has built-up on this tributary (c2_1), once the water levels have dropped in the Mornington River. The differences are greater in the 1% and 0.1% AEP events (88% to 174%) however this is not considered a like for like comparison as the modelled flows are not hydrologically driven but rather are driven by specific hydraulic mechanisms.

In the other modelled watercourses, for the 1% AEP event the modelled flows are generally well anchored to the hydrological estimates with the only significant discrepancy (above 10%) observed at the downstream extent of the tributary c2_4. Flood extents in the model at this location are out of bank in both the Mornington River and the tributary and the increase in flow is due to flow from the Mornington River flowing out of bank and into the tributary channel at the HEP.

For the 0.1% AEP there are three locations where there are significant discrepancies between the

modelled and hydrologically derived flow estimates. At this return period the standard of protection of the defences is surpassed and there are significant overland flow paths. At c2_4 the modelled flow is much larger than the hydrological estimate but this time it is almost three times as much. This is for the reason stated – flow leaving the Mornington River and entering the tributary channel at the HEP – and would not indicate a discrepancy to be investigated further. At the downstream extent of the tributary c2_1 the modelled flow through the flap valve at the HEP is over double the hydrological estimate. This is partly due to the draining out of this tributary once the flap valve can open as discussed previously. However in the 0.1% AEP event the Mornington River overtops the earthen embankments and flow enters the tributary c2_1 before exiting again at the HEP. At the downstream extent of the Mornington River at the HEP c2_DS the flows are 40% lower in the modelled event than the hydrological estimates. Flows along this portion of the model are difficult to gauge as there is a significant back water effect from the Boyne Estuary. As such flows have been extracted further up the channel than the HEP. Within the flow hydrograph however, even at Lady's Finger, the Boyne Estuary fluvial and tidal backflow effect is still evident and there is at least a portion of the discrepancy due to the backwater effect reducing the flow in the channel. There is also a significant portion of flow lost to the floodplain in the 0.1% AEP event which will not be evident in the model at the downstream HEP. Given these factors which are likely to be contributing to the reduced model flow and the fact that the model is shown to be well anchored in the higher frequency events its is considered that the reduced peak flow is accounted for.

(6) Other Information:

The following reports were received from Meath County Council:

- Flood Study for Bettystown, Northlands Flood Alleviation November 2013 (Punch Consulting Engineers)
- Flood Study for Bettystown, Northlands Housing Estate June 2013 (Punch Consulting Engineers)

4.7.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

- (a) Please refer to Section 3.4 for general assumptions using the Infoworks ICM modelling software.
- (b) The existing formal defences are effective and therefore included in the model.
- (c) It is assumed that informal defences are ineffective and are therefore not included in the model.
- (d) The bridge located in the Mornington River at chainage 3423m (Lady's Finger) starts as a triple arch but changes to three 0.9m diameter concrete flapped pipes. This bridge was represented in the model by three 0.9m diameter pipes as this was established as the critical geometry dictating the head loss through the structure. Flap valves have been included in model (model will not allow reverse flow) with headloss co-efficient 1.0, representative of disc type flap valve.
- (e) The mass error balance report showed a difference of 0.006%. This is within the acceptable limits as stated in the Environment Agency's Fluvial Design Guide.

- (f) Cross sectional data within the existing 'Mornington Flood Alleviation and Drainage Study' is assumed to be an accurate reflection of the existing channel section up to the toe of embankment level.
- (g) It is assumed that the 'as constructed' drawings used to determine the bank line and level post flood defence construction are accurate for the purposes of this Study.
- (h) The survey data used to produce the 1D portion of the model pre-dates the development of the Northlands Housing Estate. However LiDAR captures estate levels and is included within 2D zone. The model was found to be in good agreement with data recorded for the October 2011 flood event.
- (i) While the tributaries within the model have been constructed using surveyed cross sections, any structures along the tributaries have not been included in the model. Note that these tributaries represent catchments less than 1km².
- (j) There is instability at the lower reaches of the tributaries c2_3 and c2_4. This is detailed under Summary of Calibration, Section 4.7.5.

(2) Hydraulic Model Limitations and Parameters:

- (a) No drainage networks have been included in the model; as such flows have been introduced directly to the 1D domain as point inflows as determined in the hydrological analysis.
- (b) A model timestep of 1 second has been applied to the model in order to achieve a stable run.
- (c) Flood defence 'as constructed' drawings have been used to define the representation of the flood defences within the model. The level of the defences shown on the 'as constructed' drawings is considered to be fairly accurate and compares well to LiDAR where defences are captured within the LiDAR. However the position of embankments (perpendicular distance from watercourse) as shown on the 'as constructed' drawings differs from the position apparent from the LiDAR. Model bank lines have been set based on the levels recorded on the 'as constructed' drawings but the line on plan has been set based on the line apparent from the LiDAR.
- (d) Bank discharge coefficients of 0.8 have been used as there is uncertainty over the condition of the embankments. This has a minimal effect on water depth at the spill line of the embankment. It is a simplified conservative approach but is considered appropriate for this study.

Hydraulic Model Parameters:

1D Domain

Timestep (seconds)	1
Min / Max space step	0.1m / 25m (Default: 10m / 50m) Value reduced changed to aid model stability.
Max Timestep Halvings	30 (Default: 7)

	Value reduced changed to aid model stability.
Max Iterations	30
2D Domain	
Timestep (seconds)	Dynamic
Timestep Stability Control	0.95
Maximum Velocity	10m/s
Theta	0.9
Inundation Mapping Depth Threshold	0.01m
(3) Design Event Runs & Hydraulic Model Handover Notes:	
<p>The existing Mornington flood alleviation scheme performs well during the 10% and 1% AEP flood events in relation to the AFA with flood flows generally constrained between the flood embankments / walls within the AFA. At one location within the AFA however the flood defences would be overtopped and flooding is apparent during the 1% AEP fluvial event. At the downstream extent of the tributary c2_1 flood waters exit the right bank and flood the area of agricultural land behind the earthen embankments on the Mornington River on the opposite bank from Mornington Towers. This flood extent progresses around the corner eventually affecting a large number of properties at The Dunes / St. Nicholas Village (see 0 (2)) for further details.</p> <p>At draft stage the flood extent maps were showing flooding to the area of Ozanam House from the Mornington River (located off Garra Road), between Coney Hall and the Mornington River). Following review at Draft stage it was found that the spill location in the model was not accurately representing the top of defence level as per the as built drawings. This was amended following discussions with OPW and the final flood extent maps no longer show flooding to this property in the 1% AEP event.</p> <p>In addition to the two areas identified within the AFA there is a large area at the upstream extent of the Mornington River at Northlands in Bettystown which is shown to be at risk in all fluvial events. This area is not within the AFA and was not developed at the time of the "Mornington District Surface Water & Flood Protection Scheme" (2004). As such the topography of the development is captured within the model via the recently captured LiDAR which has been used to construct the 2D extents however there is uncertainty in the 1D channel extents. This area is discussed in further detail in relation to the October 2011 flood event in Section 4.7.4 (1).</p> <p>In the 0.1% AEP fluvial flood event large portions of the AFA are inundated as the flood defences and banks along the Mornington River and its tributaries are breached at multiple locations. It is estimated that over 200 properties within the Mornington AFA are at risk from the 0.1% AEP fluvial event.</p> <p>At the downstream reach of the Mornington River a large flood extent can be seen in the salt marshes at</p>	

all three design fluvial return periods. This is caused by back flow from the River Boyne; it is however prevented from reaching properties within Mornington village by the embankment at chainage 3918m, even during a 0.1%AEP event.

The Mornington River and its tributaries have reaches which have very shallow gradients and therefore with the influence of the tidal River Boyne a reversal in flow direction can be seen up to Lady's Finger.

During all coastally dominated flood events there is no significant flooding evident from the Mornington River upstream of the Lady's Finger non return valves. During the 10% AEP event there are no properties within the AFA affected by flooding as they are protected by the defence embankments along the lower reaches of the Mornington River. The dominant coastal flood mechanism in relation to the Mornington AFA is from the Boyne Estuary along the right bank for up to 1.5km upstream of the Mornington River confluence. Even during the 10% AEP flooding is shown to breach the R151 to the west of Mornington Village. For the 0.5% and 0.1% AEP coastal events large swathes of agricultural land are shown flooded to the west of Mornington Village. One property within the AFA is shown flooded during the 0.5% AEP event however in the 0.1% AEP event this increases to over 100 properties. During the 0.1% AEP event flood depths of up to 1.5m are predicted by the model in the agricultural land at the western edge of the AFA. Within developed areas flood depths of up to 1m are predicted by the model, particularly along the areas within the village served of the R151.

Mornington is not considered to be at risk from coastal flooding directly from the coastline as a continuous level of at least 4m is provided, largely by the sand dunes, along the coastline.

Model run log files show instances of "Warning : 547" regarding reverse flow. This warning relates to flow back through bank lines and is typical of where tributary reaches meet main channel on flat reaches (such as where tributary c2_3 and c2_4 meet the Mornington River) and main channel flow runs over the lower reaches of the tributary. This has no impact on model results.

(4) Hydraulic Model Deliverables:

Model deliverables are supplied in an accompanying InfoWorks ICM transportable database containing all model files as required by the brief and the relevant network and event files.

(5) Quality Assurance:

Model Constructed by:	Frances Carragher
Model Reviewed by:	Stephen Patterson
Model Approved by:	Andrew Jackson

4 HYDRAULIC MODEL DETAILS

4.8 NAVAN MODEL

Name	Local Authority	Unique ID	SSA	Status	Date
Navan	Meath	70039	AFA	Final	08/05/2017

4.8.1 General Hydraulic Model Information

(1) Introduction:

The Eastern CFRAM Flood Risk Review (IBE0600 Rp0001_Flood Risk Review_F02) highlighted Navan as an Area for Further Assessment for fluvial flooding. This was based on a review of historic flooding and the extents of flood risk determined during the PFRA.

The Navan model includes reaches of the Boyne, a number of large tributary inflows and also a large number of smaller tributaries in the vicinity of the AFA and as such is a relatively large and complex model. The main tributary inflow is from the River Blackwater, which has a small tributary (Abbeylands). Moving from upstream to downstream, the other tributaries to the River Boyne are the Athlumney House Tributary (which has three minor tributaries), Old Balreask Tributary, Trim Road Tributary (also known as the Swan River), Athlumney Tributary and Bailis Tributary (Figure 4.8.1). The Trim model is located immediately upstream (on the River Boyne) with this model connected to the Drogheda model (also on the River Boyne) at its downstream extent.

Four gauging stations are located within the model extents, with three on the main channel of the Boyne and one on the Blackwater (Kells) River. The gauge located at the upstream extent of this model is called Ballinter Bridge (07041 – EPA) and its rating has been given an FSU classification of A2. However, it has a relatively short record period (1997 – present) and as such there is fair confidence in its Q_{med} value of $161.0 \text{ m}^3/\text{s}$. The Navan Weir gauging station (07009 – OPW) is located on the Boyne in the centre of Navan, and has been given an A1 rating classification for the entire period of its rating from 1976. There is good confidence in the gauged Q_{med} value of $139.7 \text{ m}^3/\text{s}$. The other Boyne gauging station is located near to the confluence point with the Blackwater and is called Blackcastle (07037 – OPW). There is only water level data at this station and as such has no use in design flow estimation. The Blackwater gauge is located approximately 3km up from the confluence point with the Boyne on the lower reach of the Blackwater, on the outskirts of Navan and is called Liscartan (07010 – OPW). This gauging station has three classification periods under FSU, A1 pre 1972, A2 from 1972 – 1982 and A2 from 1982 to date. This would suggest that for all three periods the station should be reliable up to and above Q_{med} . The most recent Q_{med} value is $68.36 \text{ m}^3/\text{s}$.

Although the gauged flow data is generally of high quality, a NAM model has been developed at each gauging station with flow data available. Each NAM model was used to simulate an extended AMAX

series for the duration of the rainfall record and to fill in any gaps in the records which may have been missed.

The two flow gauges located on the main channel of the Boyne can be shown to have inconsistent Q_{med} values with the upstream gauging station (07041). This was found to have a higher observed Q_{med} value than the gauging station just downstream at Navan (07009). Closer inspection shows that this is largely due to the different record periods for each station. Consideration of more recent data only at the A1 gauging stations (07009 and at 07012) just downstream of the model extents results in higher Q_{med} values than the entire classified rating period which is likely due to the effect of arterial drainage within the Boyne catchment. Q_{med} values extracted from the NAM models were found to be consistent with the more recent (higher) periods of record (post arterial drainage). The observed (gauged) values are inconsistent, increasing and decreasing even as the catchment area increases, despite the gauges having confidence at Q_{med} . This is largely because the records look at different periods. The NAM models use the same parameters to represent the catchment throughout the simulation period and calibration is focussed on the recent record period. This results in a consistent (increasing) Q_{med} moving down the Boyne catchment and therefore rationalises the discrepancies encountered when using the observed values drawn from different record periods.

The Navan AFA is also affected by a number of ungauged tributaries of the Boyne. All of these watercourses are less than 10km² in catchment area and estimates of Q_{med} have been derived from catchment descriptor based methods. Where flows have not been derived from NAM modelling values, they have been derived based on the IH124 method for small catchments (< 25km²) or the FSU method; then adjusted against the calibrated NAM models where appropriate.

The River Boyne, and tributary watercourses are designated as HPW and are modelled as 1D-2D using the MIKE suite of software. The River Boyne beyond the upstream and downstream extents of the AFA is MPW and has been modelled as 1D again using the MIKE suite of software. Channel markers have been located at the right and left banks of all cross sections. Flow within these markers is calculated by the 1D model component; however when the water level rises sufficiently to meet the bank markers flow can enter the 2D domain which represents the floodplain, where the 1D model has been linked to the 2D model.

(2) Model Reference:		HA07_NAVN6
(3) AFAs included in the model:		Navan
(4) Primary Watercourses / Water Bodies (including local names):		
RIVER BOYNE OLD BALREASK TRIBUTARY - BOYNE ATHLUMNEY HOUSE TRIBUTARY - BOYNE	TRIM ROAD TRIBUTARY - BOYNE (SWAN RIVER) ATHLUMNEY TRIBUTARY - BOYNE BAILIS TRIBUTARY - BOYNE	RIVER BLACKWATER (KELLS) ABBAYLANDS TRIBUTARY - BLACKWATER (KELLS)

(5) Software Type (and version):

(a) 1D Domain: MIKE 11 (2011)	(b) 2D Domain: MIKE 21 - Rectangular Mesh (2011)	(c) Other model elements: MIKE FLOOD (2011)
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4.8.2 Hydraulic Model Schematisation

(1) Map of Model Extents:

Figure 4.8.1 illustrates the extent of the modelled catchment, river centre line, HEP locations and AFA extents as applicable. The Navan model contains 12 Upstream Limit HEPs, 1 Downstream Limit HEP and 1 Intermediate HEP. It also contains 3 Gauging Station HEPs and 14 Trib HEPs.

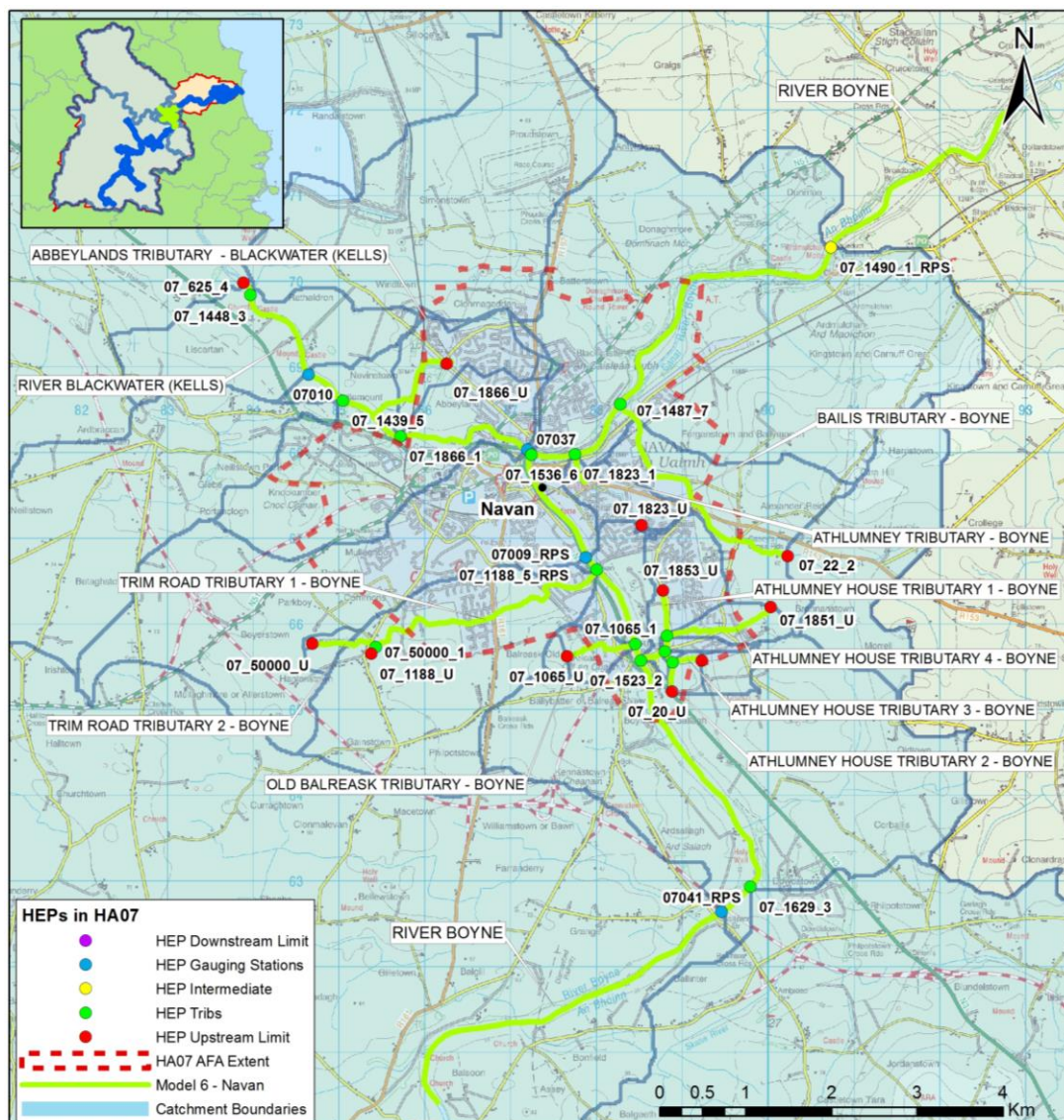


Figure 4.8.1: Location of HEPS and Modelled Rivers

(2) x-y Coordinates of River (Upstream extent):

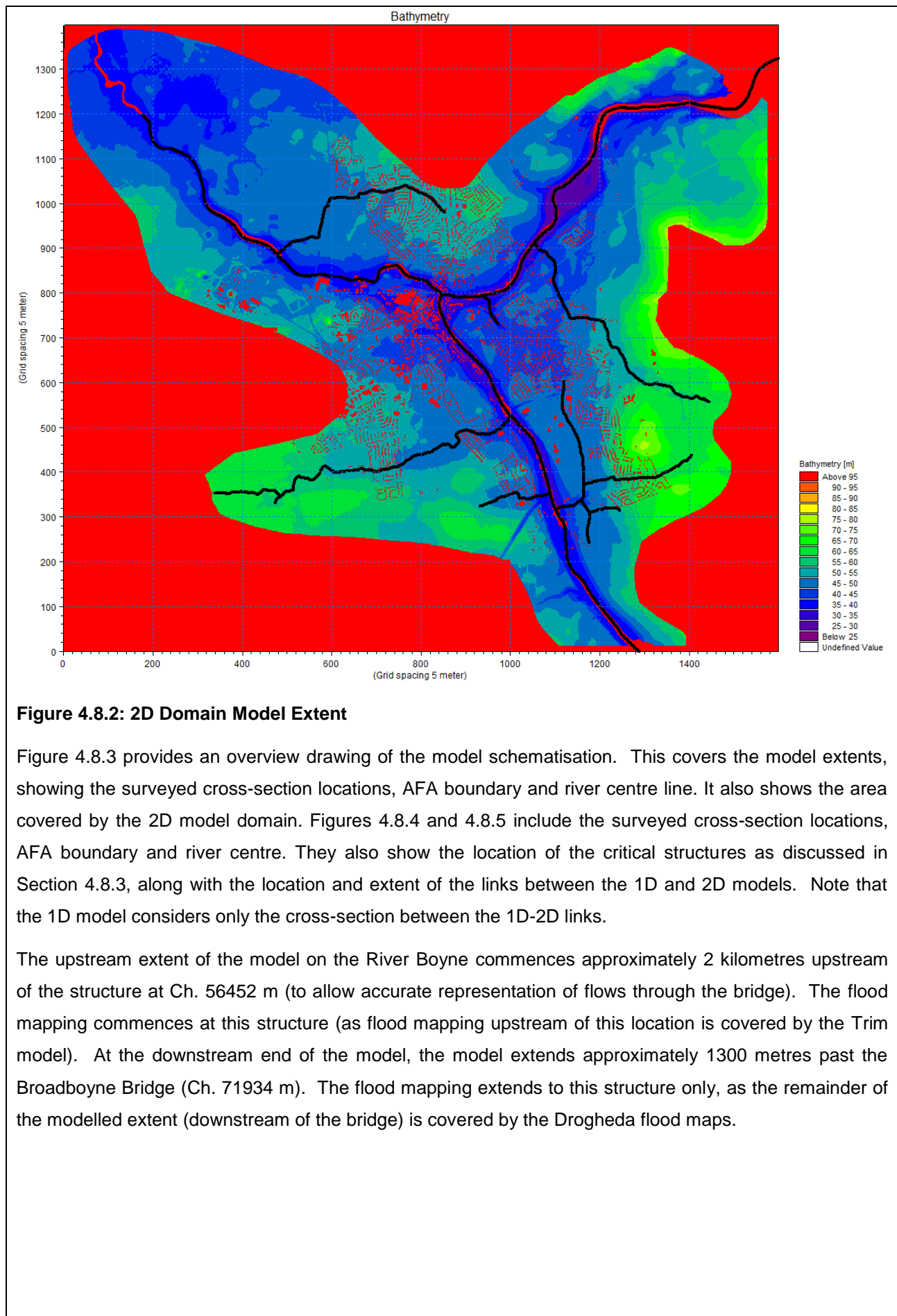
River Name	x	y
RIVER BOYNE	286145.81	260419.02
OLD BALREASK TRIBUTARY - BOYNE	287655.88	265620.49
ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	288764.85	266350.44
ATHLUMNEY HOUSE TRIBUTARY 2 - BOYNE	288879.88	265213.44
ATHLUMNEY HOUSE TRIBUTARY 3 - BOYNE	289224.68	265572.38
ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE	290027.09	266194.25
TRIM ROAD TRIBUTARY 1 - BOYNE	285370.69	265651.21
TRIM ROAD TRIBUTARY 2 - BOYNE	284670.08	265768.96
ATHLUMNEY TRIBUTARY - BOYNE	287863.08	267655.33
BAILIS TRIBUTARY - BOYNE	290223.83	266789.17
RIVER BLACKWATER (KELLS)	283881.29	269983.9
ABBAYLANDS TRIBUTARY - BLACKWATER (KELLS)	286307.77	269062.79

(3) Total Modelled Watercourse Length:

36.1 kilometres (approx.)

(4) 1D Domain only Watercourse Length:7.7 kms
(approx.)**(5) 1D-2D Domain
Watercourse Length:**28.4 kms
(approx.)**(6) 2D Domain Mesh Type / Resolution / Area:**Rectangular / 5 metres / 56 km²**(7) 2D Domain Model Extent:**

Figure 4.8.2 represents the general topography of the catchment within the 2D model domain. The modelled river centre-line is shown in black with red areas representing blocked cells i.e. buildings, the area beyond the 2D model domain or the river reaches modelled within the 1D model domain. There was no further post processing of the data contained within the mesh required. Changes in the vertical scale of this map are outlined by the index, all levels have been set to OD Malin (metres).



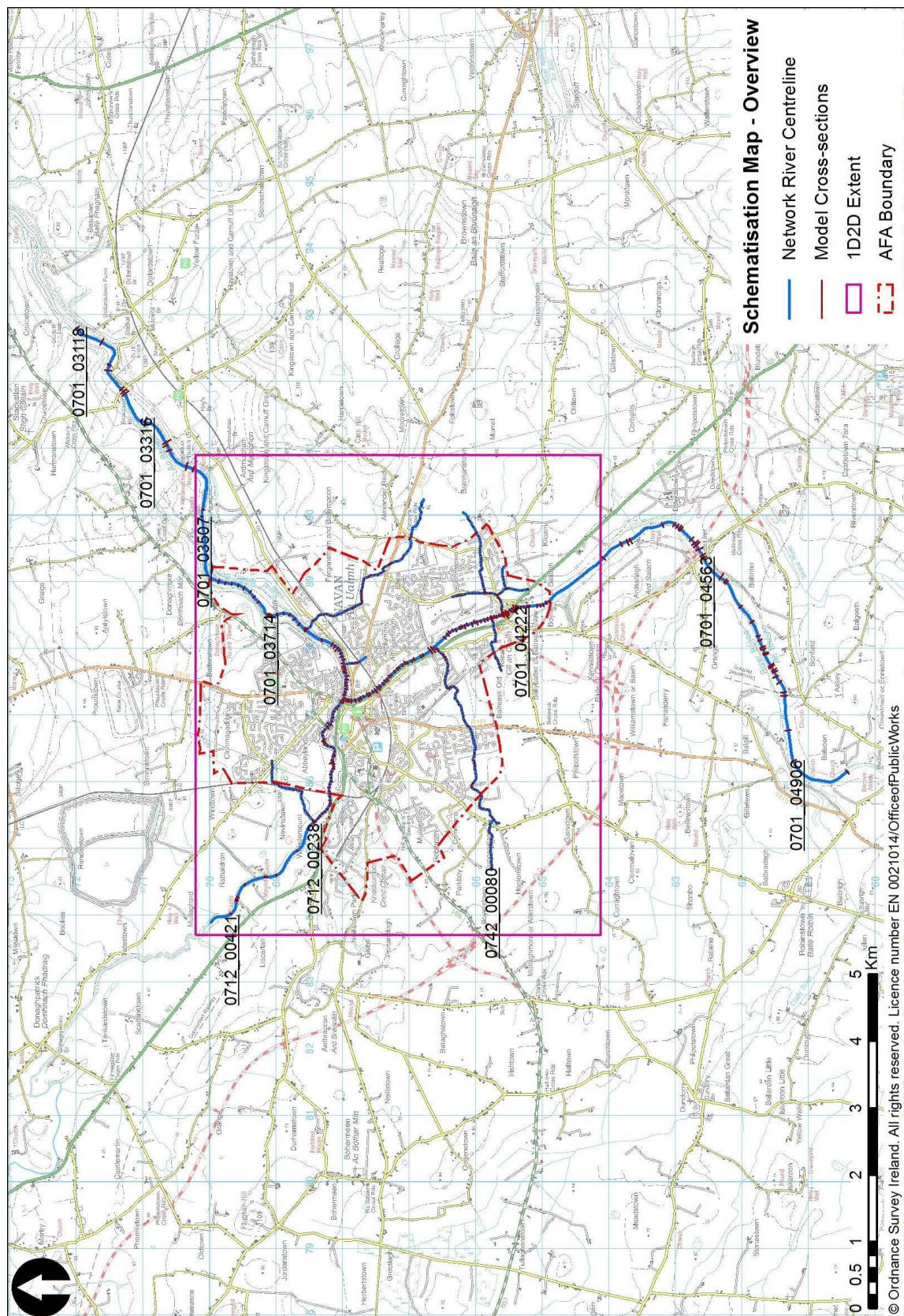


Figure 4.8.3: Overview of Model Schematisation

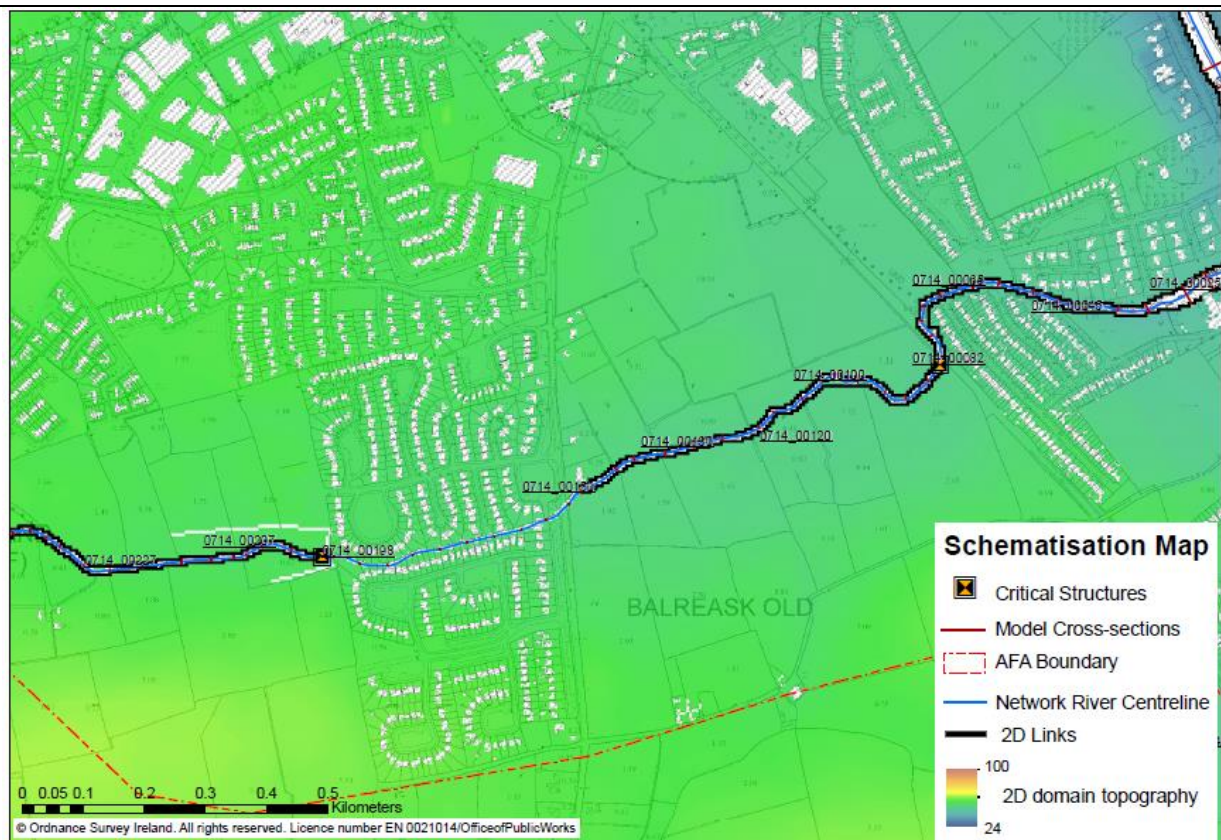


Figure 4.8.4: Model Schematisation on the Trim Road Tributary (Swan River)

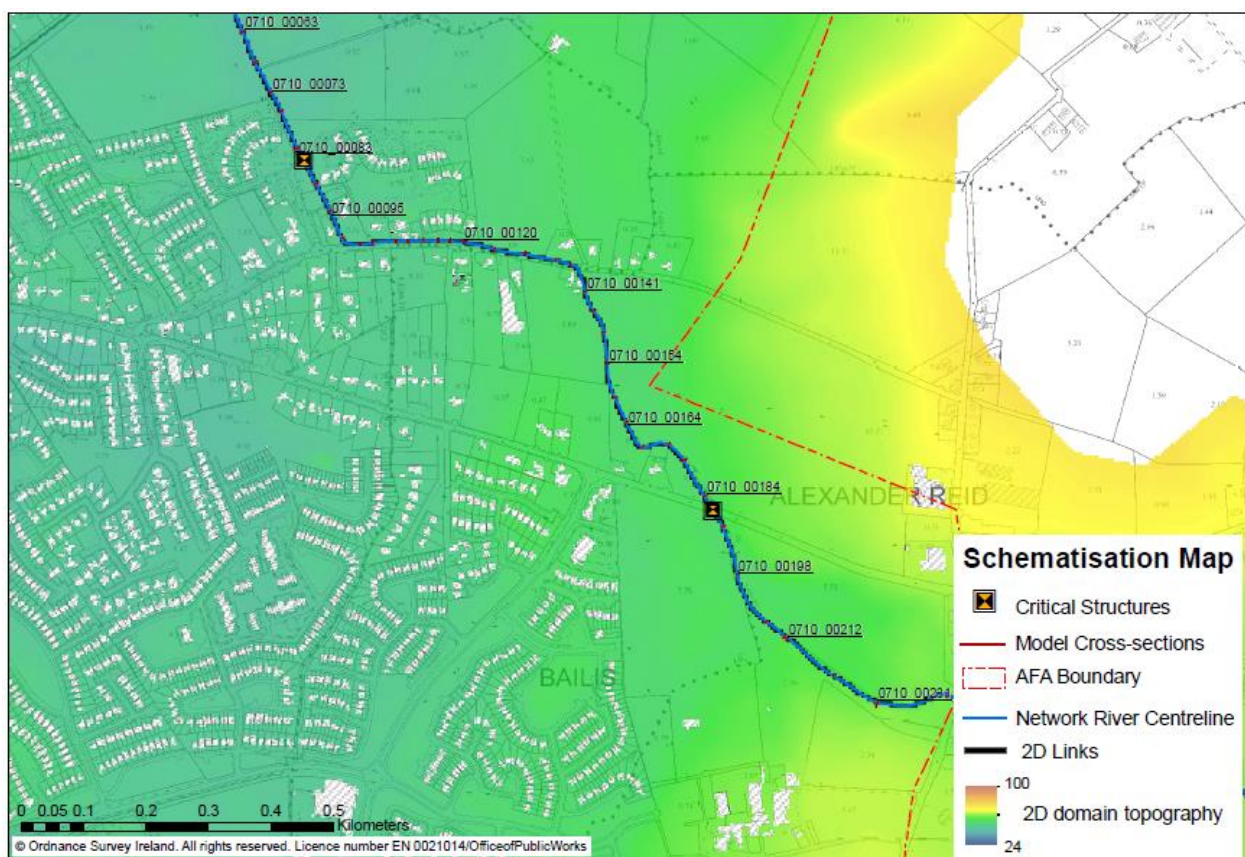


Figure 4.8.5: Model Schematisation on the Bailis Tributary

(8) Survey Information		
(a) Survey Folder Structure:		
First Level Folder	Second Level Folder	Third Level Folder
Murphy_E07_M06_WP3_0701C_120627 Where: Navan Murphy: Surveyor Name E07: Eastern CFRAM Study Area Hydrometric Area 7 M06 : Model Number 6 0701C: River Reference WP3: Work Package 3 Version: Accessible up to date 120627: Date Issued (27 JUN 2012)	GIS and Floodplain Photos (e.g. V0_0701_C_GIS and Floodplain Photos)	Flood Defence Register Structure Register Surveyed Cross Section Lines
	Ascii (e.g. V0_0701_C_Ascii)	
	XS Drawings & PDFs	
	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	
(b) Survey Folder References:		
<u>Reach ID</u>	<u>Name</u>	<u>File Ref.</u>
0701	RIVER BOYNE	Murphy_E07_M06_WP3_0701C_120627 Murphy_E07_M06_WP3_0701D_120627 Murphy_E07_M06_WP3_0701E_120627 Murphy_E07_M04_M06_WP3_0701F_120627
0715	OLD BALREASK TRIBUTARY - BOYNE	Murphy_E07_M06_WP3_0715_120627
0716	ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	Murphy_E07_M06_WP3_0716_120627
0717	ATHLUMNEY HOUSE TRIBUTARY 2 - BOYNE	Murphy_E07_M06_WP3_0717_120627
0718	ATHLUMNEY HOUSE TRIBUTARY 3 - BOYNE	Murphy_E07_M06_WP3_0718_120627
0719	ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE	Murphy_E07_M06_WP3_0719_120627
0714	TRIM ROAD TRIBUTARY 1 - BOYNE	Murphy_E07_M06_WP3_0714_120627
0742	TRIM ROAD TRIBUTARY 2 - BOYNE	Murphy_E07_M06_WP3_0742_120627
0711	ATHLUMNEY TRIBUTARY - BOYNE	Murphy_E07_M06_WP3_0711_120627
0710	BAILIS TRIBUTARY - BOYNE	Murphy_E07_M06_WP3_0710_120627
0712	RIVER BLACKWATER	Murphy_E07_M06_WP3_0712_120627

0713	ABBEYLANDS TRIBUTARY - BLACKWATER	Murphy_E07_M06_WP3_0713_120627
07009	NAVAN WEIR SURVEY	111115 Coordinate Surveys Data
07010	LISCARTAN SURVEY	111115 Coordinate Surveys Data

(9) Survey Issues:

There were no survey queries issued in relation to this model.

The cross-sections located within the 1D model reaches (between Ch 54450 - 60800 and Ch 70250 - 73298) have been reviewed and extended using the NDHM (in order to prevent 'glass walls' from affecting peak water levels and flows).

4.8.3 Hydraulic Model Construction


(1) 1D Structures (in-channel along modelled watercourses):	See Appendix H Number of Bridges and Culverts: 41 Number of Weirs: 15
<p>The survey information recorded includes a photograph of each structure, which has been used to determine the Manning's n value. The location of critical structures included in the model is presented in Figure 4.8.4 and Figure 4.8.5. Details of these structures are also presented in Appendix H.</p> <p>Figure 4.8.6 shows a 1.2 m diameter culvert located on the Trim Road Tributary1 (Swan River), this structure causes the flooding of agricultural land upstream. There are flood defences constructed upstream of this location to reduce the risk of flooding within the Balreask Manor development (see Section 4.8.3 (4)).</p>	
	
<p>Figure 4.8.6: Critical Structure 0714_00198_DN on the Trim Road Tributary 1:</p>	



Figure 4.8.7: Critical Structure 0714_00082_DN

Critical structure 0714_00082_DN, is a culvert under the railway embankment on Trim Road Tributary 1 (Swan River), this structure influences the flooding of agricultural land upstream.



Figure 4.8.8: Critical Structure 0710_00188D_DN

Critical Structure 0710_00188D_DN, is a culvert located along the Bailis Tributary (under the Kentstown Road). This critical structure contributes to the flooding of agricultural land upstream.



Figure 4.8.9: Critical Structure 0710_00086D_DN

Critical Structure 0710_00086D_DN is a culvert located on the Bailis Tributary (under Tubberclaire Meadows). This structure contributes to the flooding of roads and land upstream during the 0.1% AEP event.

(2) 1D Structures in the 2D domain (beyond the modelled watercourses):				None
(3) 2D Model structures:				None
(4) Standard of Protection of Existing Formal Defences:				
Structure Reference	Type	Watercourse	Bank	Modelled Standard of Protection (AEP)
1	Formal, wall	0701 (Boyne) Ch. 65064 to 65222m	Right	10%
2	Formal, embankment	0714 (Trim Road Trib 1 - Swan River) Ch. 1100 to 1350m	Left	1%
3	Formal, embankment	0714 (Trim Road Trib 1 - Swan River) Ch. 1240 to 1350m	Right	0.1%

Figure 4.8.10: **Defence 1 (A07NAV_008) Location and Photograph** shows Defence 1, which is an in-situ concrete raised wall in good condition, although the asset condition survey describes minor joint cracking. It is assumed that the wall has been designed as a formal flood defence - it has been identified as a defence during the topographical survey (however there is no additional information available to

confirm this assumption - see Section 4.8.6). The model shows that this defence has a standard of protection for the 10% AEP event, however, the benefitting area is negligible.



Figure 4.8.10: Defence 1 (A07NAV_008) Location and Photograph

Figure 4.8.11 shows Flood Defences 2 and 3, these embankments are recently constructed formal defences. As shown in Figure 4.8.11 this flood defence would reduce the flood risk of an area of Balreask Manor during a 0.1% AEP event. The defence is incorporated in the 2D model domain based on 'As Constructed' drawings. The design model simulations assume that there are no culvert blockages (which would result in an increased flood risk in the Balreask Manor area).

The flood defences were substantially completed by December 2013, with final completion in December 2014, as part of the Swan River Flood Alleviation Scheme. They were designed with a Standard of Protection of 1% AEP by RPS, and constructed by Ward and Burke Construction Ltd. Details of the scheme are contained within the 'Swan River Flood Alleviation Scheme Part 8 Planning Report (IBE0381/RW01/Aug 2010)'.

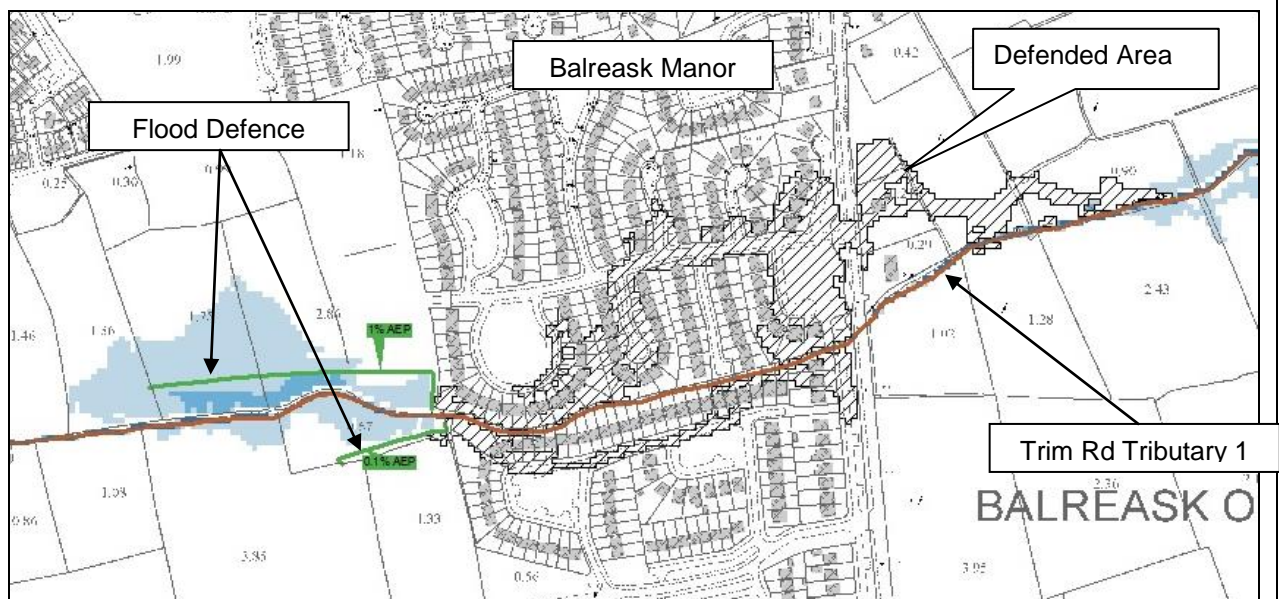


Figure 4.8.11: Flood Extents and the Defended Area in Balreask Manor

Other Structures

There are a number of structures which are not considered to be effective flood defences (and therefore are excluded from the model):

Structure 1: This is an embankment offset to the left bank of the River Boyne (Ch 62000 approx.) which does not protect low ground i.e. the embankment is linked to higher ground.

Structure 2: There are a series of concrete block raised walls offset to the left bank of the River Boyne along the Dublin and Kells Roads in Navan. Some of these are discontinuous walls. It is assumed that the walls have not been designed as formal flood defences and therefore are not effective.

Structure 3: There are stone gabion mattresses in very good condition on both banks of the Boyne close to its confluence with the River Blackwater. They are assumed to be porous structures and so are not effective defences.

(5) Model Boundaries - Inflows:

Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report_F01 - Section 4.13 and Appendix H). The boundary conditions implemented in the model are shown in Table 4.8.1.

Table 4.8.1: Model Boundary Conditions

Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
Open	Inflow	RIVER BOYNE	54450.331	0		07041_RPS
Open	Inflow	TRIM ROAD TRIBUTARY 2 - BOYNE	0	0		07_50000_U
Open	Inflow	ABBEYLANDS TRIBUTARY - BLACKWATER (KEL	1160.685	0		07_1866_1
Open	Inflow	BAILIS TRIBUTARY - BOYNE	0	0		07_22_2
Open	Inflow	ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	723.434	0		07_1853_U
Open	Inflow	ATHLUMNEY HOUSE TRIBUTARY 2 - BOYNE	0	0		07_19_1
Open	Inflow	ATHLUMNEY HOUSE TRIBUTARY 3 - BOYNE	0	0		07_21_U
Open	Inflow	ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE	0	0		07_1851_U
Open	Inflow	ATHLUMNEY TRIBUTARY - BOYNE	0	0		07_1823_1
Open	Inflow	OLD BALREASK TRIBUTARY - BOYNE	0	0		07_1065_U
Open	Inflow	RIVER BLACKWATER (KELLS)	0	0		07_625_4
Open	Q-h	RIVER BOYNE	73297.603	0		TBC
Point Source	Inflow	RIVER BOYNE	59449.061	0		07_1629_3
Distributed Source	Inflow	TRIM ROAD TRIBUTARY 1 - BOYNE	0	3280		07_1188_U-07_1188_5_RPS
Distributed Source	Inflow	RIVER BOYNE	58996.6	63952.972		07009_RPS-07041_RPS
Point Source	Inflow	RIVER BLACKWATER (KELLS)	174.991	0		07_1448_3
Point Source	Inflow	RIVER BLACKWATER (KELLS)	1959.956	0		07_1439_5
Distributed Source	Inflow	RIVER BLACKWATER (KELLS)	0	1447.61		07010
Distributed Source	Inflow	RIVER BLACKWATER (KELLS)	1447.61	4485.028		07010-07037
Distributed Source	Inflow	RIVER BOYNE	63952.972	70409.454		07_1490_1-07009_RPS
Point Source	Inflow	RIVER BOYNE	72676.828	0		07_1833_4
Open	Inflow	TRIM ROAD TRIBUTARY 1 - BOYNE	0	0		07_1188_U
Distributed Source	Inflow	ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE	0	1321		07_1851_1
Distributed Source	Inflow	ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	723.434	1675		07_1853_U-07_1523_2
Distributed Source	Inflow	OLD BALREASK TRIBUTARY - BOYNE	0	860		07_1065_U-07_1065_1
Distributed Source	Inflow	TRIM ROAD TRIBUTARY 2 - BOYNE	0	809		07_50000_U-07_50000_1
Distributed Source	Inflow	BAILIS TRIBUTARY - BOYNE	0	3060		07_22_2-07_1487_7
Distributed Source	Inflow	RIVER BOYNE	70409.453	72860		07_1490_1-TBC

In order to achieve modelled flows which correlate closely with the design (check) flows at the downstream checkpoint (07_1490_1_RPS), there were no amendments required to lateral or point inflows at draft stage - please refer to Section 4.8.5(2) which discusses model updates for the hydraulic model deliverables. One adjustment was required to the River Blackwater inflow hydrograph (07_625_4) - the peak of this hydrograph was brought forward by 8.5 hours in order to achieve closer correlation between modelled flows and design flows on the River Boyne, downstream of the Blackwater confluence. Refer to

Section 4.8.5(6) for further information.

Figure 4.8.10 provides an example of the associated upstream hydrographs on the River Boyne and River Blackwater at HEPs 07041_RPS and 07_625_4, respectively

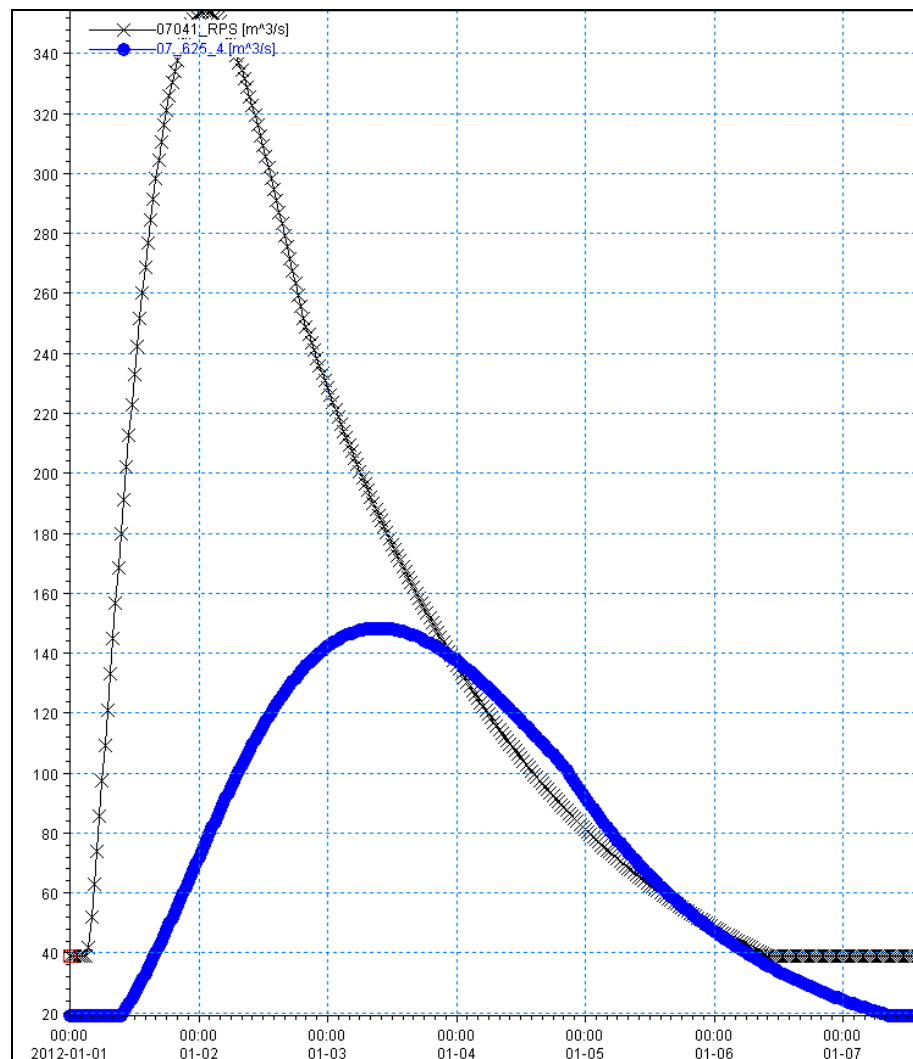


Figure 4.8.12: Upstream Hydrographs on the River Boyne and River Blackwater

(6) Model Boundaries – Downstream Conditions:

The downstream boundary condition (Chainage 73297.603) is an auto-calculated Q-h relationship based on Manning's formula, generated based on the cross-section at the downstream extent of the model, the slope and the Manning's n value at this location. The slope component is based on the river reach from Chainage 72398.59 to Chainage 73297.603.

(7) Model Roughness:

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.100
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.035

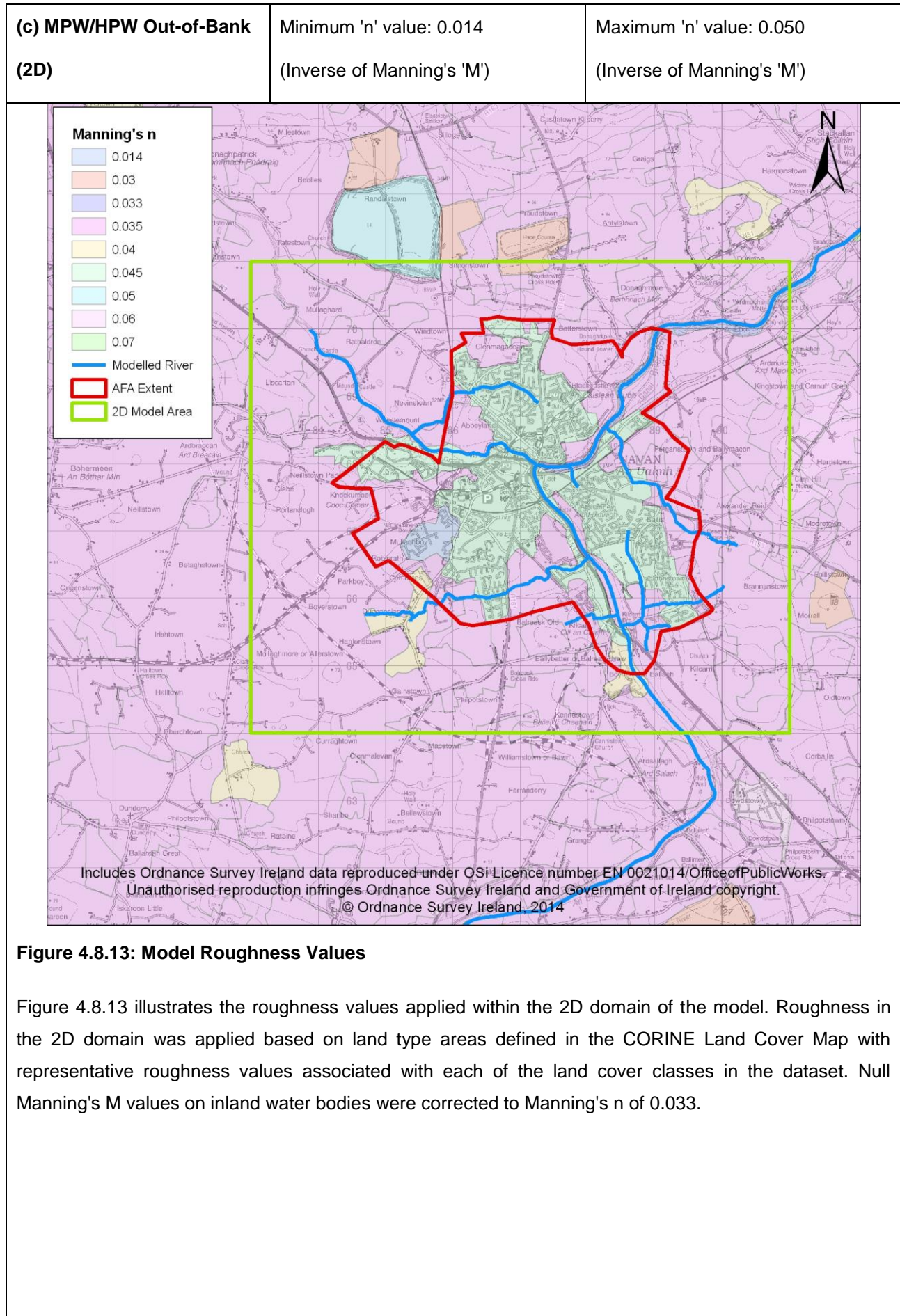


Figure 4.8.13: Model Roughness Values

Figure 4.8.13 illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the CORINE Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset. Null Manning's M values on inland water bodies were corrected to Manning's n of 0.033.

(d) Examples of In-Bank Roughness Coefficients *(based on Table 3.2 in Chapter 3.3.5)***Figure 4.8.14: Manning's $n = 0.045$**

Trim Road Tributary 1 - 0714_00135J_DN

River with shallows and meanders and noticeable aquatic growth

**Figure 4.8.15: Manning's $n = 0.035$**

River Boyne - 0701_003366_DN

Standard natural stream or river in stable condition

**Figure 4.8.16: Manning's $n = 0.100$**

Old Balreask Tributary - 0715_00059_DN

Slow flow meandering river with pools, slight rapids, very weedy and overgrown

**Figure 4.8.17: Manning's $n = 0.060$**

Abbeylands Tributary - 0713_00030_DN

River or stream with rocks and stones, shallow and weedy

4.8.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact 1% AEP hydraulic model within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a moderate increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.8.18. This outcome indicates that the Navan model demonstrates a moderate sensitivity to changes in roughness parameters. The increase in flood extents has a high impact upon properties located within the Navan AFA. Particularly, along Academy Street and the Bailis area of Navan. An additional 60 receptors are impacted accounting for an 85% increase compared to the design event.

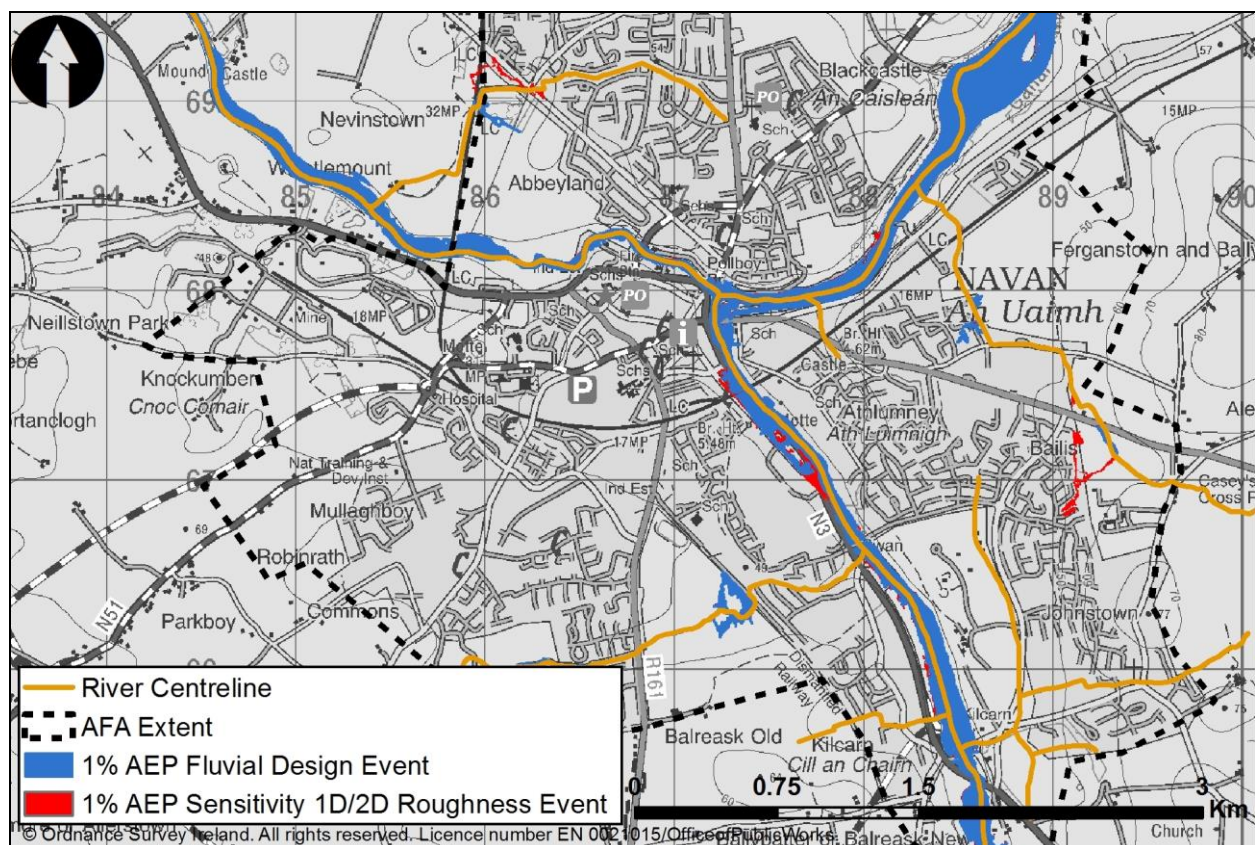


Figure 4.8.18: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in 1D/2D Roughness

- b) Downstream boundary increase –The downstream boundary is located approximately 19km downstream of the AFA and is 23m lower than bed levels within the AFA. The downstream boundary parameter was evaluated as having no potential impact within the Navan AFA and therefore a sensitivity simulation was not required.
- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Navan model was assessed as having low uncertainty/sensitivity, generally due to

the availability of high quality gauging data (refer to the Hydrology Report IBE0600Rp0012 for further detail); factors of 1.37 and 1.68 are applied to design flows for the sensitivity simulation. Figure 4.8.19 shows that the Navan model has a moderate sensitivity to increased inflow parameters; this is reflected by the moderate increase of the flood extent, particularly associated with the Bailis, Academy Street and Tubberclaire Meadows. These changes have a high impact on receptors as approximately 75 additional buildings are affected. This is a relative 106% increase when compared to the 1% AEP design results.

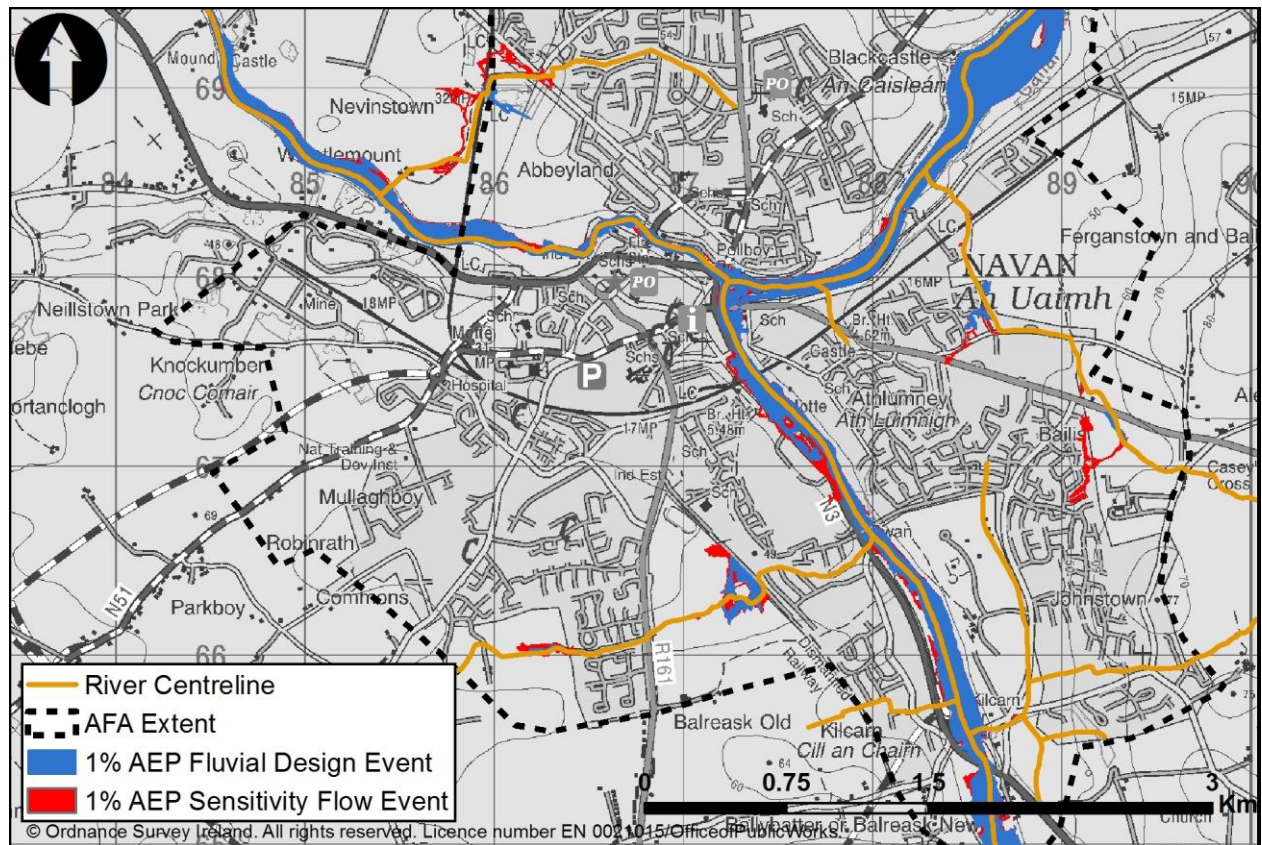


Figure 4.8.19: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event

- d) Afflux/Head loss at key structures – A number of simulations were carried out to assess the sensitivity of flood extents to changing the head loss coefficients of key structures. Several weirs were assessed including 0712_00076W, 0712_00050D, 0712_00044W and 0701_4050W, respectively located on the River Blackwater and River Boyne. Figure 4.8.20, illustrates that the Navan model has a low sensitivity to head loss parameter changes as there was little change in flood extents. Consequently, there is no further impact to receptors within the AFA.

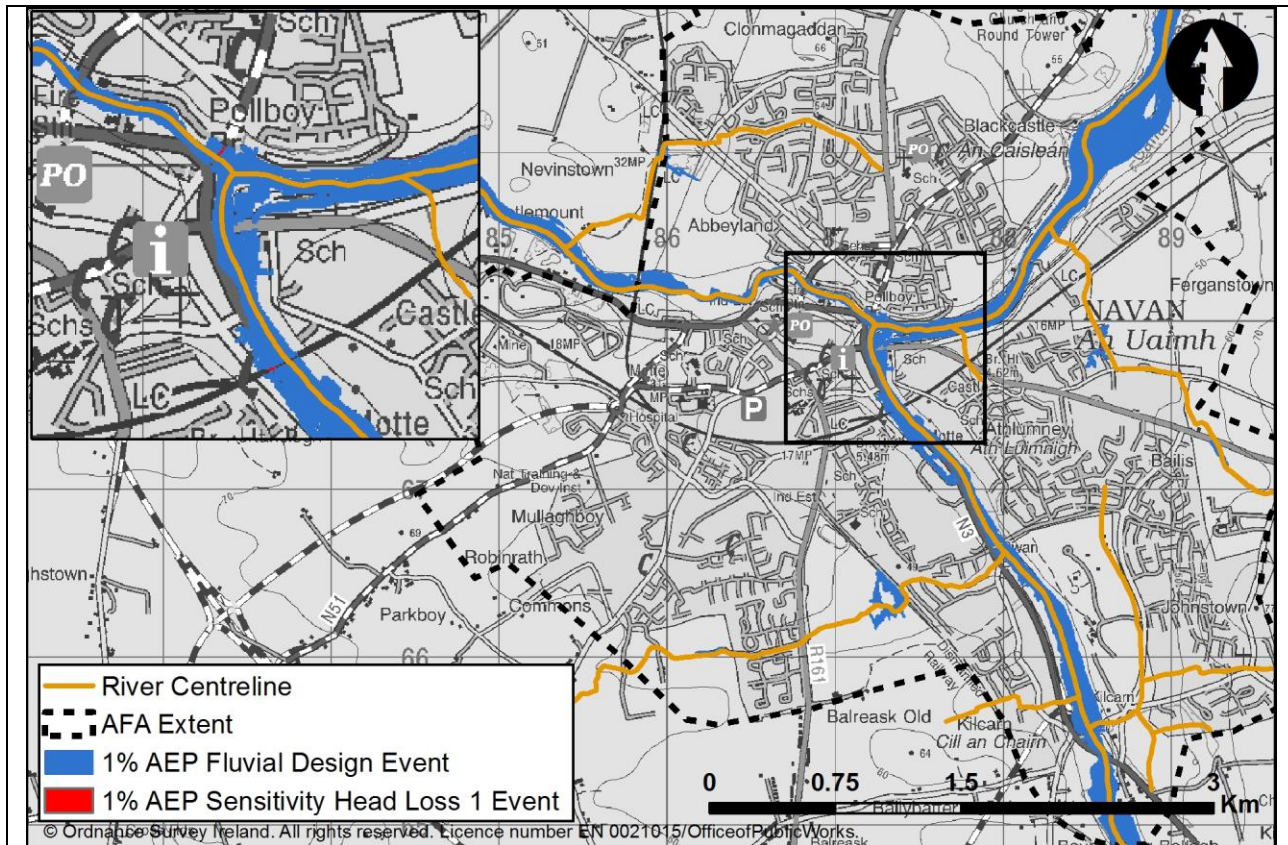


Figure 4.8.20: Comparison between 1% AEP Design Event and 1% AEP Sensitivity (Head Loss 1)

- e) Building representation – Building representation was modelled by adjusting the roughness of cells within the building footprint to a Manning's n of 0.3. The topography within the 2D model domain was based on LiDAR - the cells within building footprints remained 'unblocked'. Figure 4.8.21 shows that the Navan model has low sensitivity to building representation, as revealed by the overall low increase in the 1% AEP flood extent. This change has a high impact on receptors as approximately 18 additional buildings are affected. Affected properties are located along Academy Street and Pollboy Street which are situated within a close proximity to the River Boyne and River Blackwater, respectively. In this instance, the removal of riverside buildings has the extension and re-direction of flood flow paths allowing increased flooding riverside locations. Overall, this is a 25% increase when compared to the 1% AEP design results.

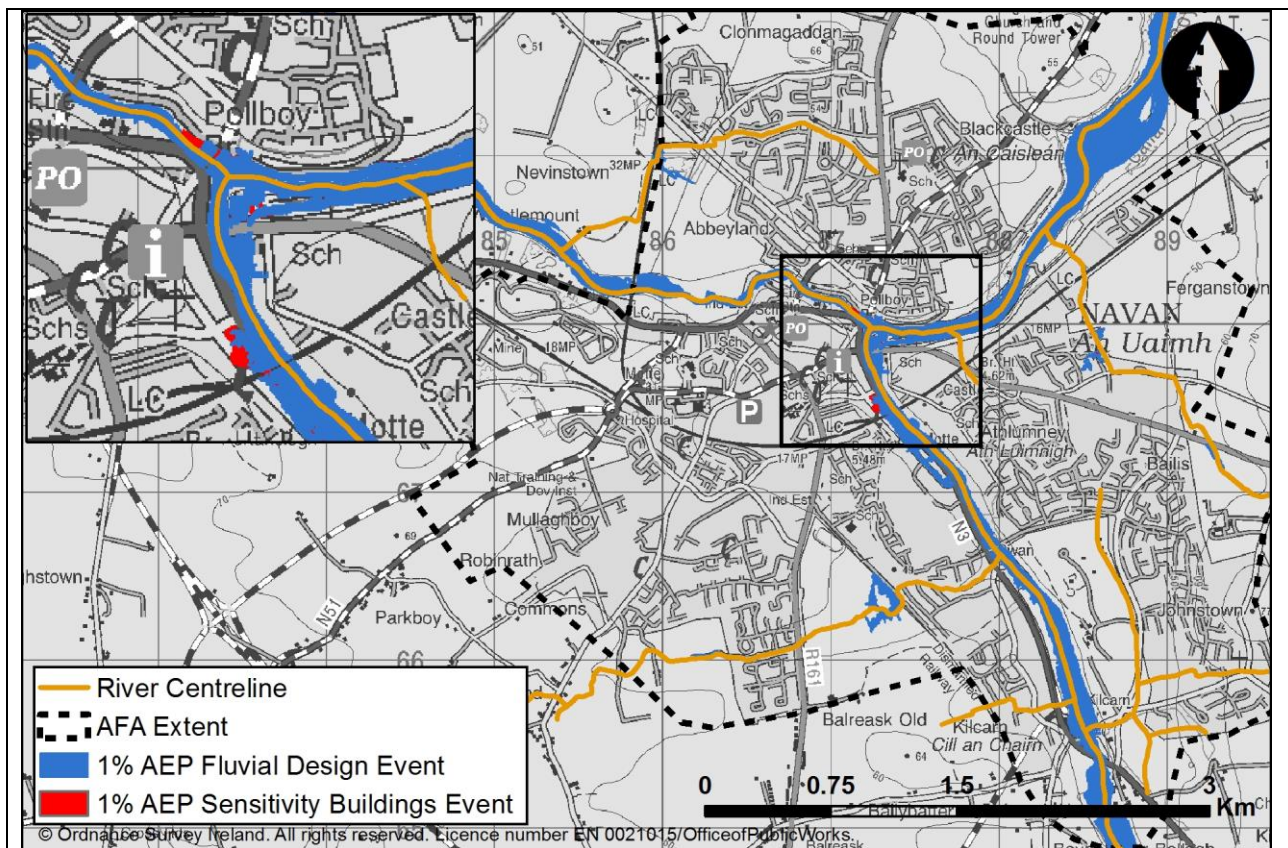


Figure 4.8.21: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Buildings Event

Table 4.8.2 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Navan model. Of these parameters assessed, the model demonstrates a moderate sensitivity to an increase of model inflows. Table 8.1 of the Hydrology Report assessed that the Navan model was assessed as having low uncertainty/sensitivity, generally due to the availability of high quality gauging information. '1D/2D Roughness' and 'Removal' also revealed moderate model sensitivity which also resulted in a high impact to properties located within the AFA. The model demonstrates low sensitivity to the other parameters, with the resulting analysis identifying a low to negligible increase in flood extents and low impact to receptors within the AFA. Similarly, the 'Buildings Event' also revealed low model sensitivity; conversely the results revealed a high impact to properties located within the AFA. This impact tended to be site specific, as the majority of affected properties are associated with densely populated localities that are within a close proximity to the River Blackwater and River Boyne.

Table 4.8.2: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	Moderate	High
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	Moderate	High
1% AEP Sensitivity Head Loss 1 Event	Low	Low
1% AEP Sensitivity Head Loss 2 Event	Low	Low
1% AEP Sensitivity Head Loss 3 Event	Low	Low
1% AEP Sensitivity Buildings Event	Low	High

4.8.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0600Rp0004_HA07 Inception Report_F02 unless otherwise specified):

(a) 20/11/2009	<p>A press article states how firemen in Navan pumped water from Academy Street and on the Commons Road throughout the night to keep floodwaters at bay.</p> <p>Data at the 07009 gauging station on the River Boyne is missing for this flood event. The MIKE NAM rainfall-runoff model (see Hydrology Report) estimates a peak discharge of 219.106 m³/s on 20/11/09. This is less than the 10% AEP modelled flow (266.01 m³/s). There is no information available at the 07041 gauging station on the River Boyne for this event.</p> <p>A preliminary assessment of past floods (Table 4.9 in the Inception Report) states the approximate AEP of this event is greater than 1% at 07010 on the River Blackwater. The peak water level recorded at 07010 was 3.842 m (at 23:00 on 19/11/09), which is equivalent to 39.44 mOD and a peak discharge of 170.024 m³/s - the highest on record. This preliminary estimate was calculated using a Single Site Analysis based on 53 years of records, and therefore it is appropriate to say that the event had an AEP of at least 1% (as the analysis is unable to generate a return period greater than twice the length of the period of record). The recorded peak water level and flow are significantly higher than the corresponding model values (which are 38.314 mOD and 147.19 m³/s respectively) for the 1% AEP event in the defended model.</p> <p>The peak water level recorded at 07037 was 2.790 m on 20/11/09. This is equivalent to 31.28 mOD which is less than the modelled water level of 31.473 mOD for the 10% AEP event in the defended model. It is noted that an almost identical level was recorded at this gauge (07037) on 17/08/2008 (see Section 4.8.5 1(b)). At</p>
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	<p>the 07010 gauge, the peak recorded level was 2.705 m during the August 2008 flood event, which is 1.137 m lower than the corresponding level recorded during the November 2009 event.</p> <p>RPS contacted OPW Hydrometrics about the level recorded by the 07010 gauge during the November 2009 flood event. OPW Hydrometrics stated that the logger was not reliable during this period meaning that the recorded level is definitely incorrect (email correspondence to RPS on 16/02/15).</p> <p>The MIKE NAM Rainfall Runoff Model and the modelled flows at the 07037 gauge suggest that the November 2009 flood event had a frequency less than 10% AEP. For the August 2008 event, the modelled flows at both the 07010 and 07037 gauges support the estimated return period of 10-6.67% along the River Blackwater. This leads to a discrepancy with the recorded level at the 07010 gauge during the flood event on 20/11/09. The estimated flow is almost twice as much as the next highest flow recorded during the period of record. RPS have contacted OPW Hydrometrics to query the validity of the meter reading and will provide an update in the final version of this report.</p> <p>Academy Street shows flooding during the 1% and 0.1% AEP events. Section 4.6.2 of the Navan Development Plan 2013-2019, Flood Risk Assessment and Management Plan Draft Report, January 2014 (hereafter referred to as the Development Plan) states that flooding in Academy Street could be from a combination of fluvial and pluvial sources, as the storm water system in the area outfalls to the River Boyne.</p> <p>There is no information on which part of the Commons Road is affected by flooding, or the source of the flooding. The Commons Road crosses the Trim Road tributary (Figure 4.8.12), however, there is no flooding of the road at this location. As the modelled flows show a good correlation with design flows (Section 4.8.5(6)), it is assumed that the model is representative and that the flooding is from an alternative source or due to culvert blockage. There are no further details of flood extents, levels or the source of flooding for this flood event.</p>
(b) 17/08/2008	<p>Heavy rainfall on 16th August resulted in the River Boyne overtopping its banks in Navan when the drainage capacity of the sewers was overwhelmed. The Newgrange Hotel in Navan was flooded and some roads/streets including Cannon Row, Circular Road, Commons Lane and roads at Ardsallagh, Cannistown and Bloomsbury Bridge were also flooded.</p> <p>A preliminary assessment of past floods (Table 4.9 in the Inception Report) states the approximate AEP of this event is 20-10% at 07009 and 10-6.67% at 07010. The peak water level recorded at 07009 was 2.577 m (at 11:30 on 17/08/08), which is equivalent to 32.865 mOD and a peak discharge of 239.155 m³/s. This is less than the modelled water level of 33.186 mOD and peak flow of 266.01 m³/s for the 10%</p>

	<p>AEP event in the defended model. Therefore, this supports the preliminary estimate of return period provided in the Inception report at 07009.</p> <p>The peak water level recorded at 07010 was 2.705 m (at 15:30 on 17/08/08), which is equivalent to 38.303 mOD and a peak discharge of 93.635 m³/s. This is slightly less than the modelled water level of 38.314 mOD for the 1% AEP event in the defended model. The peak modelled flow at this location for the 10% AEP event is 105.75 m³/s which is more than the recorded flow. The modelled water levels and flows support the preliminary estimate of return period provided in the Inception report.</p> <p>The peak water level recorded at 07037 was 2.792 m at 11:00 on 17/08/08. This is equivalent to 31.282 mOD which is less than the modelled water level of 31.473 mOD for the 10% AEP event in the defended model. The modelled water levels support the preliminary estimate of return period provided in the Inception report for the River Boyne (which dominate the water level at the 07037 location).</p> <p>Section 4.6.2 of the Development Plan states that the cause of flooding to the Newgrange Hotel has been attributed to a blockage in the sewer pumping station, was has now been resolved. It also states that the cause of flooding at Cannon Row and Commons Lane is thought to be due to the inability of the storm water system to adequately convey surface water.</p> <p>The road at Bloomsbury Bridge and roads at Ardsallagh and Cannistown are located beyond the extents of the model - see Figure 4.8.22.</p>
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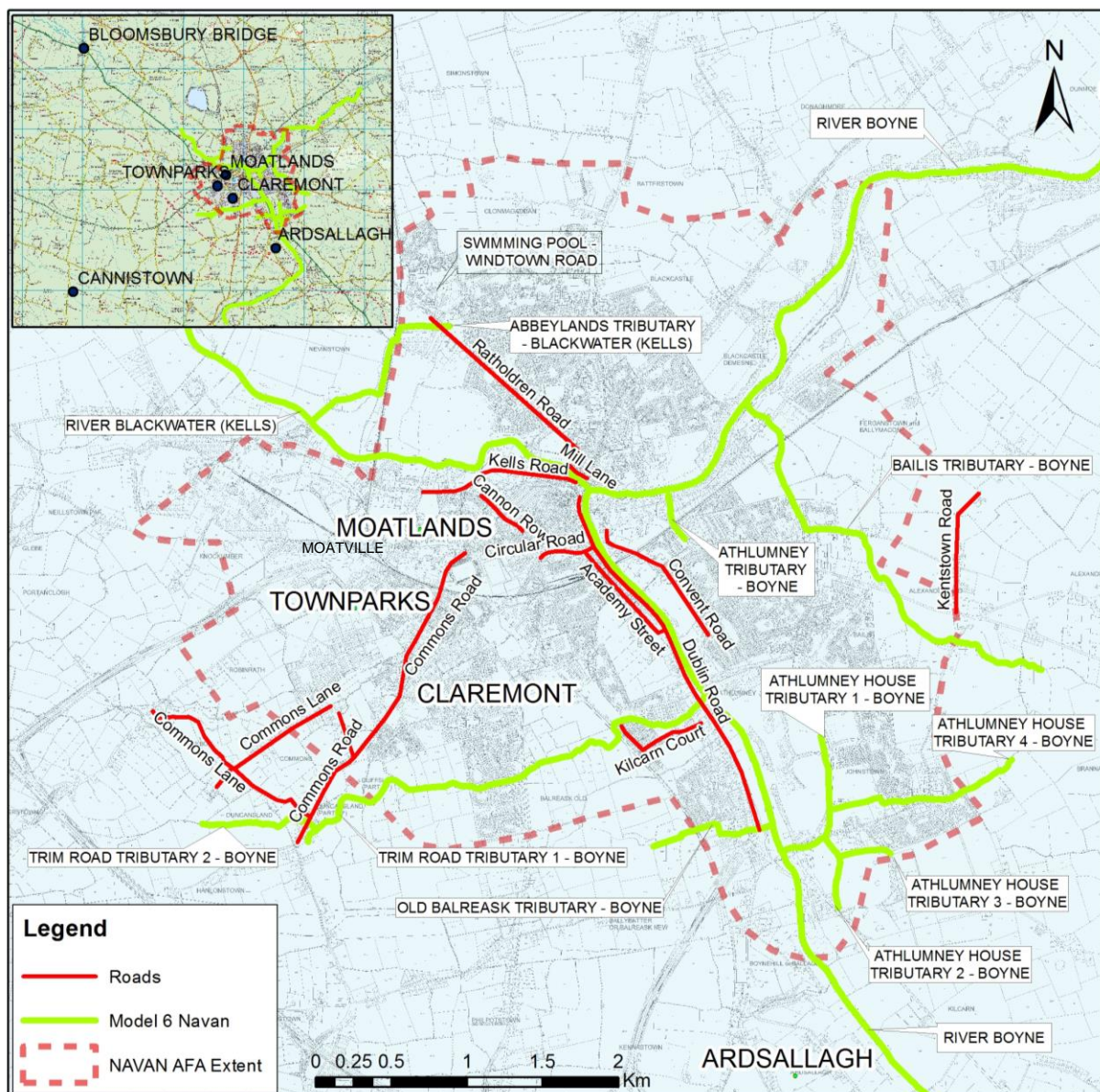


Figure 4.8.22: Historical Flooding of Navan Roads

(c) 15/11/2002

The "Flood Risk Assessment Study of Mill Lane and Convent Road Sites, Navan" states that the flood event in Navan had an AEP of 10%. Flooding occurred in the Townparks, Academy Street, Claremont and Moatlands areas of Navan.

The peak water level recorded at 07009 was 2.842 m (at 12:30 on 15/11/02), which is equivalent to 33.130 mOD and a peak discharge of 289.976 m³/s. This is 56 mm less than the modelled water level of 33.186 mOD for the 10% AEP event in the defended model. Figure 4.8.23 shows how these levels compare and that both the recorded hydrograph and 10% AEP modelled hydrograph are similar in shape for this flood event. The peak modelled flow at this location for the 10% AEP event is 266.01 m³/s which is within 10% of the recorded flow. This supports the estimate of return period provided by The "Flood Risk Assessment Study of Mill Lane and Convent Road Sites, Navan" report.

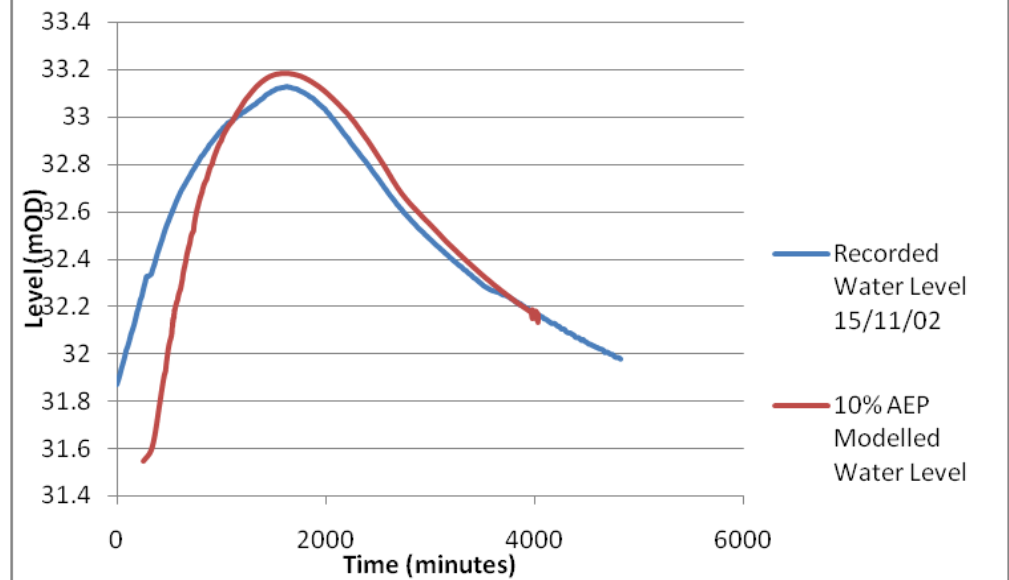


Figure 4.8.23: Recorded and Modelled Water Level Hydrographs at 07009

The peak water level recorded at 07010 was 2.377 m (at 00:15 on 15/11/02), which is equivalent to 37.975 mOD and a peak discharge of 75.190 m³/s. The recorded level is within 70 mm of the modelled water level of 37.911 mOD for the 10% AEP event. The peak modelled flow at this location for the 10% AEP event is 105.75 m³/s which is more than the recorded flow. The model water levels support the estimate of return period provided in the "Flood Risk Assessment Study of Mill Lane and Convent Road Sites, Navan" report. As the modelled flows show a good correlation with design flows (within 2% for each return period - see Section 4.8.5(5)), it is assumed that the model is representative.

The peak water level recorded at 07041 was 3.055 m at 11:15 on 15/11/02, which is equivalent to 39.345 mOD and a peak discharge of 252.520 m³/s. The recorded level is 138 mm less than the modelled water level of 39.483 mOD and the recorded flow correlates well with the modelled flow of 249.586 m³/s for the 10% AEP event in the defended model. The model supports the estimate of return period provided in the "Flood Risk Assessment Study of Mill Lane and Convent Road Sites, Navan" report.

There is no information on the source of the flooding identified above. The Townparks, Claremont and Moatlands areas are not located in areas which could be affected by flooding from the modelled watercourses, and so it is considered that the flooding identified above is from an alternative source (Figure 4.8.22). Academy Street shows flooding during the 1% and 0.1% AEP events. Section 4.6.2 of the Development Plan states that flooding in Academy Street could be from a combination of fluvial and pluvial sources, as the storm water system in the area outfalls to the River Boyne.

Generally, the model supports the estimate of return period provided in the "Flood

	<p>Risk Assessment Study of Mill Lane and Convent Road Sites, Navan" report. A preliminary assessment of past floods (Table 4.9 in the Inception Report) states the approximate AEP of this event is 4% at 07009. The modelled levels and flows estimate a more frequent flood event than the Inception report. However, as the modelled flows show a good correlation with design flows (within 4% for each return period - see Section 4.8.5(5)), it is assumed that the model is representative.</p>
(d) 07/11/2000	<p>In Navan, the flooding was evident by roads being impassable and the swimming pool was flooded. The Moatville, Academy Street, Liscartan and Kilcarn Court areas also flooded. It was estimated in an OPW memo that the flood event had an AEP of 3.33% based on the flow in the River Boyne at Slane Castle.</p> <p>A preliminary assessment of past floods (Table 4.9 in the Inception Report) states the approximate AEP of this event is 6.67% at 07009 and 10% at 07010. The peak water level recorded at 07009 was 2.806 m (at 22:30 on 06/11/00), which is equivalent to 33.094 mOD and a peak discharge of 282.490 m³/s. This is 92 mm less than the modelled water level of 33.186 mOD for the 10% AEP event in the defended model. The peak modelled flow at this location for the 10% AEP event is 266.01 m³/s which is less than the recorded flow. This modelled flow supports the preliminary estimate of return period provided in the Inception report and the OPW memo.</p> <p>The peak water level recorded at 07010 was 2.454 m (at 01:45 on 07/11/00), which is equivalent to 38.052 mOD and a peak discharge of 79.404 m³/s. This is less than the modelled water level of 38.314 mOD for the 1% AEP event and 141 mm greater than the modelled water level of 37.911 mOD for the 10% AEP event in the defended model. The peak modelled flow at this location for the 10% AEP event is 105.75 m³/s which is more than the recorded flow. The model supports the preliminary estimate of return period provided in the Inception report.</p> <p>The peak water level recorded at 07037 was 3.150 m on 07/11/00 (estimated from post-event survey). This is equivalent to 31.64 mOD which is more than the 10% AEP modelled water level but less than the 1% AEP level at this location (31.473 mOD and 32.19 mOD respectively). The model supports the estimate of return period provided in the OPW memo.</p> <p>The peak water level recorded at 07041 was 2.956 m at 00:45 on 07/11/00, which is equivalent to 39.246 mOD and a peak discharge of 237.184 m³/s. The recorded level is 237 mm less than the modelled water level of 39.483 mOD the 10% AEP event in the defended model. The recorded flow is slightly less than the modelled flow of 249.586 m³/s for the 10% AEP event in the defended model. The modelled levels and flows estimate a more frequent flood event than the Inception report. However, as the recorded flow is significantly higher than the Q_{med} value of 161.0 m³/s, and as the modelled flows show a good correlation with design flows (Section 4.8.5(5)), it is assumed that the model is representative.</p>

There is no information on the source of the flooding to some of the areas identified above. The Moatville area is not located in an area which could be affected by flooding from the modelled watercourses, and so it is considered that the flooding identified above is from an alternative source (Figure 4.8.22).

The Liscartan area, which is located along the boundary of the 1D-2D model domain, shows flooding within the 10% AEP event (as shown in Figure 4.8.24) which corresponds to the historical flooding information given above.

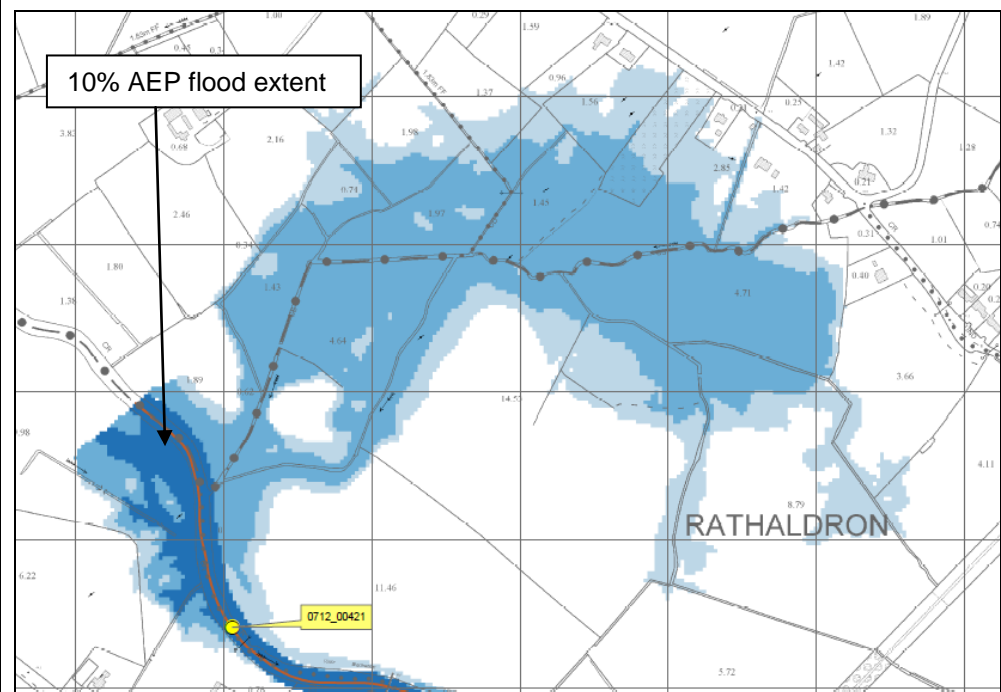


Figure 4.8.24: Modelled Flood Extents within the Liscartan Area

The Kilcarn Court area is located adjacent to Trim Road Tributary 1 (see Figure 4.8.25). The flood flows are attenuated by the embankment upstream of Kilcarn Court. The model incorporates a culvert under the embankment which is representative of the culvert recorded as part of the topographical survey. In order for the model to generate flood extents covering part of the Kilcarn Court area, an assumption would be required either that the culvert under the embankment is incorrect, or that there is an unknown alternative flow path. For the purposes of this report, it is assumed that the culvert is modelled correctly and no alternative flow paths are known. All culverts that have been identified have been included in the model. It should be noted that the ground levels at Kilcarn Court are approximately 10 metres higher than the levels adjacent to the River Boyne and so a backwater effect would not be the mechanism for flooding at Kilcarn Court.

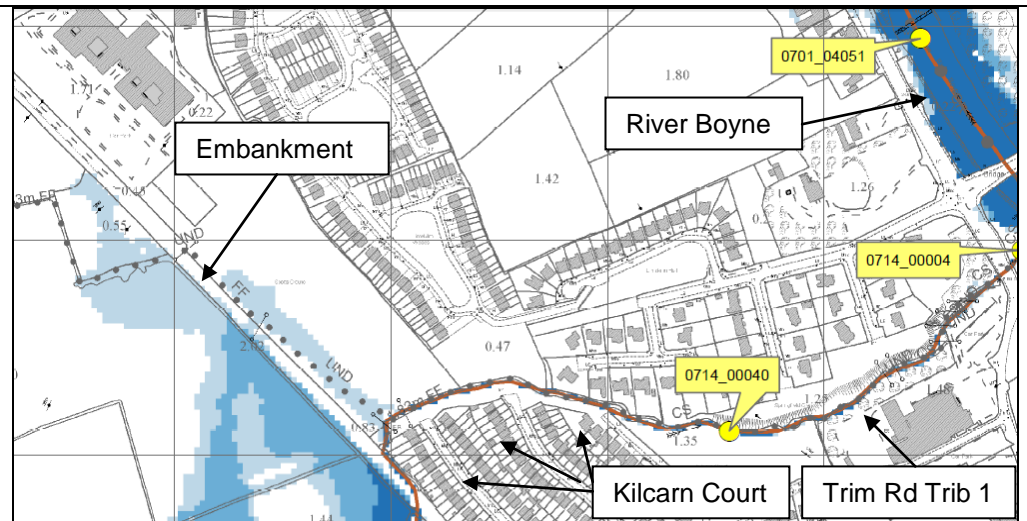


Figure 4.8.25: Modelled Flood Extents along the Swan River, in Kilcarn

The swimming pool, which is located off the Windtown Road (Figure 4.8.22) was subject to flooding during this event. As it is not located near to any modelled watercourses, it is assumed that the flooding is from an alternative source.

Academy Street shows flooding during the 1% and 0.1% AEP events. Section 4.6.2 of the Development Plan states that flooding in Academy Street could be from a combination of fluvial and pluvial sources, as the storm water system in the area outfalls to the River Boyne. Generally, the model supports the estimate of return period provided in the Inception report.

(e) 12/06/1993

In Navan, both the River Boyne and River Blackwater overtopped their banks.

The peak water level recorded at 07009 was 2.457 m at 18:15 on 12/06/93, which is equivalent to 32.745 mOD and a peak discharge of 217.841 m³/s. Both the recorded peak level and flow are less than the 10% AEP modelled level and flow (33.186 mOD and 266.01 m³/s respectively).

The peak water level recorded at 07010 was 2.176 m at 11:15 on 12/06/93, which is equivalent to 37.774 mOD and a peak discharge of 68.682 m³/s. Both the recorded peak level and flow are less than the 10% AEP modelled level and flow (37.993 mOD and 105.75 m³/s, respectively).

The peak water level recorded at 07037 was 2.72 m on 11/06/93, which is equivalent to 31.21 mOD. This is less than the 10% AEP modelled level of 31.473 mOD.

The above analysis suggests that the return period of the June 1993 event was more frequent than an event of 10% AEP. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.

(f) January 1991	<p>Outline information is available for a flood event in Navan in January 1991. No details of cause of flooding, source or flows are available, with the only information reported being of flooding in Academy Street.</p> <p>The peak water level recorded at 07009 was 1.579 m at 16:15 on 06/01/91, which is equivalent to 31.867 mOD and a peak discharge of 91.289 m³/s. This is less than the modelled water level of 33.186 mOD and peak flow of 266.01 m³/s for the 10% AEP event in the defended model. The 10% AEP model does not predict flooding of Academy Street. However, Section 4.6.2 of the Development Plan states that flooding in Academy Street could be from a combination of fluvial and pluvial sources, as the storm water system in the area outfalls to the River Boyne.</p> <p>The peak water level recorded at 07010 was 2.254 m at 13:30 on 06/01/91, which is equivalent to 37.852 mOD and a peak discharge of 72.750 m³/s. This is less than the modelled water level of 37.993 mOD for the 10% AEP event in the defended model. The peak modelled flow at this location for the 10% AEP event is 105.75 m³/s which is more than the recorded flow, suggesting a return period for this event more frequent than 10% AEP. Due to the limited information available for this flood event, it has not been used during model calibration or validation.</p>
(g) January 1969	<p>An OPW report indicates that flooding occurred during December 1968/January 1969 in Navan when the River Boyne overflowed. The affected area was at Kilcarn, near Navan. However, no exact date or any further details are available for this flood event. Please see Part (d) above for additional details on the flood risk at Kilcarn.</p> <p>Kilcarn is located south of Navan, along the river Boyne between the 07041 and 07009 gauging stations. There is no data available from either station for this flood event and so this flood event has not been used for model calibration or validation.</p>
(h) November 1965	<p>In Navan, the River Boyne overflowed. The peak flow at Liscartan Hydrometric Station (07010) was estimated to be 65.7m³/s in a report by Hydro Environmental Ltd in 2004. Reports indicate that Academy Street worst affected by the flooding where seven families were evacuated. The bridge in Navan flooded and the Dublin Road was also affected.</p> <p>Academy Street shows flooding during the 1% and 0.1% AEP events. The peak modelled flows along the River Boyne correlate well with the estimated flows (see Section 4.8.5(5)). There is no information available from the River Boyne gauging stations for this event. Section 4.6.2 of the Development Plan states that flooding in Academy Street could be from a combination of fluvial and pluvial sources, as the storm water system in the area outfalls to the River Boyne.</p> <p>The peak water level recorded at 07010 was 2.256 m at 11:00 on 26/11/65, which is equivalent to 38.048 mOD and a peak discharge of 63.59 m³/s. This is less than the</p>

	modelled water level of 38.314 mOD for the 1% AEP event and greater than the modelled water level of 37.911 mOD for the 10% AEP event in the defended model. The peak modelled flow at this location for the 10% AEP event is 105.75 m ³ /s which is more than the recorded flow. There is no information available on the areas affected by flooding (if any) along the River Blackwater during this event. Due to the limited information available for this flood event, it has not been used during model calibration or validation.
(i) Other events	Other flood events have been recorded during December 1981, December 1968, December 1954, January 1965 and March 1947. However no details of cause of flooding, source or flows are available and so these events are deemed to be unsuitable for use in model calibration.

Summary of Calibration

At Navan, there is a long history of flooding events with relatively good quality recorded water level and flow data at three gauging station locations, with water level data available at a fourth gauging station. Whilst there are details available on the road or street which has flooded, there is little detail on the source, extent or depth of flooding. Gauged information therefore has been relied on to ensure the model is calibrated and that the initial model parameters are representative. This includes in-channel and floodplain roughness coefficients; and structure roughness and head loss coefficients which did not require to be amended from those initially selected (based on normal bounds).

Model flows were checked against the estimated flows at HEP check points where possible to ensure they were within an acceptable range. One adjustment was required to the River Blackwater inflow hydrograph (07_625_4) - the peak of this hydrograph was brought forward by 8.5 hours in order to achieve closer correlation between modelled flows and design flows on the River Boyne, downstream of the Blackwater confluence. Section 4.8.5(5) states that the modelled flows generally correlate well with the estimated flows at each HEP, with justification provided where there is a significant difference - please refer to Section 4.8.5(5) for further details.

It is evident that there is a discrepancy with the peak recorded level and flow at the 07010 gauging station on the River Blackwater catchment during the November 2009 flood event. The model is unable to replicate the recorded flows at this location and at the 07037 gauging station on the River Blackwater (approximately 30 m upstream of the confluence with the River Boyne) during the same simulation for a particular return period.

Academy Street shows flooding during the 1% and 0.1% AEP events. There have been reports of flooding during more frequent flood events. Section 4.6.2 of the Navan Development Plan 2013-2019 states that flooding in Academy Street could be from a combination of fluvial and pluvial sources, as the storm water system in the area outfalls to the River Boyne. Due to close correlation between modelled flows and design flows along the River Boyne and the rating curves at both gauging stations, it is assumed that regular flooding on Academy Street is pluvially dominated.

There are a number of areas which are reported to have a history of flooding but are not located to modelled

watercourses - it is assumed that the reported flooding is from an alternative source or from a watercourse which has not been included in the model.

There are no significant instabilities within the model. The mass error in the model was calculated to ensure the model schematisation is robust. The mass error in the 1% AEP design run was found to be 1.83%, which is within acceptable limits meaning that the model is considered to be robust and stable. Overall the model is considered to be performing well for design event simulation and is supported by historic and hydrometric information.

(2) Post Public Consultation Updates:

All recorded comments were investigated following informal public consultation and formal S.I. public consultation periods in 2015, however, this did not result in a requirement to update the model. Following on from formal S.I. public consultation period, and in response to comments made by the OPW on the draft hydraulics deliverables, general model updates were applied to refine model resolution and improve model stability. This resulted in minor amendments to the flood extents - a revised set of flood hazard and risk mapping has been issued as Final to reflect these changes.

(3) Gauging Stations:

There are four gauging stations within the model extents, of which three have water level and flow information available - Navan Weir (07009), Liscartan (07010) and Ballinter Bridge (07041). There is a level-only gauge at Blackcastle (07037) which has water level data available from 1982. The recorder was removed from 08/09/04 to 03/04/06. A rating has not been developed at this site and so is excluded from the discussion below.

(a) Navan Weir (07009)

The Navan Weir gauging station (07009 – OPW) is located in the centre of the Navan AFA and has been given an A1 rating classification for the entire period of its rating from 1976. There is good confidence in the gauged Q_{med} value of 139.7 m³/s. This gauging station is subject to a rating review.

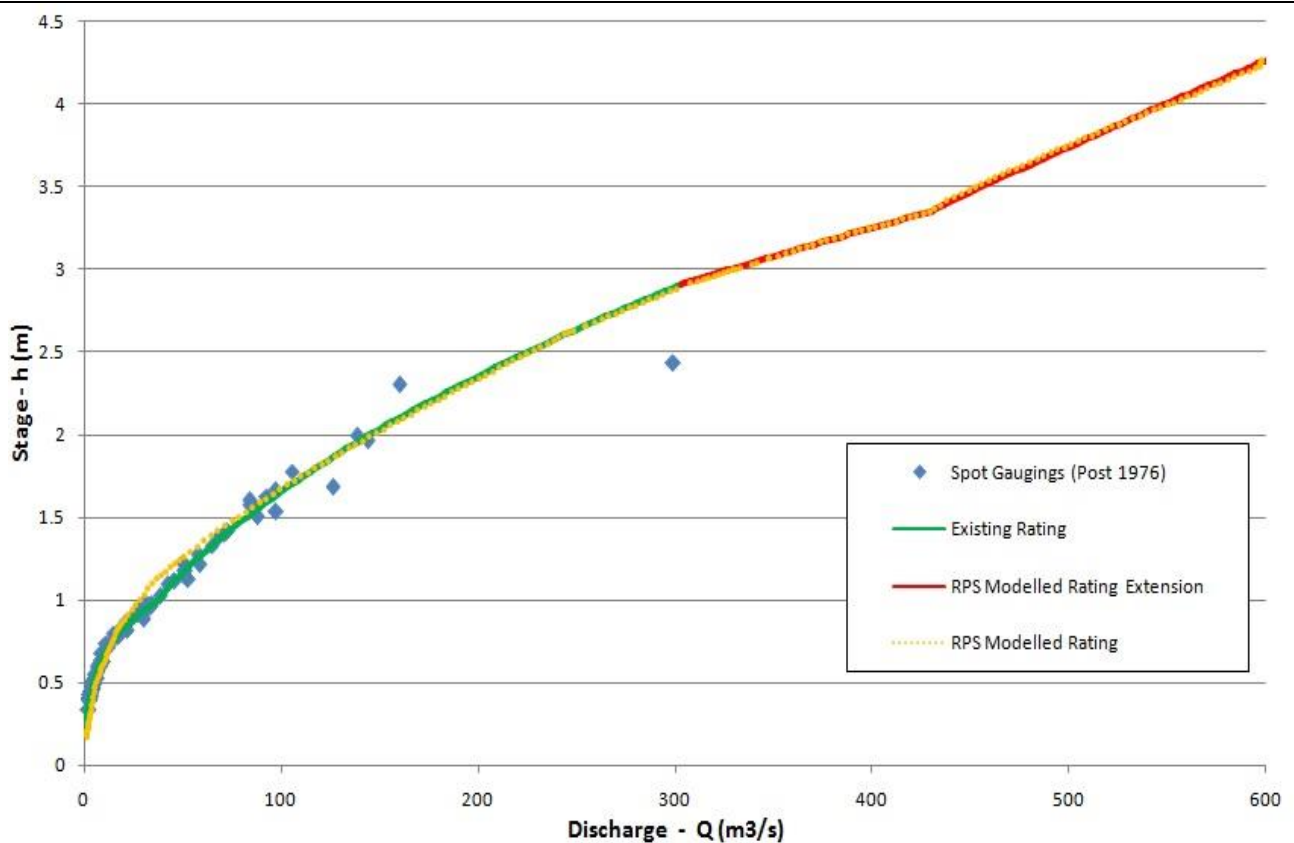


Figure 4.8.26: Navan Weir GS (07009) - Comparison of the OPW and Modelled Rating Curves

Figure 4.8.26 shows that the RPS rating curve acts as a 'line of best fit' through the post-1976 gaugings, with some gaugings lying above and other gaugings lying below the rating curve due to a fair amount of scatter in the higher range of spot gaugings. The existing rating equation extends to a stage height of 2.908m but is listed as of poor quality by the OPW past 1.658m due to a lack of spot gaugings past this level. However, the rating curve is validated by the modelled stage discharge relationship and so the existing equations are retained up to 2.908m.

The sensitivity of hydraulic influence of the weir structure was assessed by adjusting the default co-efficients and was found to be significant. However, as the modelled rating relationship using the default co-efficients (Figure 4.8.26) was found to be well calibrated against both the post-1976 spot gaugings and the existing A1 rating curve then it is assumed that the modelled weir coefficients are accurate. Further details on the rating review can be found in Appendix C of the Hydrology Report.

(b) Liscartan (07010)

The Liscartan gauging station (07010 – OPW) has three classification periods under FSU, A1 pre 1972, A2 from 1972 – 1982 and A2 from 1982 to date. This would suggest that for all three periods the station should be reliable up to and above Q_{med} . The most recent Q_{med} value is $68.36 \text{ m}^3/\text{s}$. For the purposes of the Eastern CFRAM study, spot gauges and flows from the Liscartan gauge based on the latest OPW rating (i.e. Rating Curve 13) up to the highest gauged flow ($64.7 \text{ m}^3/\text{s}$) for the period after the site was rebuilt (07/12/1986) are being considered. This gauging station is subject to a rating review.

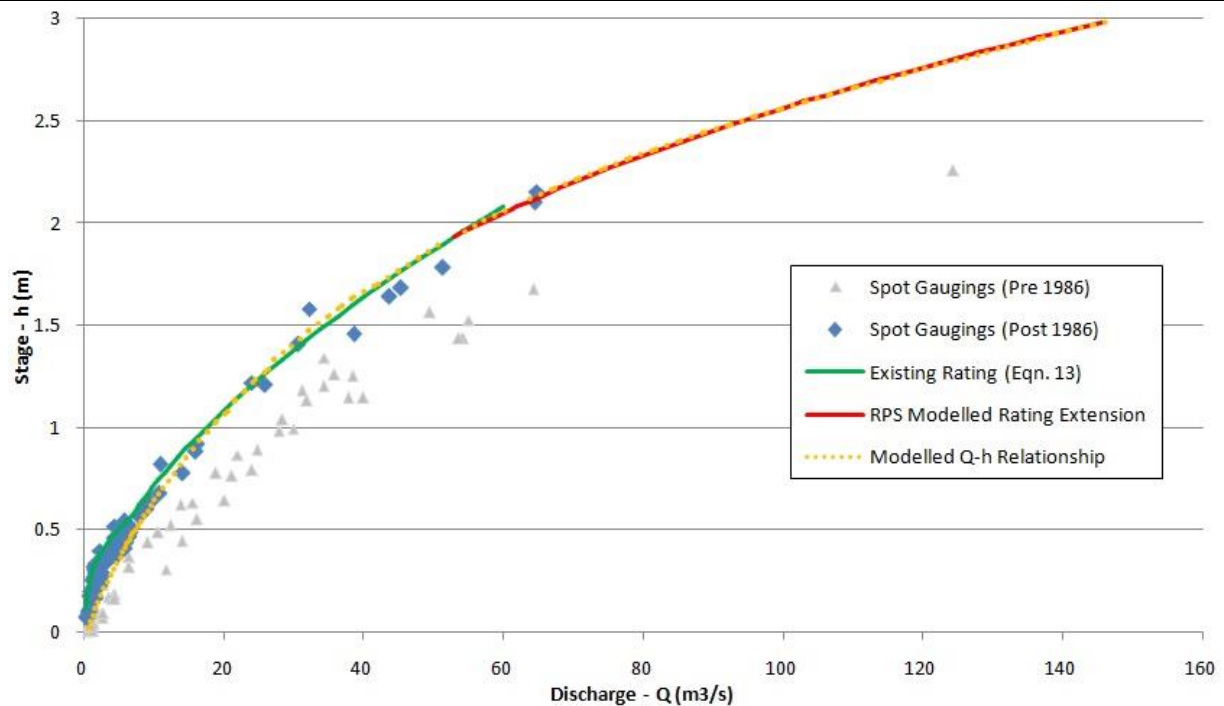


Figure 4.8.27: Liscartan GS (07010) - Comparison of the OPW and Modelled Rating Curves

Figure 4.8.27 shows that the RPS rating curve acts as a 'line of best fit' through the post-1986 gaugings, with some gaugings lying above and other gaugings lying below the rating curve. Further details on the rating review can be found in Appendix C of the Hydrology Report.

(c) Ballinter Bridge (07041)

The Ballinter gauging station (07041 – EPA) has been given an FSU classification of its rating of A2 although it has a relatively short record period (1997 – present) and as such there is fair confidence in its Q_{med} value of 161.0 m³/s. The comparison of the modelled Q-h relationship and the rating curve shows that they are within 200 mm of each other (as shown in Figure 4.8.28).

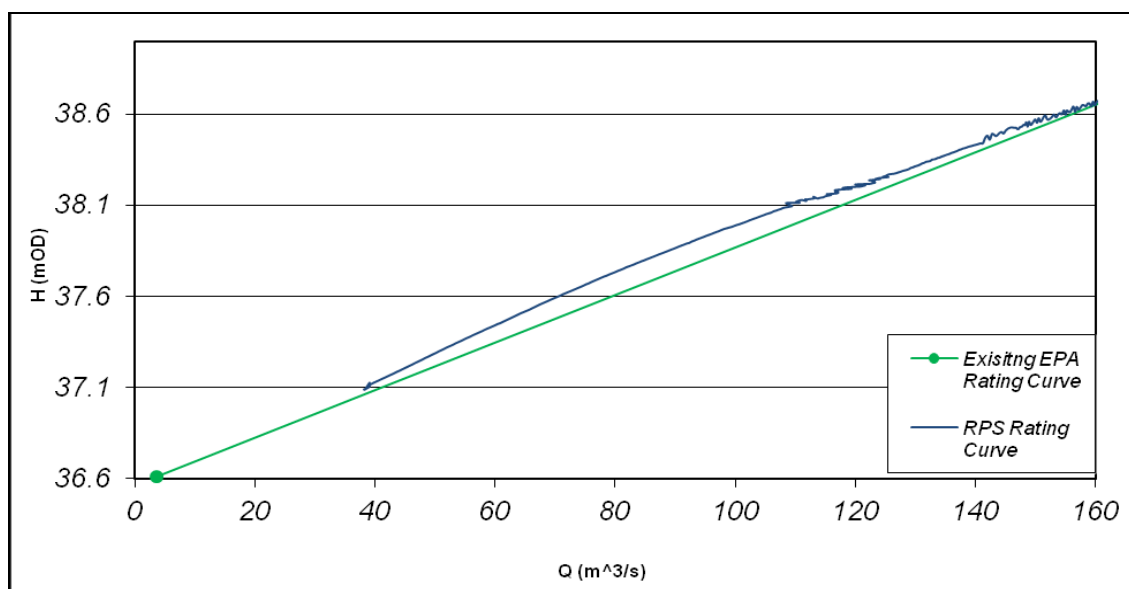


Figure 4.8.28: Ballinter Bridge GS (07041) - Comparison of the EPA and Modelled Rating Curves

(4) Validation with MIKE NAM:**(a) Navan Weir (07009)**

As outlined in Section 3.5.3, NAM modelling provides a further layer of simulated hydrometric data for calibration of the hydraulic models. Flood events which are outside the continuous flow record period at a gauge are available through the simulated time series flow data where this modelling was undertaken. At the Navan Weir gauging station, there is observed data available for four of the five flood events listed in Table 4.8.3. There is no observed data for the flood event which occurred on 20/11/09 – the NAM modelling undertaken has predicted that the peak discharge reached during this event was $219 \text{ m}^3/\text{s}$, as discussed in Section 4.8.5 (1a).

The difference between the modelled peak water level achieved at the peak discharge estimated by MIKE NAM and the observed peak water level (where available) is provided in Table 4.8.3. The difference is less than 500mm for each event. This comparison is provided for information purposes only, and has not been used during the calibration or validation of the model. Model calibration is based on observed data, which takes precedence over the data provided from the NAM hydrological model. Further details on the NAM model are provided in the Hydrology Report.

(b) Liscartan (07010)

At the Liscartan gauging station, there is observed data available for all five flood events listed in Table 4.8.4. As discussed in Section 4.8.5 (1a), the observed data recorded for the flood event which occurred on 20/11/09 has been confirmed as inaccurate, and so this information has not been used during model calibration. The NAM modelling undertaken has predicted that the peak discharge reached during this event was $133 \text{ m}^3/\text{s}$, as discussed in Section 4.8.5 (1a).

The difference between the modelled peak water level achieved at the peak discharge estimated by MIKE NAM and the observed peak water level (where available) is provided in Table 4.8.4. The difference is less than 530mm for each event. This comparison is provided for information purposes only, and has not been used during the calibration or validation of the model. Model calibration is based on observed data, which takes precedence over the data provided from the NAM hydrological model. Further details on the NAM model are provided in the Hydrology Report.

Table 4.8.3: Comparison of Observed, Hydrologically (NAM) and Hydraulically Modelled Data for Flood Events at the Navan Weir Gauging Station (07009)

	Observed Peak		MIKE FLOOD Simulated Peak at Observed WL	MIKE NAM Simulated Peak	MIKE FLOOD Simulated Peak at NAM Discharge	Water Level Difference
Flood Event	Water Level (mOD)	Discharge (m ³ /s)	Discharge (m ³ /s)	Discharge (m ³ /s)	Water Level (mOD)	(m)
20/11/2009	-	-	-	219.106	32.77	-
17/08/2008	32.865	239.155	235.035	275.915	33.07	-0.21
15/11/2002	33.130	289.976	287.293	293.426	33.16	-0.03
07/11/2000	33.094	282.490	276.800	199.978	32.60	0.49
12/06/1993	32.745	217.841	214.379	318.664	33.27	-0.53

Table 4.8.4: Comparison of Observed, Hydrologically (NAM) and Hydraulically Modelled Data for Flood Events at the Liscartan Gauging Station (07010)

	Observed Peak		MIKE FLOOD Simulated Peak at Observed WL	MIKE NAM Simulated Peak	MIKE FLOOD Simulated Peak at NAM Discharge	Water Level Difference
Flood Event	Water Level (mOD)	Discharge (m ³ /s)	Discharge (m ³ /s)	Discharge (m ³ /s)	Water Level (mOD)	(m)
20/11/2009	39.44*	170.024*	-	133.202	-	-
17/08/2008	38.303	93.635	120.18	149.590	38.6	-0.30
15/11/2002	37.975	75.190	88.85	98.913	38.1	-0.13
07/11/2000	38.052	79.404	96.06	94.988	38.0	0.05
12/06/1993	37.774	68.682	71.00	107.222	38.3	-0.53

* Data not used during model calibration due to being inaccurate

(5) Comparison of Flows:**Table 4.8.5: Comparison of Modelled Flows with Check Flows**

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m ³ /s)	Model Flow (m ³ /s)	Diff (%)
BAILIS TRIBUTARY - BOYNE 2951.42	10%	2.25	2.07	-8.00
07_1487_7	1%	4.21	3.14	-25.42
	0.1%	7.63	5.27	-30.93
RIVER BLACKWATER (KELLS) 1466.57	10%	105.30	105.75	0.43
07010	1%	148.82	147.19	-1.10
	0.1%	203.51	203.67	0.08
RIVER BLACKWATER (KELLS) 4432.7	10%	108.2	106.94	-1.16
07037	1%	152.92	150.36	-1.67
	0.1%	209.13	227.82	8.94
TRIM ROAD TRIBUTARY 1 - BOYNE 3194.59	10%	1.50	1.49	-0.66
07_1188_5_RPS	1%	2.81	2.29	-18.51
	0.1%	5.09	4.21	-17.29
ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE 1289.73	10%	0.47	0.42	-10.64
07_1851_1	1%	0.88	0.82	-6.82
	0.1%	1.59	1.50	-5.66
RIVER BOYNE 59002.4	10%	249.75	249.59	-0.06
07041_RPS	1%	354.38	354.27	-0.03
	0.1%	487.48	487.34	-0.03
RIVER BOYNE 63955	10%	253.66	266.01	4.87
07009_RPS	1%	359.94	384.96	6.95
	0.1%	495.12	532.83	7.62
RIVER BOYNE 70426.9	10%	345.93	332.80	-3.80
07_1490_1_RPS	1%	490.86	472.75	-3.69
	0.1%	675.21	652.81	-3.32

Table 4.8.5 provides details of the flow in the model, flows generated during the hydrology flow estimation and the percentage difference at HEP Check-Points.

On the River Blackwater, the modelled flows correlate very well with the check flows at both gauging stations (07010 and 07037) with the difference being less than 10% for all return periods.

On the River Boyne, the modelled flows at the Navan Weir Gauging Station (07009_RPS) which is located upstream of Navan town, correlate well with the check flows for all return periods (less than 8% difference). The HEP at the downstream end of the model on the Boyne (07_1490_1_RPS) correlates very well, with modelled flows being less than 4% lower than the check flows for all return periods.

A comparison between modelled flows and check flows at the confluence of the Boyne tributaries shows that the flows are lower at 07_1188_5_RPS (confluence of Trim Road Tributary 1 and Boyne) for the 1%

and 0.1% AEP events. As discussed within the report, there are two critical structures on this watercourse resulting in attenuation of flood flows which will contribute to modelling flows being less than check flows. The Athlumney House Tributary 4 modelled flows correlate well with the check flows for all return periods (less than 11% difference). The Bailis Tributary modelled flows are up to 31% lower than the check flows. This is due to differences in the catchment descriptors between the catchment at the upstream extent of the modelled watercourse and the downstream extent. This results in a significant difference between the time to peak of the inflow and lateral inflow hydrographs, resulting in a reduced peak flow compared to the check flow at the 07_1487_7 check point.

Overall, there is good correlation between the modelled flows and the check flows for the Navan model. The approach undertaken has been outlined above and is believed to achieve the best representation of the modelled watercourses within the Navan area.

(6) Other Information:

(a) *Leighsbridge Stream Culvert Drainage Improvement Scheme, January 2003* - The Leighsbridge Stream is not a watercourse included within this model.

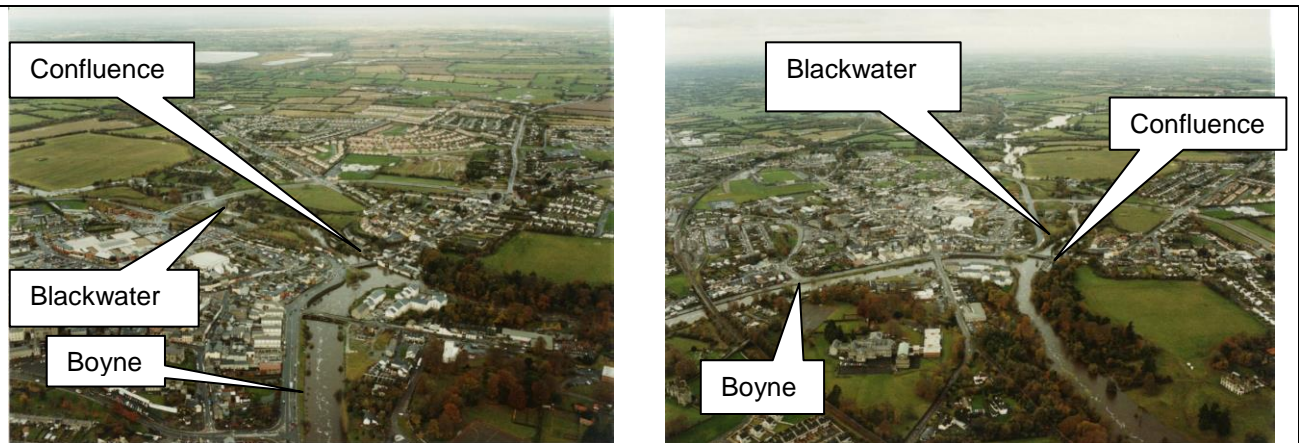
(b) *Aerial Photographs of the flood event on 7th November 2000, www.floodmaps.ie* - the photographs (see Figure 4.8.29) have been taken after the peak of the flood has passed (at approximately 11:00 am on 07/11/00 - the peak water level was recorded between 22:30 on 06/11/00 and 01:45 on 07/11/00 depending on location). The peak flow at 11:00 am on 07/11/00 at each gauging station is: 256.282 m³/s (07009), 75.233 m³/s (07010) and 217.153 m³/s (07041).



(i) River Blackwater at Kells Road



(ii) River Blackwater at Blackwater Park



(iii) Boyne and Blackwater Confluence (looking north) (iv) Boyne and Blackwater Confluence (north-west)

Figure 4.8.29: Aerial Photographs following the November 2000 Flood Event in Navan

A comparison has been made between the modelled flood extents (when modelled flows equal the recorded flows at 11:00 am on 07/11/00) and the recorded flood extents. The railway line visible in Figure 4.8.29 Photographs (i) and (ii) is approximately 1.4 kilometres downstream of the 07010 gauging station. There is some out-of-bank flooding visible in both photographs. Figure 4.8.30 shows the modelled flood extents when the discharge at 07010 is $74.574 \text{ m}^3/\text{s}$. This shows that the modelled flood extents are very similar to those shown in the photographs. The exception is immediately downstream of the railway embankment where there is a short reach of flooding shown in the photograph, but not by the model. Given the close correlation between the design flows and modelled flows at the check point downstream of this location (07037 - see Section 4.8.5(5)), and the uncertainty in when the photographs were taken, no adjustments were made to the model.

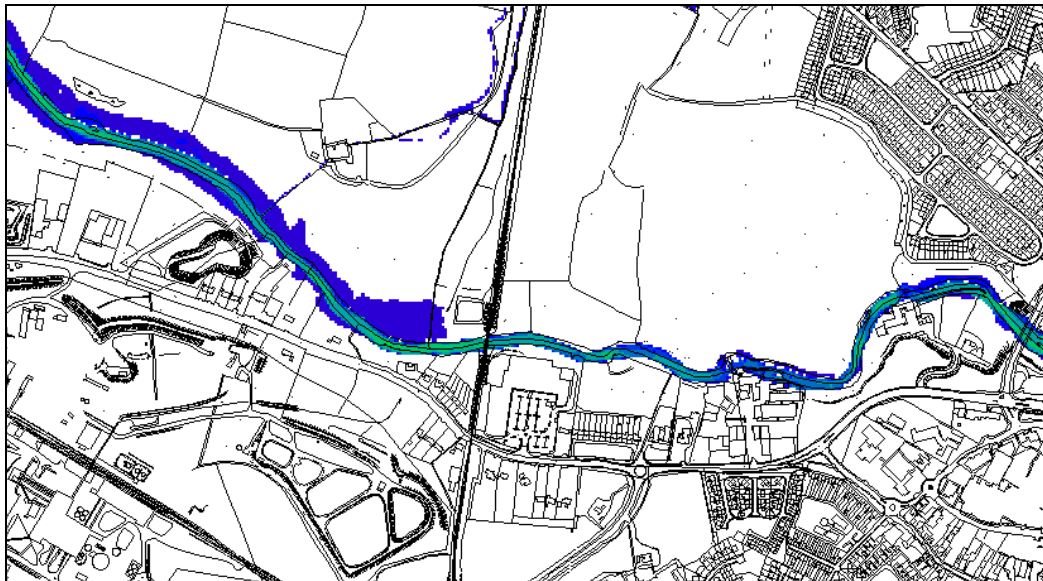


Figure 4.8.30: Modelled Flood Extents (at approximate time of Photos (i) and (ii) along the River Blackwater)

Photographs (iii) and (iv) do not appear to show any out of bank flooding along the Boyne, with the exception of a short reach along the left bank after the Blackwater confluence. The Boyne and Blackwater confluence is approximately 1.4 kilometres downstream of the 07009 gauging station. Figure 4.8.31 shows

the modelled flood extents when the discharge at 07009 is $254.93 \text{ m}^3/\text{s}$ and 07041 is $229.39 \text{ m}^3/\text{s}$. This also shows little out of bank flooding along the Boyne, with the exception of short reaches along both banks downstream of the Blackwater confluence. Generally, the modelled flood extents represent the flood extents evident on the photographs. Given the close correlation between the design flows and modelled flows at the check points upstream and downstream of this location (07009_RPS and 07_1490_1_RPS respectively - see Section 4.8.5(5)), and the uncertainty in when the photographs were taken, no adjustments were made to the model.

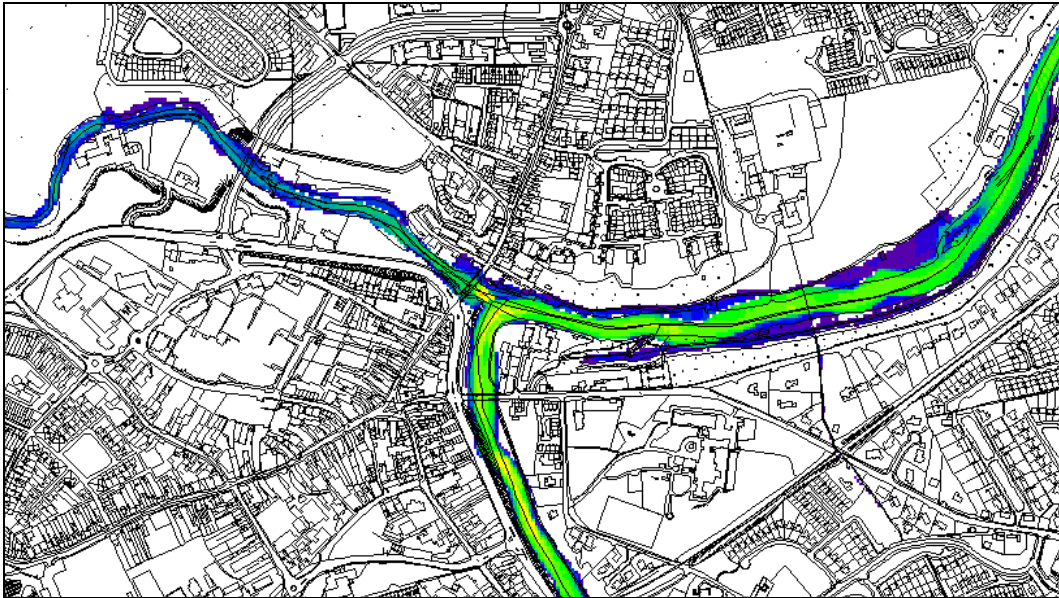


Figure 4.8.31: Modelled Flood Extents (at approximate time of Photos (iii) and (iv) along the River Boyne)

4.8.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

- (a) It is assumed that there are no alternative flow paths allowing flood water to reach Kilcarn Court (on the downstream side of the embankment).
- (b) Section 4.8.6 (3) lists the skewed weirs located within the model extents. It is a limitation of the software that it does not enable the explicit modelling of skewed weirs. DHI guidance has been adopted in order to incorporate these structures within the model in a representative way.
- (c) The weir recorded on the Old Balreask tributary at 0715_00013W was not included as it is lower than the upstream cross-section and is not considered to be hydraulically significant.
- (d) The culvert on the Athlumney Tributary (Ch 284) has been excluded from the model as it is not considered to be hydraulically significant and removal of the structure facilitates model stability.
- (e) There is a raised stone wall in good condition (see Figure 4.8.32) which was initially identified as a defence by the surveyors. It is not reached by flood waters during any of the modelled return periods and so does not have an associated benefitting area. RPS have assumed that this structure is not a defence.
- (f) Defence 1 (Section 4.8.3 (4)) is assumed to be an effective defence - the defence asset condition survey classifies it as being in good condition.

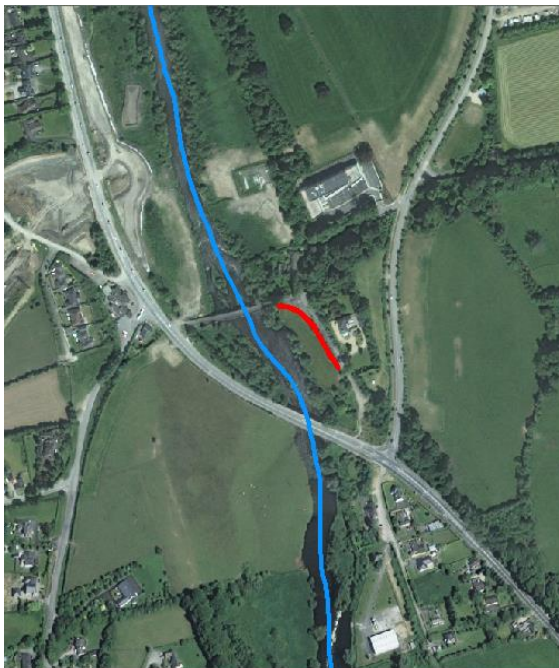


Figure 4.8.32: Location and View of Wall (identified as a defence by the survey team)

(2) Hydraulic Model Limitations and Parameters:

- (a) A grid resolution of 5 metres has been selected.
- (b) The Cross-section and Network files are identical for all design run simulations. The parameters within the HD parameter file are also identical.

(c) A Parameter file has been selected in the 1D model component during all design runs. A water depth of 0.2 m has been selected for the Initial Conditions in the HD Parameters file for each model.

(d) Where the watercourses are modelled as 1D-2D, markers have been located at the top of the bank of the cross-sections within the 1D model to denote the point at which water is transferred from the 1D model to the 2D domain. Where the watercourses are modelled as 1D only, the floodplain is contained within the cross-section in the 1D model, as the markers are placed at the cross-section extents.

(e) During model construction, some instabilities were detected. A review of the model schematisation was conducted in an attempt to remove or reduce the impact of these instabilities. This involved minor adjustments (< 10 m) to cross-section locations around structures in order to improve model resolution. Once this process had been completed, remaining instabilities were caused during the transfer of flow from MIKE 11 to MIKE 21. A review of the lateral links was conducted to ensure they were appropriate. After this process was completed, a review of the lateral link parameters was conducted to resolve remaining instabilities, resulting in the selection of the parameters identified in the table below.

(f) A weir structure was removed from the .nwk file at Ch 3249 on Trim Road Tributary 1, as it was not hydraulically significant and was a source of instability within the model. The cross-section along the crest of the weir remains as part of the .xns file.

(g) There are some minor instabilities within the model, which result in minor fluctuations (< 50mm) in water level. Following a review of the model, it was not possible to eradicate these instabilities completely, so their significance and impact on model results was reviewed. The instabilities do not affect the peak discharge or water level, and no erroneous out-of-bank flooding is caused - they do not have a significant impact on model results. An example is shown in Figure 4.8.33 during the 0.1% AEP event at Ch 58460 along the River Boyne.

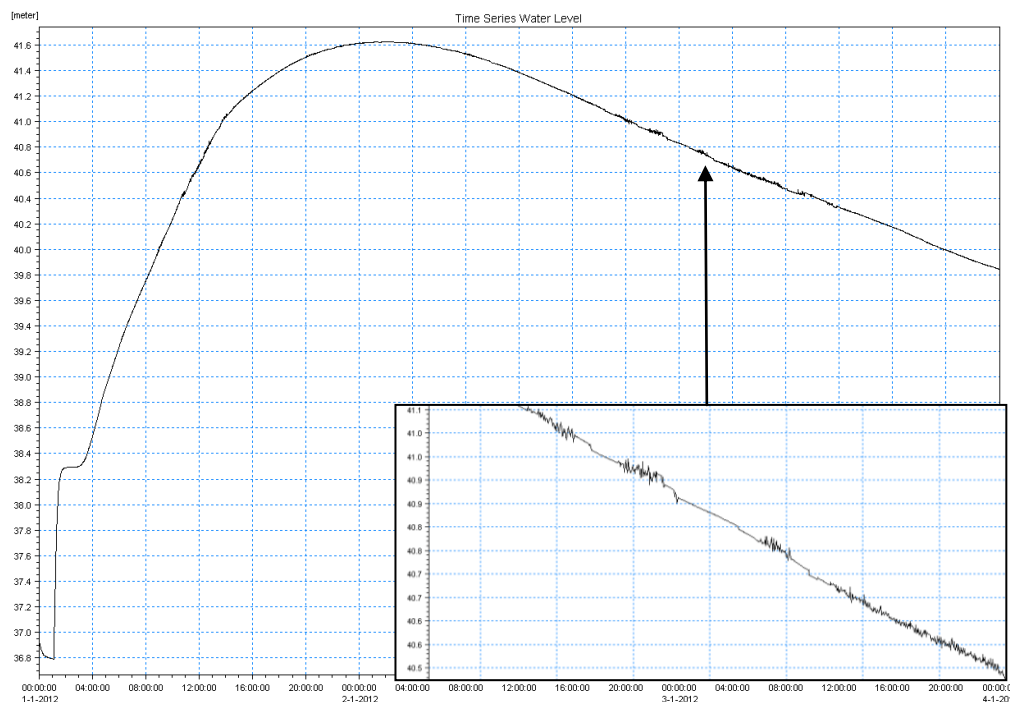


Figure 4.8.33: Water Level Profile for 0.1% AEP Design Run (at Ch 58460 along the River Boyne)

MIKE 11	
Timestep (seconds)	1
Wave Approximation	High Order Fully Dynamic
Delta	0.85
MIKE 21	
Timestep (seconds)	1
Drying / Flooding depths (metres)	0.02 / 0.03
Eddy Viscosity (and type)	0.5 (Flux Based)
MIKE FLOOD	
Link Exponential Smoothing Factor (where non-default value used)	0.6 for River Boyne: 61989, 62767, 63471, 61989, 62580, 63471 0.6 for Athlumney House Tributary 1: 1233 0.8 for all remaining links
Lateral Link Depth Tolerance (m) (where non-default value used)	0.2 for Athlumney House Tributary 1: 1233
(3) Design Event Runs & Hydraulic Model Handover Notes:	
<p>(a) The topographical survey information provided by Murphys Surveys for HA07 has been supplemented with topographical survey information recorded for the purpose of the Gauging Station Rating Reviews (in order to facilitate model calibration at the gauging station location).</p> <p>(b) Post-processing of the flood extents to remove those bridges and culverts which do not flood for the modelled return period from the flood extent (as discussed in Chapter 3). There was no further post-processing of the flood extents required.</p> <p>(c) There are some weirs along the River Boyne which are skewed. As MIKE does not provide an option to explicitly model skewed weirs, the approach to modelling these structures follows one of the options detailed in DHI guidance (and as described in Chapter 3). This means that the weir structure will be perpendicular to the river centre line, fitting within the river channel with the structure coefficients selected to represent the flow over the weir.</p> <p>(d) On the River Boyne, there is flood water a short distance beyond the top of bank but this does not generally affect any properties. The exceptions are the Dublin Road and Academy Street areas, which flood during the 1% and 0.1% AEP events due to insufficient capacity in the channel. There is also out-</p>	

of-bank flooding affecting properties at the Blackwater confluence during the 1% AEP event (on the right bank) and flooding affecting a rural part of the Blackcastle area (see Appendix H).

(e) On the Trim Road Tributary 1, flooding occurs upstream of the long culvert passing under Balreask Manor (see Section 4.8.3) but is restricted by the recently constructed flood embankment for the 10% and 1% AEP events. Further downstream, the culvert passing under the embankment at Kilcarn Court (see Section 4.8.3) causes a restriction resulting in the attenuation of flood water for all return periods at this location.

(f) On the River Blackwater, there is extensive flooding of agricultural land in the Liscartan area. There is also out of bank flooding along the watercourse from this location to downstream of the railway line (near to Blackwater Park) but this does not generally affect any properties.

(g) On the Abbeylands tributary, there is flooding during the 1% AEP and 0.1% AEP flood event upstream of the railway line, due to insufficient capacity in the culvert under the embankment. There is also associated flooding on the Ratholdren Road, adjacent open land and single dwellings during this event due to insufficient capacity of the culvert under the road.

(h) There is some flooding of properties from the Bailis tributary from the 0.1% AEP event, due to insufficient capacity in the channel and culverts (see Section 4.8.3).

(i) There is little or no flood risk associated with the Old Balreask, Athlumney and Athlumney House tributaries of the Boyne, at any of the modelled return periods.

(j) Section 4.8.5 (Part 1) identifies a number of areas which have been subject to flooding, but are not located near to a watercourse. The assumption has been made that the flooding is coming from an alternative source (i.e. not from the modelled watercourses). The calibration achieved at the gauging stations (Section 4.8.5 Part (3)) is deemed as very good and so gives confidence that the model has been successfully calibrated and validated.

(k) The simulation period for the 1% AEP and 0.1% AEP models is 4 days, to allow the peak of the flood to pass through the model. The simulation period for the 10% AEP model was able to be reduced to 3 days, whilst still recording peak flood levels, flows and extents.

(k) In conclusion, the model is considered to be performing well for design event simulation and is supported by historic and hydrometric information. The modelled flood extents cover areas which have a history of flooding and modelled flows correlate well with estimated flows providing confidence that the model adequately represents the hydraulics of the modelled watercourses.

(4) Hydraulic Model Deliverables:

Please see Appendix H for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Stephen Patterson
Model Reviewed by:	Malcolm Brian
Model Approved by:	Grace Glasgow

4 HYDRAULIC MODEL DETAILS

4.9 TRIM MODEL

Name	Local Authority	Unique ID	SSA	Status	Date
Trim	Meath	70041	AFA	Final	08/05/2017

4.9.1 General Hydraulic Model Information

(1) Introduction:

Following a review of historic flooding and the extents of flood risk determined during the PFRA, the Eastern CFRAM Flood Risk Review (IBE0600Rp0025_Flood Risk Review) identified Trim, in the Boyne catchment, as an AFA for fluvial flooding.

The Edenderry, Longwood, Ballivor and Athboy AFA models are all located immediately upstream of the Trim model on the Boyne, Longwood Stream/Blackwater River, Ballivor/Stonyford Rivers and Athboy/Tremblestown Rivers respectively. The Trim model is connected to the Navan model at its downstream extent.

The Trim model includes a large stretch of the Boyne from Longwood to just upstream of Navan, a number of large tributary inflows and also a large number of smaller tributaries to be modelled in the vicinity of the Trim AFA extents and as such is a large and complex model. The key tributaries are Boyce Town River, Knightsbrook River, Newtown Bridge Stream, Friars Park Stream and Butter Stream (see Figure 4.9.1 and Figure 4.9.2).

The main channel of the Boyne is well gauged in this portion of the modelled watercourses with a gauging station at the upstream extents, downstream extents and one in the middle of the AFA extents (i.e. in Trim itself). The OPW gauging station called Boyne Aqueduct (07007) is located at the upstream boundary of Model 4, just west of Longwood and on the Boyne main channel where the Royal Canal traverses the river. This gauging station has three classification periods under FSU, A1 pre 1962, A1 from 1962 – 1973 and B from 1979 to date. This would suggest that, post arterial drainage scheme, there is less confidence in the rating but for all three periods the station should be reliable up to Q_{med} . The values for the three periods of Q_{med} are 37.15, 31.04 and 35.70 m³/s, respectively. The OPW gauging station at Trim (07005) has been given an FSU classification of its rating of A1, for the entire period of the rating, and as such there is good confidence in the Q_{med} value of 104.4 m³/s. The gauging station at the downstream extents of the model called Ballinter Bridge (07041 – EPA) has been given an FSU classification of its rating of A2 although it has a relatively short record period (1997 – present) but again there is confidence in the Q_{med} value of 161.0 m³/s.

In addition to the fluvial flood risk posed by the main channel of the River Boyne there are also a number of tributaries affecting the Trim AFA including significant tributaries namely the Knightsbrook and Boycetown Rivers. There are also a number of smaller tributaries which run through the town which pose a significant flood risk. None of these tributaries watercourses are gauged but some are significant enough

or pose such a significant flood risk that rainfall run-off models have been constructed such as to supplement the ungauged estimates of Q_{med} .

Four rainfall run-off models have been developed for Model 4 (Trim). Although the gauged flow data is of high quality, three NAM models have been developed, one at each of the gauging stations to achieve calibration of the NAM models, simulate an extended AMAX series for the duration of the rainfall record and to fill in any gaps in the records which may have been missed. A NAM model has also been developed for the River Deel tributary where good calibration could be achieved. Where index flood flows have not been derived from NAM modelling the values are derived based on the IH124 method for small catchments (all catchments < 25km²) or the FSU method and adjusted against the calibrated NAM models where appropriate.

The Boyne reach from section 0701_06099 to section 0701_05380X, along with the Boyce Town River, Boyce Town River Trib, Knightsbrook River, Newtown Bridge Stream, Friars Park Stream, Friars Park Stream Trib, Butter Stream and Butter Stream Trib have been designated as HPWs and have therefore been modelled as 1D-2D using the MIKE suite of software. Sections 0701_08447 to 0701_06158X and 0701_05356 to 0701_04544 at the upstream and downstream ends respectively in this model on the River Boyne have been modelled as 1D. Channel markers have been located at the right and left banks of all cross sections. Flow within these markers is calculated by the 1D model component; however when the water level rises sufficiently to meet the bank markers flow can enter the 2D domain which represents the floodplain, where the 1D model has been linked to the 2D model.

(2) Model Reference:	HA07_TRIM4
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(3) AFAs included in the model:	TRIM
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(4) Primary Watercourses / Water Bodies (including local names/modelled names):
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Reach ID	Name
0701	RIVER BOYNE
0720	BOYCE TOWN RIVER (TRIM TRIBUTARY 1 - BOYNE)
0721	BOYCE TOWN RIVER TRIB (TRIM TRIBUATRY 2 - BOYNE)
0722	KNIGHTSBROOK RIVER (TRIM TRIBUTARY 3 - BOYNE)
0723	NEWTOWN BRIDGE STREAM (BLACKFRIARY TRIBUTARY - BOYNE)
0724	FRIARS PARK STREAM (STONEHALL TRIBUTARY 1 – BOYNE)
0725	FRIARS PARK STREAM TRIB (STONEHALL TRIBUTARY 2 – BOYNE)
0726	BUTTER STREAM (ATHBOY ROAD TRIBUTARY 1 – BOYNE)
0727	BUTTER STREAM TRIB (ATHBOY ROAD TRIBUTARY 2 – BOYNE)

(5) Software Type (and version):		
(a) 1D Domain: MIKE 11 (2011)	(b) 2D Domain: MIKE 21 – Rectangular Mesh (2011)	(c) Other model elements: MIKE FLOOD (2011)

4.9.2 Hydraulic Model Schematisation

(1) Map of Model Extents:
<p>Figure 4.9.1 and Figure 4.9.2 illustrate the extent of the modelled catchment, river centre line, HEP locations and AFA extents as applicable. The Trim Catchment has been designated largely as a HPW. The Trim catchment contains 8 Upstream Limit HEPs, 1 Intermediate HEP and 17 Trib HEPs. The catchment also contains 3 Gauging Station HEPs, 1 of which is located at the upstream model extent on the River Boyne and another of which is located at the downstream model extent on the River Boyne.</p>

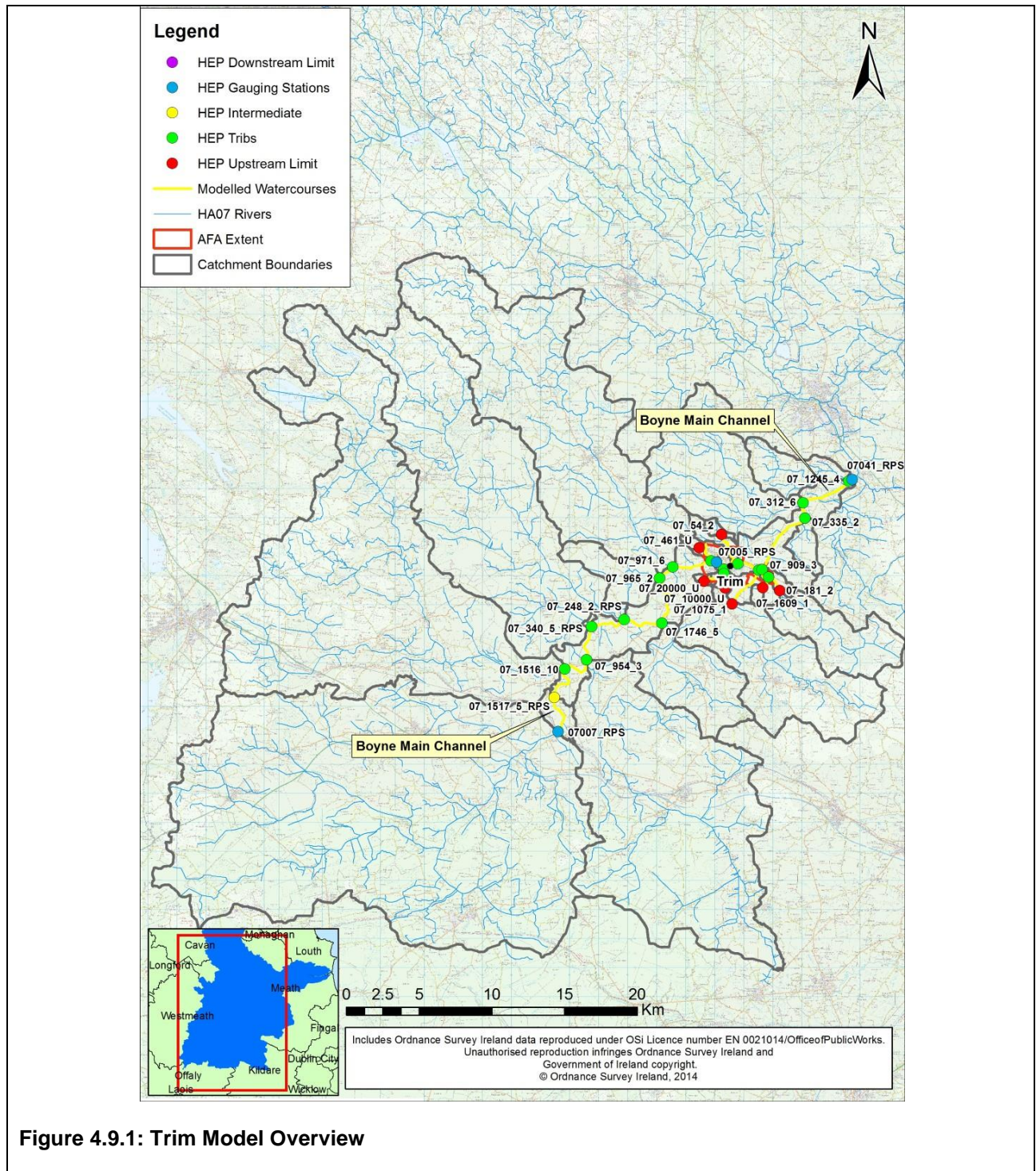


Figure 4.9.1: Trim Model Overview

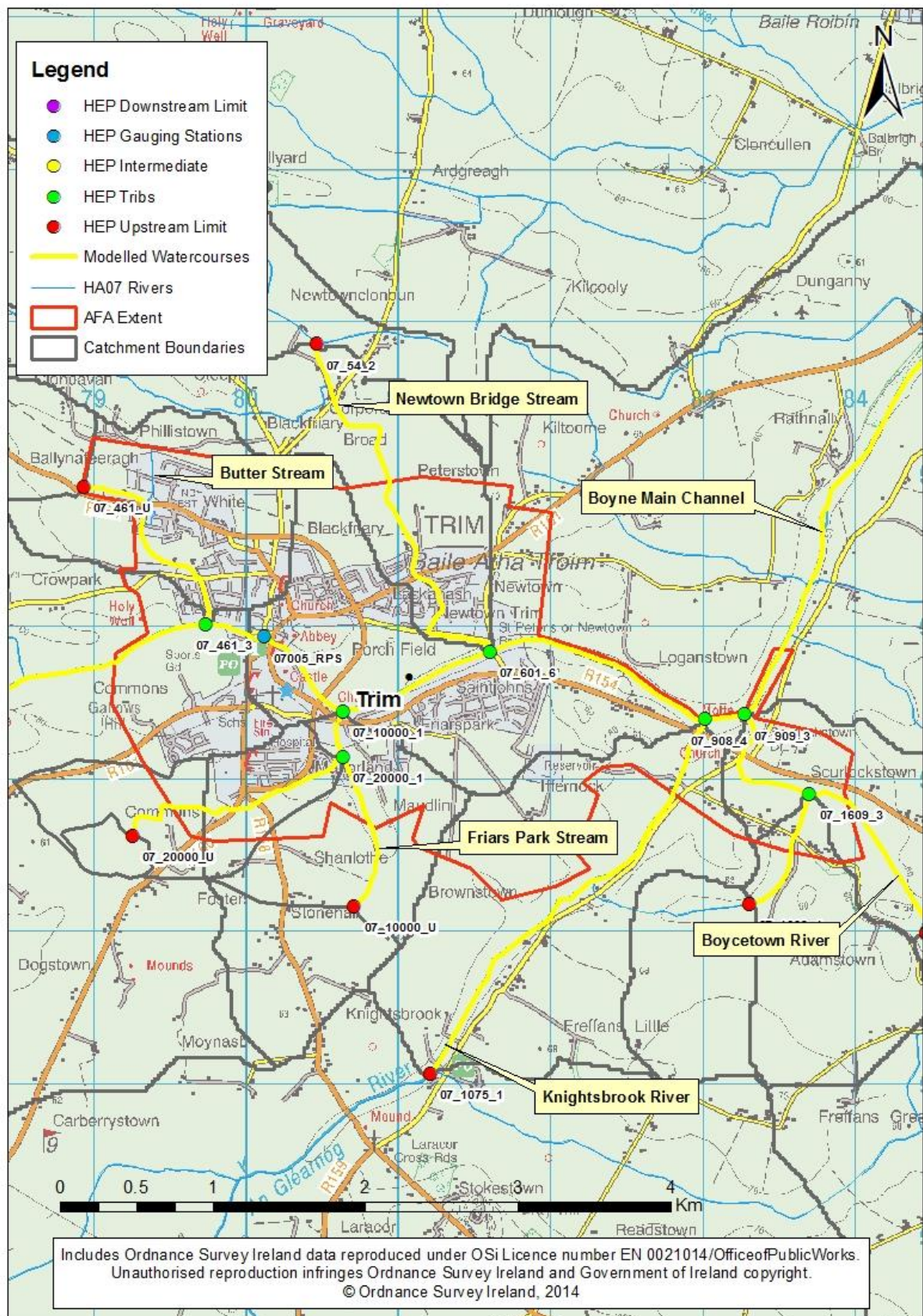


Figure 4.9.2: Trim AFA Extent

(2) x-y Coordinates of River (Upstream extent):**Table 4.9.1: x-y Coordinates of River**

River Name		x	y
0701	RIVER BOYNE	269211.24	245272.51
0720	BOYCE TOWN RIVER	284460.28	254992.83
0721	BOYCE TOWN RIVER TRIB	283292.20	255183.58
0722	KNIGHTSBROOK RIVER	281205.64	254069.07
0723	NEWTOWN BRIDGE STREAM	280574.05	258493.65
0724	FRIARS PARK STREAM	280690.98	255147.56
0725	FRIARS PARK STREAM TRIB	279254.66	255630.74
0726	BUTTER STREAM	279306.72	257607.92
0727	BUTTER STREAM TRIB	278789.10	257958.68

(3) Total Modelled Watercourse Length:

52.3 kilometres (approx)

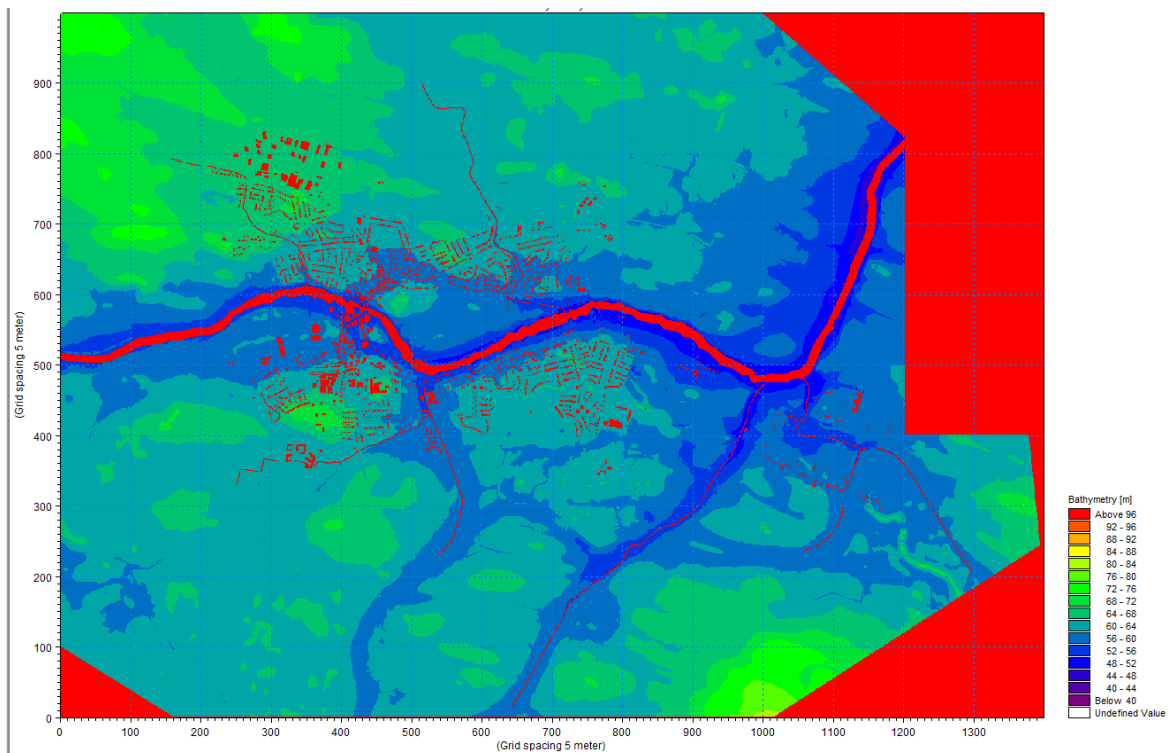
(4) 1D Domain only Watercourse Length:31.1 km
(approx.)**(5) 1D-2D Domain****Watercourse Length:**21.2 km
(approx.)**(6) 2D Domain Mesh Type / Resolution / Area:**Rectangular / 5 metres/ 35km²**(7) 2D Domain Model Extent:****Figure 4.9.3: Extent of Trim 2d Model Domain**

Figure 4.9.3 represents the modelled extents and the general topography of the catchment within the 2D

model domain. The river centre-line is shown in red with red areas also representing blocked cells i.e. river centre lines, buildings or the area beyond the 2D model domain.

The ground elevation (based on LiDAR data used to generate 2D rectangular mesh) is shown to provide an overview of the modelled area topography. Changes in the vertical scale of this map are outlined by the index, all levels have been set to OD Malin. There was no further post processing required of the data contained within the mesh.

Figure 4.9.4 and Figure 4.9.5 provide an overview drawing of the model schematisation. Figure 4.9.6 to Figure 4.9.9 show detailed views. The overview diagram covers the model extents, showing the surveyed cross-section locations, AFA boundary and river centre line. It also shows the area covered by the 2D model domain. The detailed areas are provided where there is the most significant risk of flooding. These diagrams include the surveyed cross-section locations, AFA boundary and river centre. They also show the location of the critical structures as discussed in Section 4.9.3.1, along with the location and extent of the links between the 1D and 2D models. For clarity in viewing cross-section locations, the model schematisation diagrams show the full extent of the surveyed cross-sections. Note that the 1D model considers only the cross-section between the 1D-2D links.

The upstream extent of the model on the River Boyne commences approximately 150m upstream of the first structure, a bridge at cross-section 0701_08431D located at chainage 20198.033m. At the downstream end of the model, mapping occurs to chainage 56452m as flood mapping downstream of this location is covered by the Navan model.

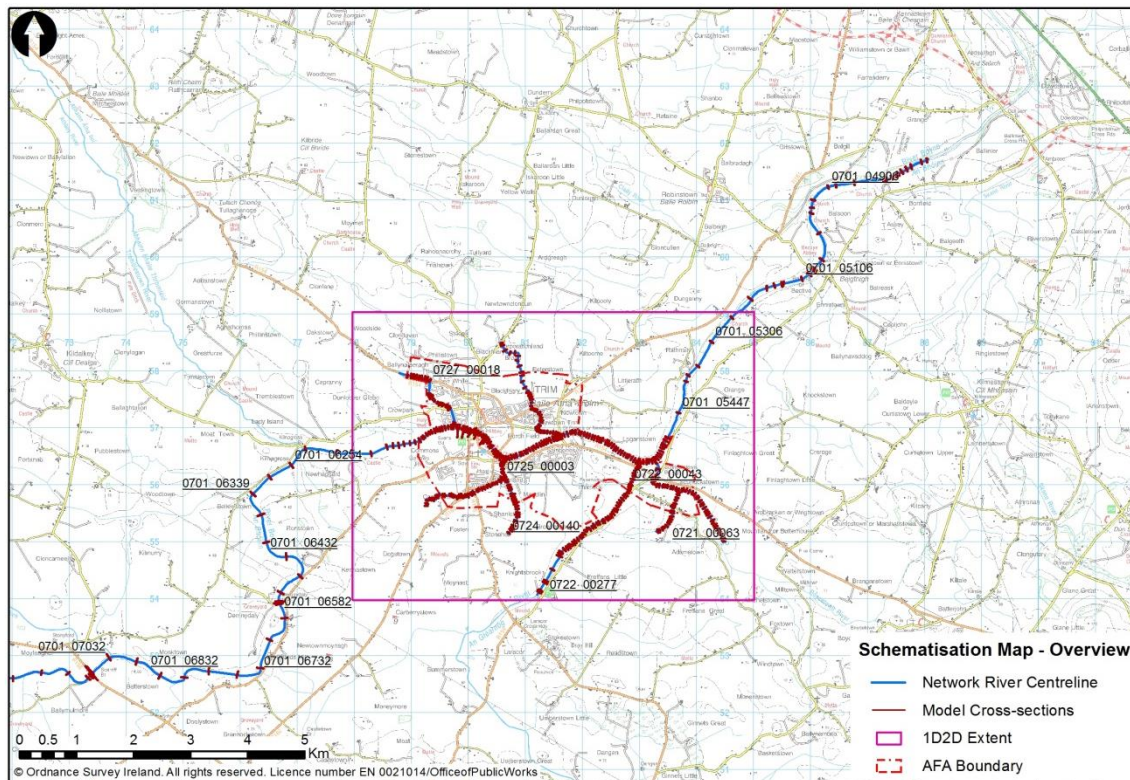


Figure 4.9.4: Overview of Model Schematisation (1 of 2)

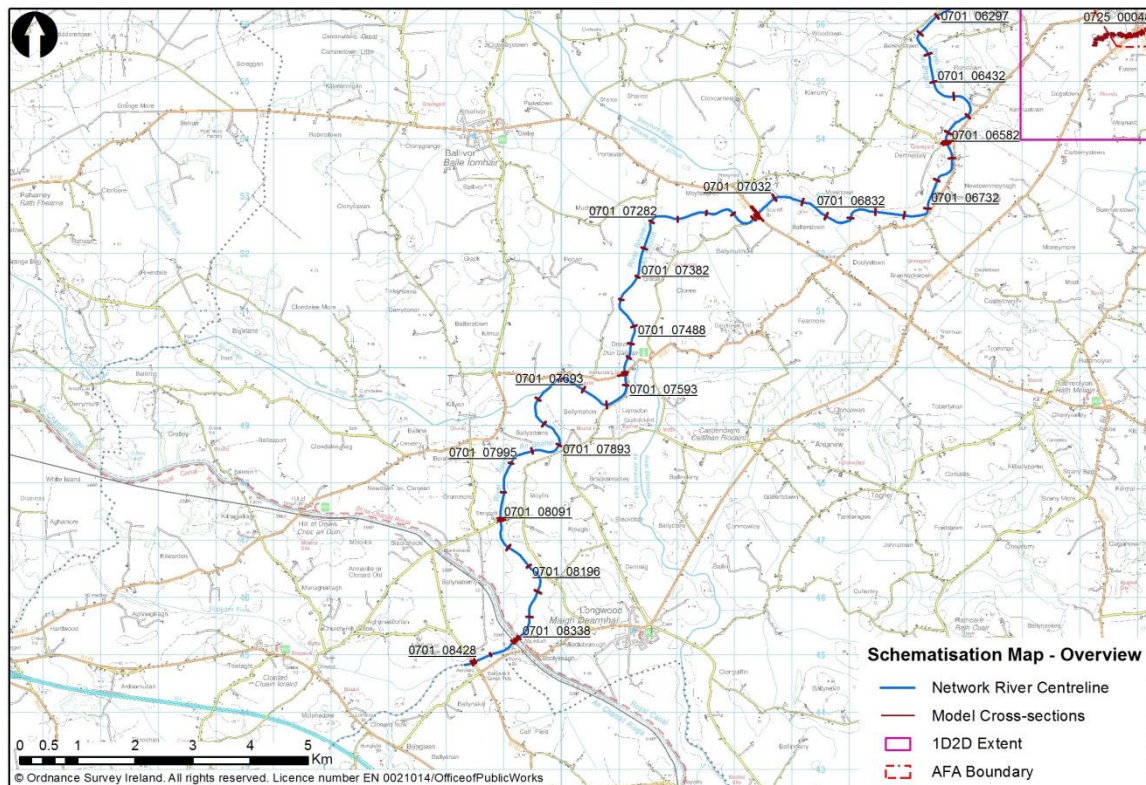


Figure 4.9.5: Overview of Model Schematisation (2 of 2)

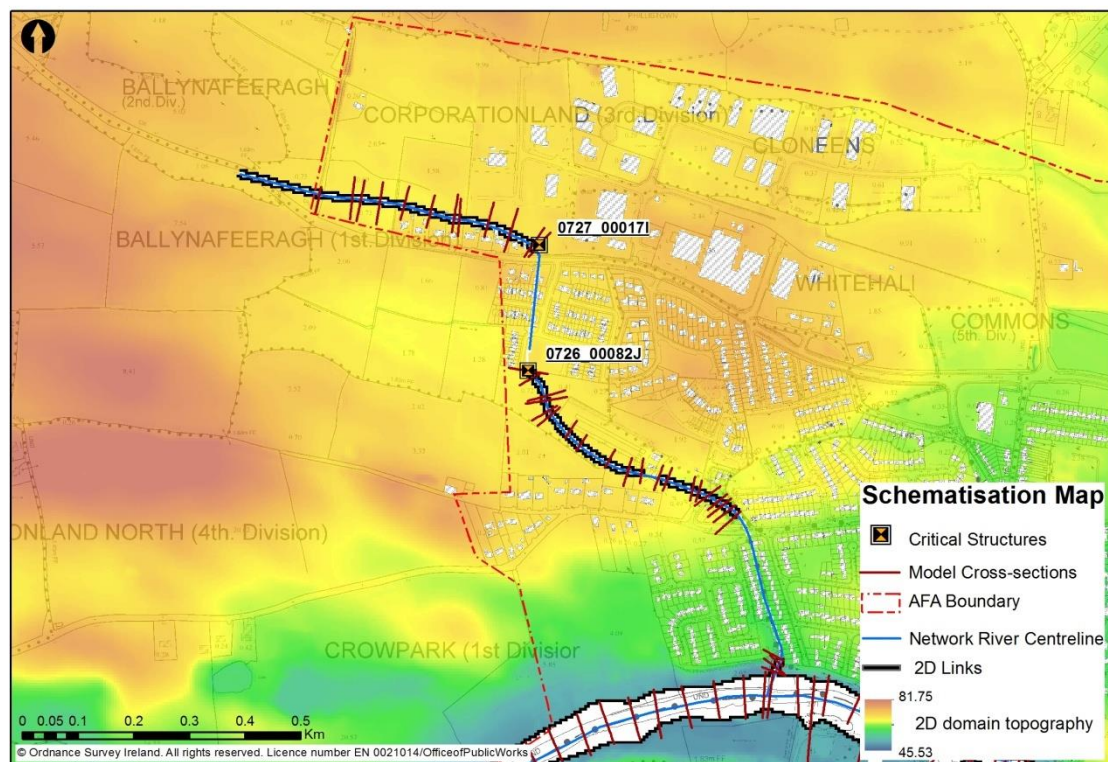


Figure 4.9.6: Model Schematisation of a reach of the Butter Stream

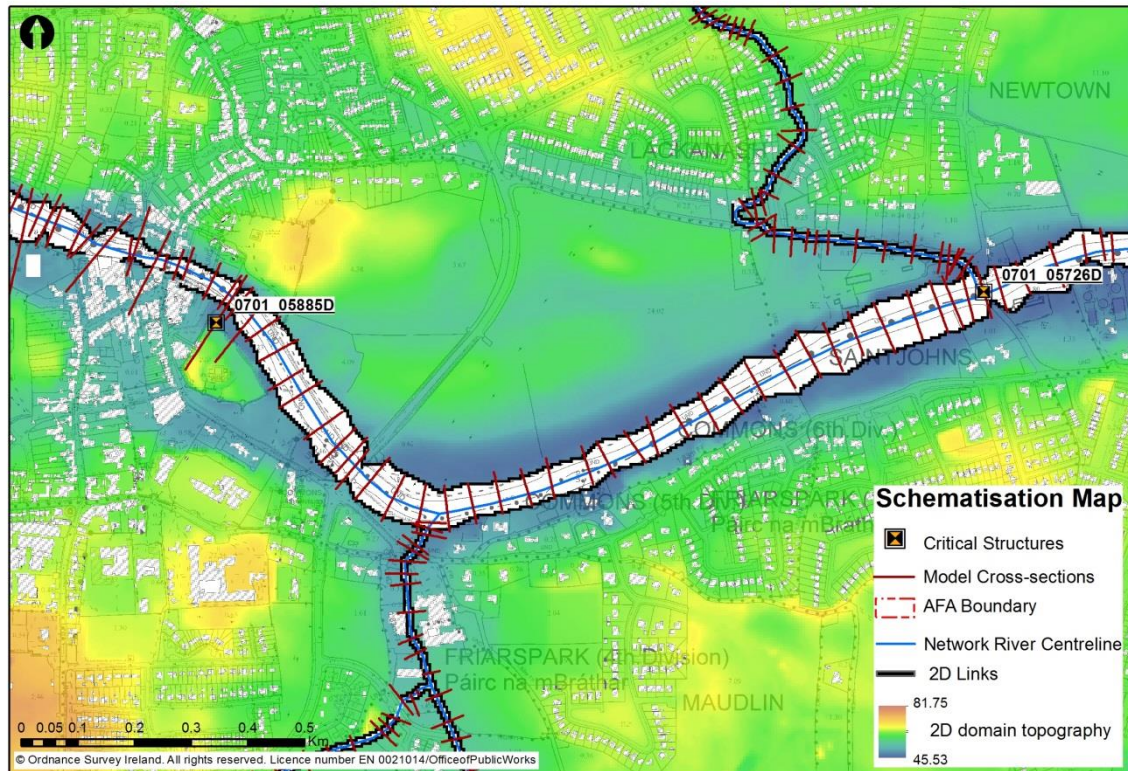


Figure 4.9.7: Model Schematisation of a reach of the River Boyne

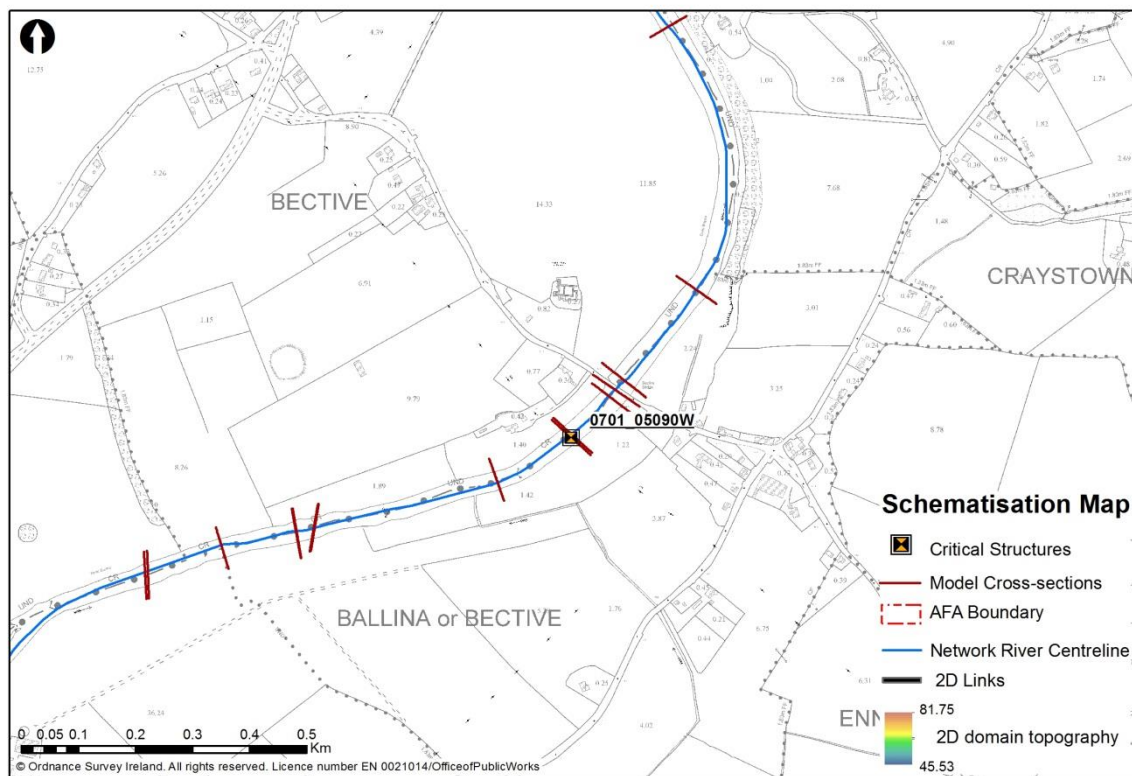


Figure 4.9.8: Model Schematisation of a reach of the River Boyne

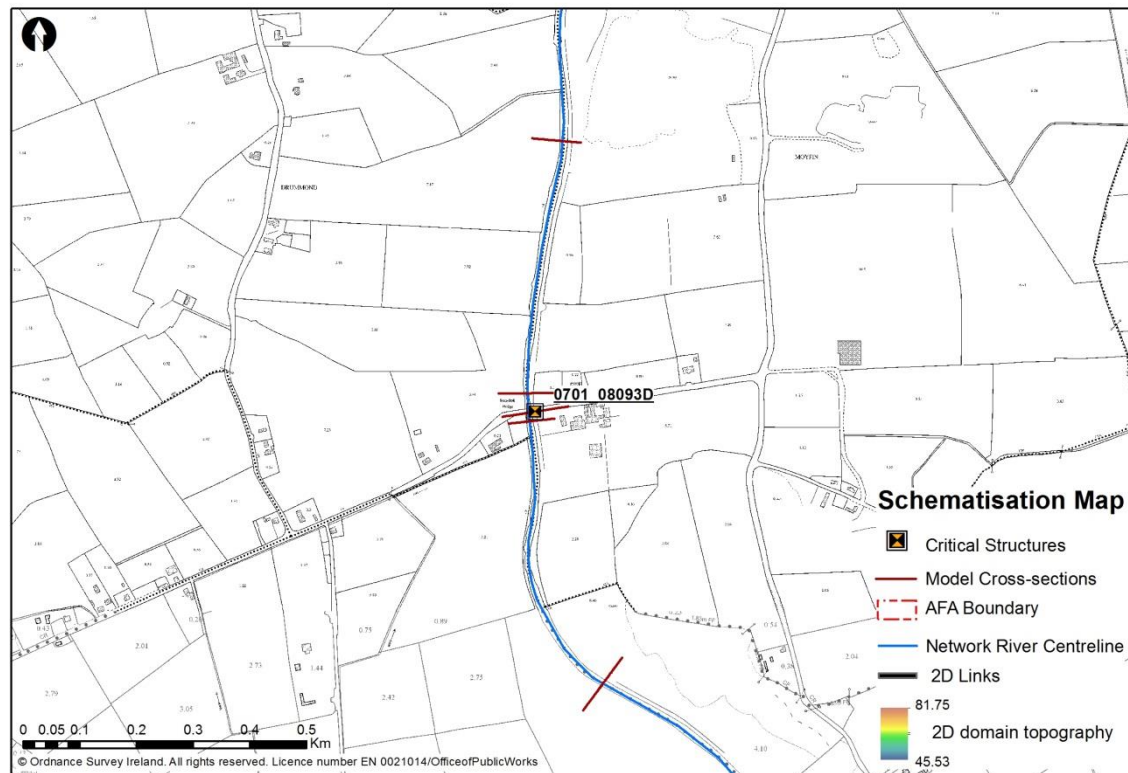


Figure 4.9.9: Model Schematisation of a reach of the River Boyne (in the Stoneyford area)

(8) Survey Information

(a) Survey Folder Structure:

First Level Folder	Second Level Folder	Third Level Folder
<i>Murphy_E07_M04_WP1_0701J_120620</i>	V0_0701_J_GIS and Floodplain Photos	Flood Plain Photos and Shapefiles
Trim		
Murphy: Surveyor Name	V0_0701_J_Videos	
E07: Eastern CFRAM Study Area, Hydrometric Area 7	V1_0701_J_XS Drawings and PDFs	4409-0701_J_V1
M04: Model Number 4		
0701J: River Reference	Photos (<i>Naming convention is in the format of Cross-Section ID and orientation - upstream, downstream, left bank or right bank</i>)	
WP1: Work Package		
Version: Most up to date		
120620: Date Issued (20 th JUN 2012)		

(b) Survey Folder References:		
Reach ID	Name	File Reference
0701	RIVER BOYNE	<i>Murphy_E07_M04_M06_WP3_0701F_120627</i>
0701	RIVER BOYNE	<i>Murphy_E07_M01_M04_WP2_0701I_120625</i>
0701	RIVER BOYNE	<i>Murphy_E07_M04_WP1_0701J_120620</i>
0701	RIVER BOYNE	<i>Murphy_E07_M04_WP1_0701K_120620</i>
0701	RIVER BOYNE	<i>Murphy_E07_M04_WP1_0701L_120620</i>
0720	BOYCE TOWN RIVER	<i>Murphy_E07_M04_WP1_0720_120702</i>
0721	BOYCE TOWN RIVER TRIB	<i>Murphy_E07_M04_WP1_0721_120620</i>
0722	KNIGHTSBROOK RIVER	<i>Murphy_E07_M04_WP1_0722_120620</i>
0723	NEWTOWN BRIDGE STREAM	<i>Murphy_E07_M04_WP1_0723_120620</i>
0724	FRIARS PARK STREAM	<i>Murphy_E07_M04_WP1_0724_120620</i>
0725	FRIARS PARK STREAM TRIB	<i>Murphy_E07_M04_WP1_0725_120620</i>
0726	BUTTER STREAM	<i>Murphy_E07_M04_WP1_0726_120620</i>
0727	BUTTER STREAM TRIB	<i>Murphy_E07_M04_WP1_0727_120620</i>
(9) Survey Issues:		
There were no survey queries issued in relation to this model.		
The cross-sections located within the 1D only model reaches (between chainage 20043.692 - 42933 and chainage 52461.542 - 59096.925) have been reviewed and extended using the NDHM (in order to prevent 'glass walls' from affecting peak water levels and flows).		

4.9.3 Hydraulic Model Construction

(1) 1D Structures (in-channel along modelled watercourses):	<p>See Appendix I</p> <p>Number of Bridges and Culverts: 56</p> <p>Number of weirs: 28 (25 weirs, 3 cross-sections entered in network model acting as a weir)</p>
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Figure 4.9.10: River Boyne – 6159W (photo 0701_06160W_RB)

Figure 4.9.10 shows a weir on the River Boyne, cross section 0701_06160W at chainage 42911. This structure was removed from the network file as a weir to improve model run stability. The inclusion of this modelled structure produced a very large build-up of water, and consequent unreal difference between the water levels upstream and downstream of the weir. Model instability was diagnosed by erratic flickering of the water level and a courant number exceeding 10 (ideally should be less than 1). To represent the weir within the model, the surveyed cross-sections were retained; this approach allows the representation of the weir and improves model stability. Considering that this modelled weir drowns out during all design runs it is considered to be representative of reality.

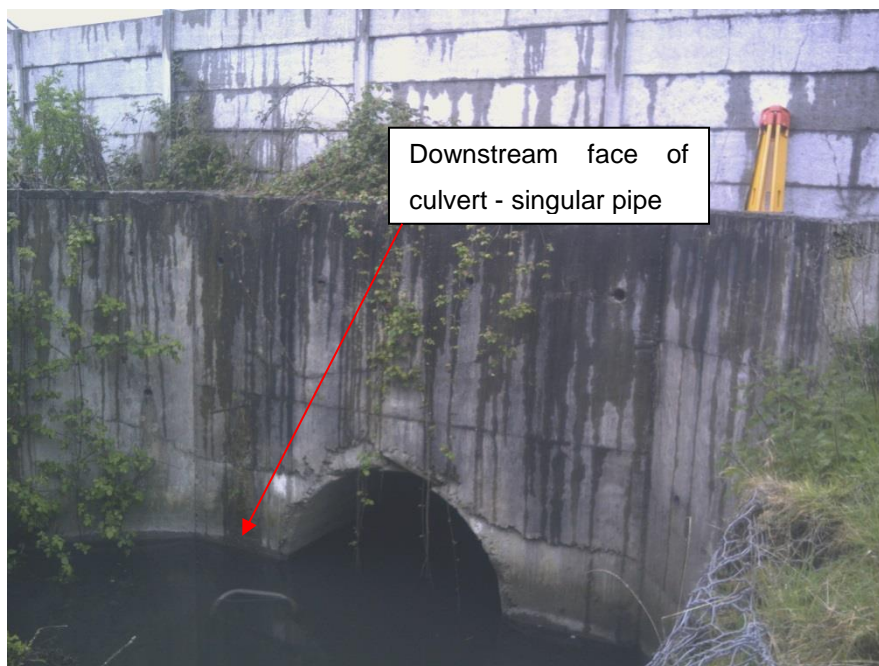


Figure 4.9.11: 0726, Butter Stream – 82J (photo 0726_00082J_UP)



Figure 4.9.12: 0727, Butter Stream Tributary – 17I (photo 0727_00017I_DN)

Culvert 17I on Butter Stream Tributary was found to flow directly into Butter Stream, with its downstream face being 82J. It can be seen on the survey photographs in Figure 4.9.11, which shows the downstream face and Figure 4.9.12, which shows the upstream face. It is noted that variance exists between the two faces of this culvert, with no detailed information relating to the nature of this change. Subsequently, assumptions have made regarding the internal transition between the upstream and downstream culvert faces. Further information can be found in Section 4.9.6 of this report. No significant afflux was found to occur due to this culvert during any design run.



Figure 4.9.13: 0701, River Boyne – 8093D (photo 0701_08093D_LB)

Figure 4.9.13 shows bridge 8093D at chainage 23577.994 on the River Boyne. The small arch on the far left hand side of the bridge was removed for model run purposes. Further information is stated in 4.9.6.1 Hydraulic Model Assumptions.



Figure 4.9.14: 0701, River Boyne – 5885D (photo 0701_05885D_DN)

Figure 4.9.14 shows a foot bridge on the River Boyne at chainage 45662.687 which was identified as a critical structure. Water builds up to a slight extent upstream of the bridge in the higher return periods due to the banks at this bridge intruding into the channel. The bridge was found to become surcharged during design runs of 0.1% AEP as the soffit level of the bridge is 53.71mOD Malin and the maximum calculated water level during this design run is 54.06mOD Malin. This results in an afflux of approximately 0.2m across the structure.



Figure 4.9.15: 0701, River Boyne – 5726D (photo 0701_05726D_DN)

Figure 4.9.15 shows a multi-arched bridge (5 arches) contained within the channel on the River Boyne at chainage 47255.772 which was identified as a critical structure, again due to the small amount of water attenuation occurring here.



Figure 4.9.16: 0701, River Boyne – 5090W (photo 0701_05090W_UP)

Figure 4.9.16 shows a critical structure identified on the River Boyne at chainage 53603.826. This weir causes an afflux of approximately 0.2m at low flow, resulting in a build-up of water upstream. This weir was found to become flooded out in extreme events.

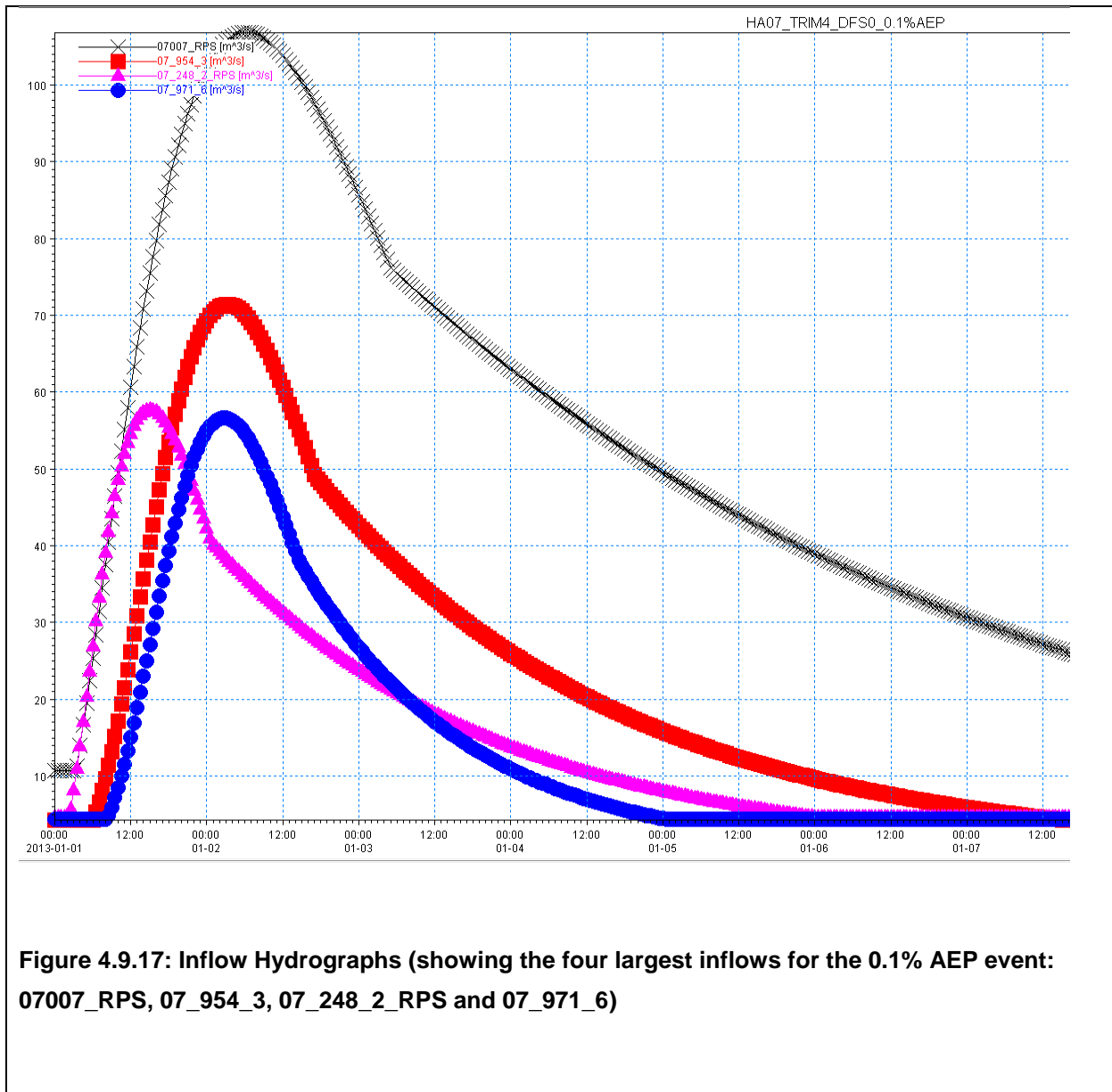
(2) 1D Structures in the 2D domain (beyond the modelled watercourses):	None
(3) 2D Model structures:	None
(4) Defences:	
No formal or informal defences present.	
(5) Model Boundaries - Inflows:	
Full details of the flow estimates are provided in the Hydrology Report (IBE0600Rp0012_HA07 Hydrology Report). The boundary conditions implemented in the model are shown in Table 4.9.2.	

Table 4.9.2: Model Boundary Conditions

	Boundary Description	Boundary Type	Branch Name	Chainage	Chainage	Gate ID	Boundary ID
1	Open	Inflow	River Boyne	20043.692	0		07007_RPS
2	Point Source	Inflow	River Boyne	26564.284	0		07_1516_10
3	Point Source	Inflow	River Boyne	29072.341	0		07_954_3
4	Point Source	Inflow	River Boyne	31688.433	0		07_340_5_RPS
5	Point Source	Inflow	River Boyne	34187.779	0		07_248_2_RPS
6	Point Source	Inflow	River Boyne	37176.594	0		07_1746_5
7	Point Source	Inflow	River Boyne	41118.918	0		07_965_2
8	Point Source	Inflow	River Boyne	42362.291	0		07_971_6
9	Open	Inflow	Athboy Road Tributary 2 - Boy	146.808	0		07_461_U
10	Distributed Source	Inflow	Athboy Road Tributary 1 - Boy	655.613	1476.4104		Top-up between 07_461_U & 07_461_3
11	Distributed Source	Inflow	River Boyne	24062.396	45466.64		Top-up between 07_1517_5_RPS & 07005_RPS
12	Open	Inflow	Stonehall Tributary 1 - Boyne	0	0		07_10000_U
13	Open	Inflow	Stonehall Tributary 2 - Boyne	0	0		07_20000_U
14	Distributed Source	Inflow	Stonehall Tributary 2 - Boyne	0	1680.415		07_20000_1
15	Distributed Source	Inflow	Stonehall Tributary 1 - Boyne	0	1430.9456		Top-up between 07_10000_U & 07_10000_1
16	Open	Inflow	Blackfriary Tributary - Boyne	389.337	0		07_54_2
17	Distributed Source	Inflow	Blackfriary Tributary - Boyne	389.337	2833.3057		Top-up between 07_54_2 & 07_601_6
18	Open	Inflow	Trim Tributary 3 - Boyne	0	0		07_1075_1
19	Distributed Source	Inflow	Trim Tributary 3 - Boyne	0	3065.3067		Top-up between 07_1075_1 & 07_908_4
20	Open	Inflow	Trim Tributary 1 - Boyne	0	0		07_181_2
21	Open	Inflow	Trim Tributary 2 - Boyne	0	0		07_1609_1
22	Distributed Source	Inflow	Trim Tributary 2 - Boyne	0	905.1724		Top-up between 07_1609_1 & 07_1609_3
23	Distributed Source	Inflow	Trim Tributary 1 - Boyne	0	2281.4743		Top-up between 07_181_2 & 07_909_3
24	Point Source	Inflow	River Boyne	53946.859	0		07_335_2
25	Point Source	Inflow	River Boyne	55090.861	0		07_312_6
26	Point Source	Inflow	River Boyne	58485.151	0		07_1245_4
27	Open	Q-h	River Boyne	59096.925	0		07041_RPS
28	Distributed Source	Inflow	River Boyne	45466.64	58902.713		Top-up between 07005_RPS & 07041_RPS

The top-up hydrograph for 07_1517_5_RPS & 07005_RPS was delayed by 3 hours to coincide with the peak of the 07007_RPS inflow in the Boyne and therefore increase the downstream flow for the 07005_RPS gauging station. This was carried out to match gauged flows recorded at 07005_RPS. No changes were made to flows to increase them to match the level required, it was merely a case of delaying the peak of 07_1517_5_RPS & 07005_RPS so it coincided with the peak of 07007_RPS. No changes were made to lateral flows in the model.

Figure 4.9.17 provides examples of the largest input hydrographs and Figure 4.9.18 provides examples of the smallest inputs.



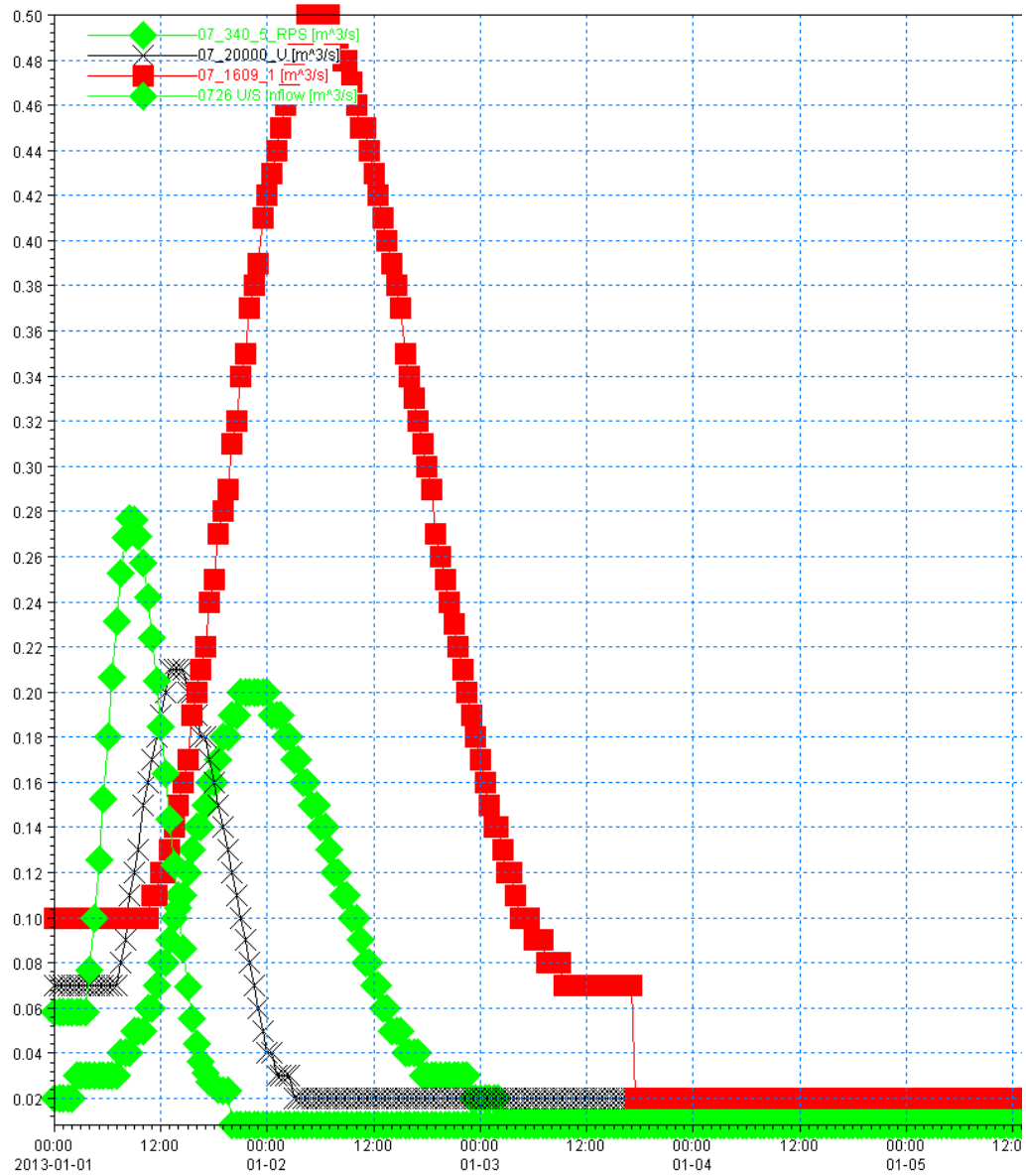


Figure 4.9.18: Inflow Hydrographs (showing the four smallest inflows for the 0.1% AEP event: 07_240_5_RPS, 07_20000_U, 0726 U/S Inflow and 07_1609_1)

(6) Model Boundaries – Downstream Conditions:

The downstream boundary condition (cross section 0701_04544 at chainage 59096.925) is an auto-calculated Q-h relationship based on Manning's formula, generated based on the cross-section at the downstream extent of the model, the slope and the Manning's value at this location. This relationship is plotted in Figure 4.9.19.

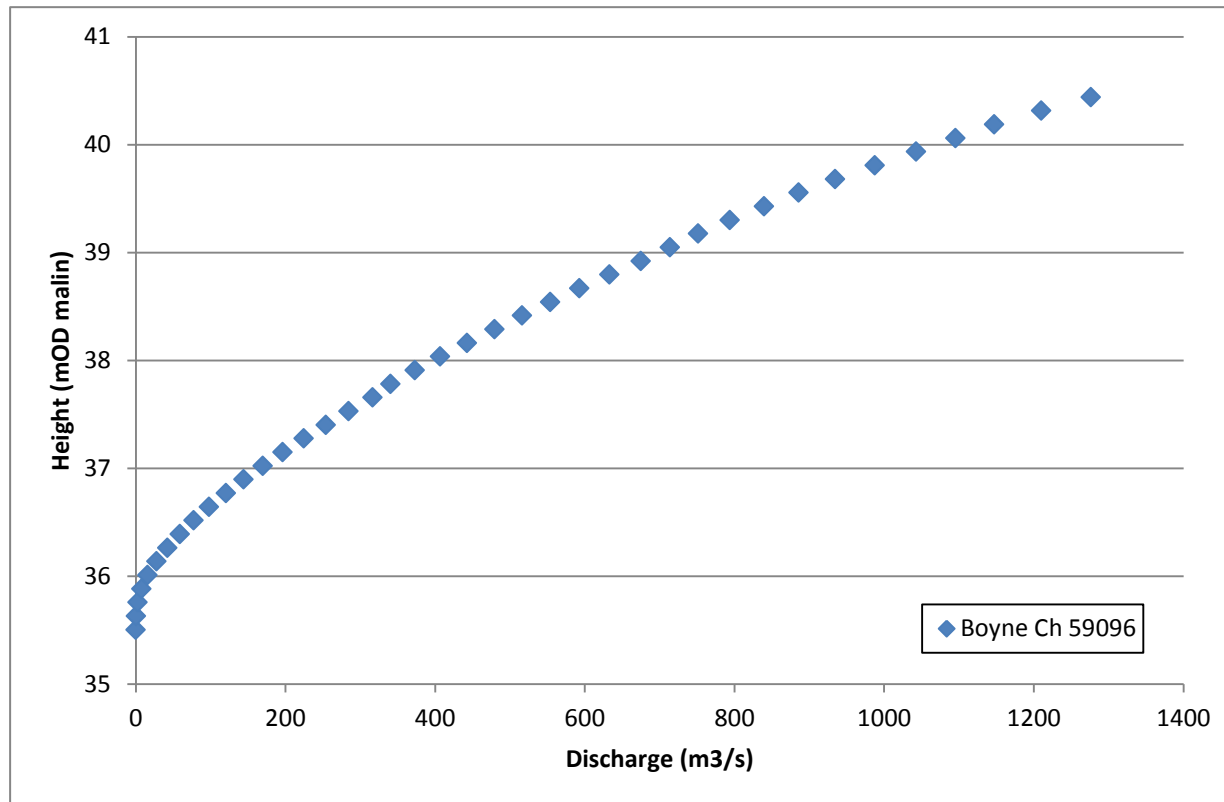


Figure 4.9.19: Downstream Boundary Condition (Q-h relationship at Cross-Section 07_04544)

(7) Model Roughness: (see Section 3.5.1 'Roughness Coefficients')

(a) In-Bank (1D Domain)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.050
(b) MPW Out-of-Bank (1D)	Minimum 'n' value: 0.035	Maximum 'n' value: 0.035
(c) MPW/HPW Out-of-Bank (2D) - see Figure 4.9.19	Minimum 'n' value: 0.03 (Inverse of Manning's 'M')	Maximum 'n' value: 0.045 (Inverse of Manning's 'M')

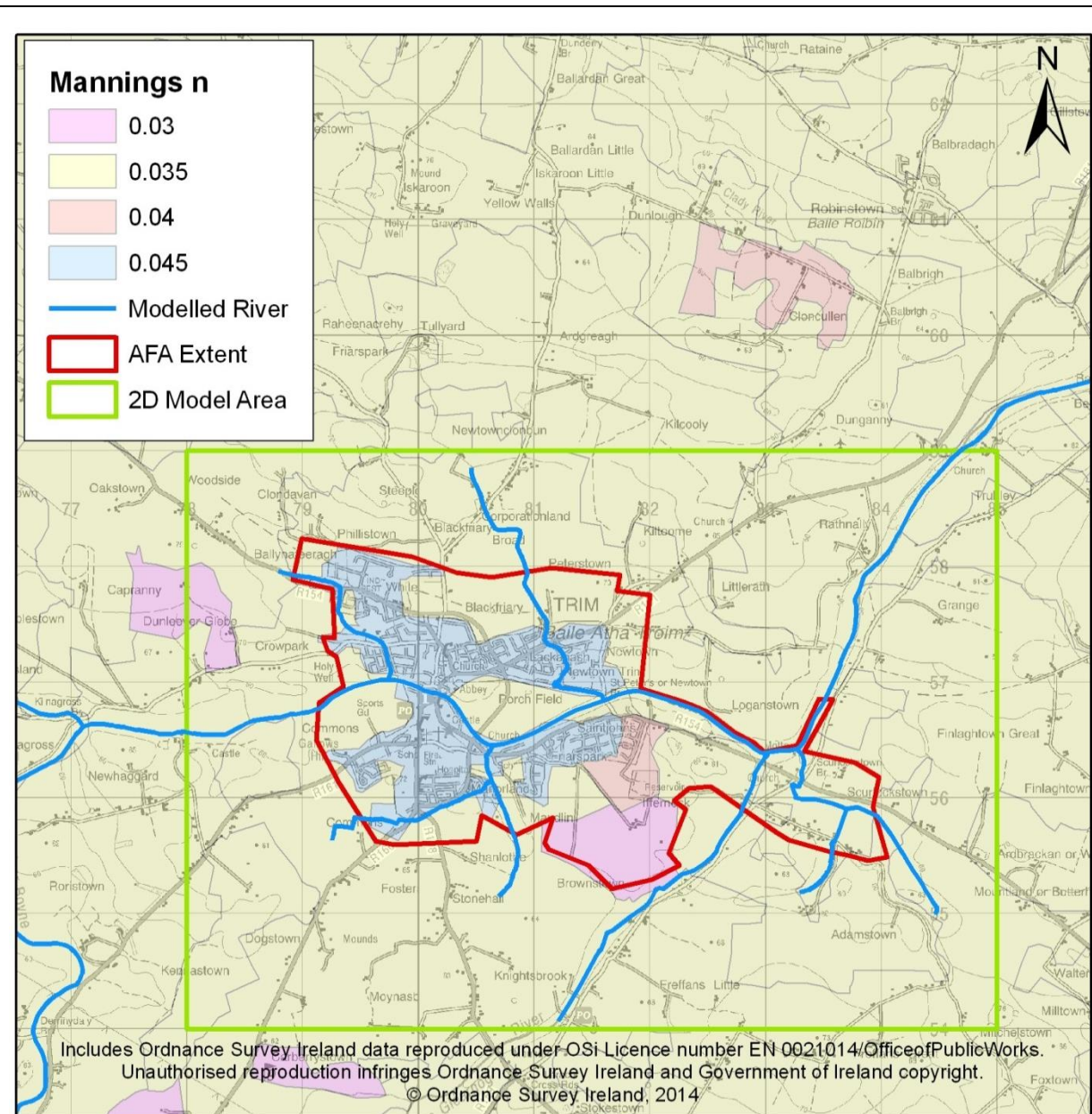


Figure 4.9.20: Model Roughness Values

The map in Figure 4.9.20 above illustrates the roughness values applied within the 2D domain of the model. Roughness in the 2D domain was applied based on land type areas defined in the CORINE Land Cover Map with representative roughness values associated with each of the land cover classes in the dataset.

(d) Examples of In-Bank Roughness Coefficients**Figure 4.9.21: Manning's $n = 0.035$ (0722)**

Knightsbrook River– 0722_00183_UP (Trim Tributary 3 - Boyne in the model) at chainage 1233.258.

Standard natural stream or river in stable condition

**Figure 4.9.22: Manning's $n = 0.040$ (0725)**

Friars Park Stream Tributary 0725_00090I_UP (Stonehall Tributary 2 - Boyne in the model) at chainage 782.414.

Standard natural stream or river in stable condition, with more stones and weeds.

**Figure 4.9.23: Manning's $n = 0.050$ (0723)**

0723_00186_UP Cross section 0723_00186 located on Newtown Bridge Stream (0723) (Blackfriary Tributary - Boyne in model) at chainage 923.795.

Clean, winding, some pools and shoals but more weeds and stones.

**Figure 4.9.24: Manning's $n = 0.050$ (0720)**

0720_00158_DN located at cross section 0720_00158 located on the Boyce Town River (0720) (Trim Tributary 1 - Boyne in the model) at chainage 706.093.

Clean, winding, some pools and shoals with more weeds and stones.



Figure 4.9.25: Manning's $n = 0.035$ (0701 chainage 38538.64)

0701_06597_UP located at cross section 0701_06597 located on the River Boyne (0701) at chainage 38538.64.

Standard natural stream or river in stable condition



Figure 4.9.26: Manning's $n = 0.035$ (0701 chainage 23568)

0701_08093D_UP located at cross section 0701_08093D located on the River Boyne (0701) at chainage 23568.

Standard natural stream or river in stable condition

4.9.4 Sensitivity Analysis

In accordance with OPW guidance sensitivity simulations have been completed. The purpose is to assess the sensitivity and impact 1% AEP hydraulic model within the AFA boundary by adjusting various parameters. The following sensitivity analysis model simulations have been carried out:

- a) Channel roughness and flood plain roughness increased to upper bound values and flood plain roughness decreased to lower bound values – the change in channel and floodplain roughness values resulted in a moderate increase in flood extents, compared to the original extent of flooding within the AFA as shown in Figure 4.9.27. This outcome indicates that the Trim model demonstrates a moderate sensitivity to changes in roughness parameters. This increase in flood extents now affects 18 receptors, which is a 50% increase compared to the 1% AEP design event. The main area of impact is located around the Market Street area of Trim.

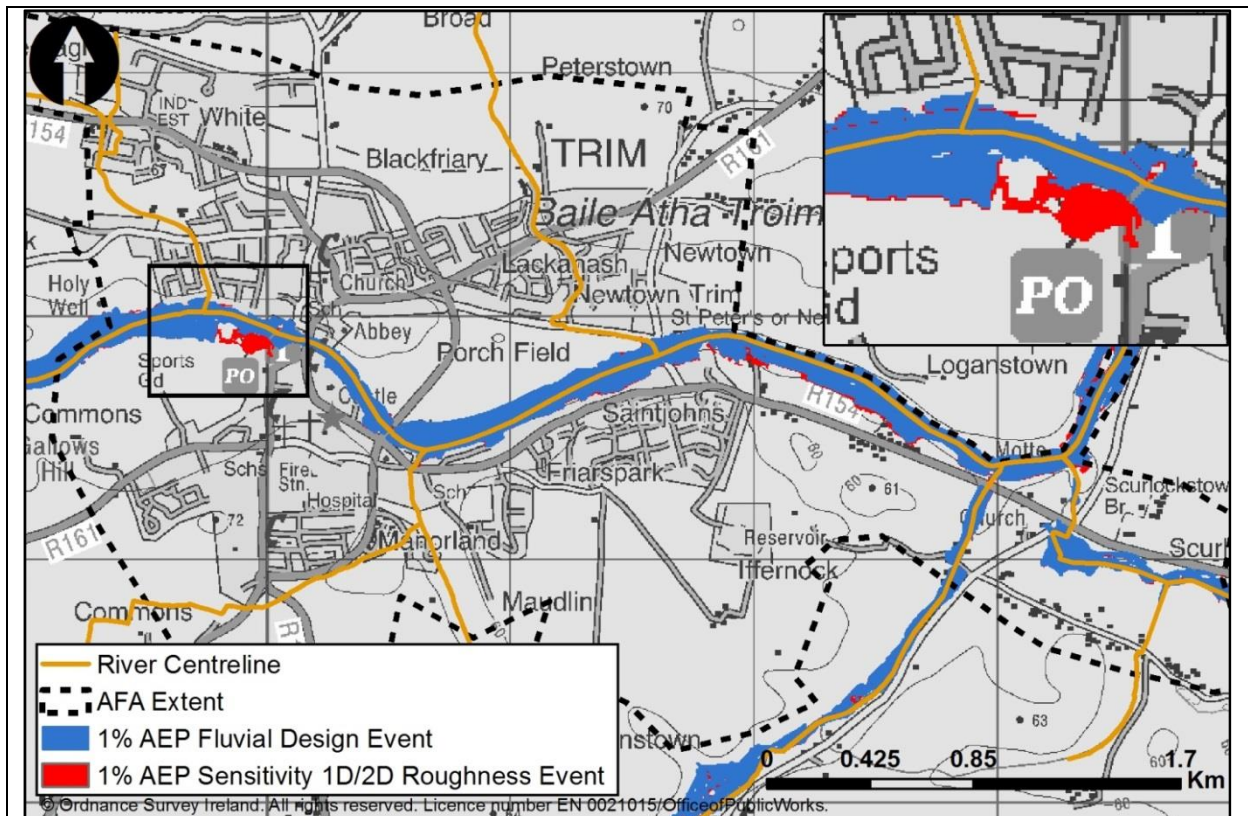


Figure 4.9.27: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Event – Change in 1D/2D Roughness

- b) Downstream boundary increase – The downstream boundary is located approximately 9.6km downstream of the AFA boundary and is approximately 8m lower than bed levels within the AFA. The downstream boundary parameter was evaluated as having no potential impact within the AFA and therefore a sensitivity simulation was not required. The Trim model has no sensitivity to changes in downstream water level boundary within the AFA.
- c) Increase in flow – The 1% AEP sensitivity flows were used to assess the sensitivity of the model to inflows. The Trim model was assessed as having Medium/Low uncertainty/sensitivity (refer to the Hydrology Report IBE0600Rp0012 for further detail); factors of 1.37, 1.57, 1.68 and 2.06 are applied to design flows for the sensitivity simulation. Figure 4.9.28 shows that the Trim model has a high sensitivity to increased inflow parameters; this is reflected by the increase of flood extent. This increase in flood extents now affects 41 properties, a 242% increase when compared to the 1% AEP design event. The main area of impact is located around the Market Street area of Trim and properties located close to the River Boyne.

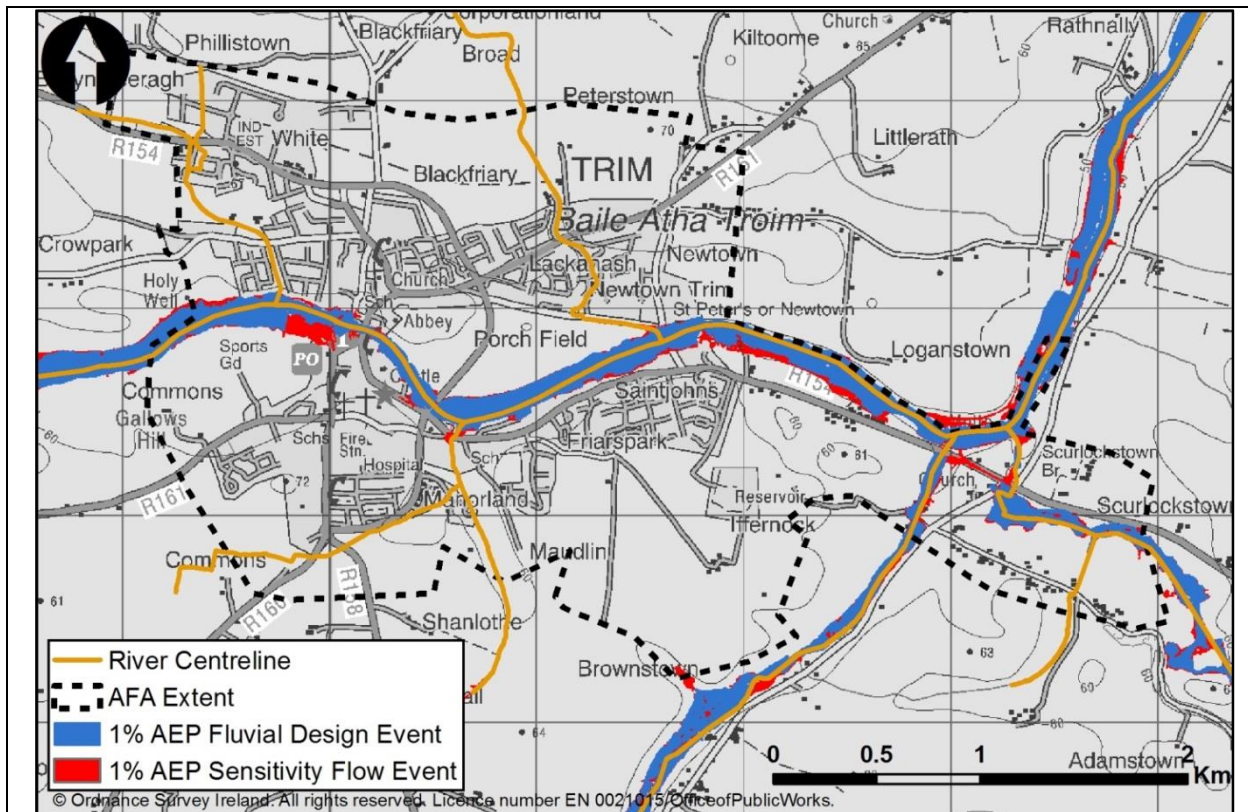


Figure 4.9.28: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Flow Event

- d) Afflux/Head loss at key structures – A number of simulations were carried out to assess the sensitivity of flood extents to changing the head loss coefficients of key structures. Three structures were assessed including 0701_05898D, 0701_05726D and 0720_00043W on the River Boyne and Boyce Town River (Boyne Tributary 1). The Trim model has shown a moderate sensitivity to head loss parameter changes when compared to the design event. Increasing the head loss parameters of 0701_05898D had the most impact as 15 properties located downstream of this structure are affected see Figure 4.9.29. Increasing the head loss parameters of 0720_00043W impacted 13 properties see Figure 4.9.30. Altering the head loss parameters of these structures impacted properties located mainly around the Market Street area of the town. Structure 0701_05726D revealed no sensitivity or impact.

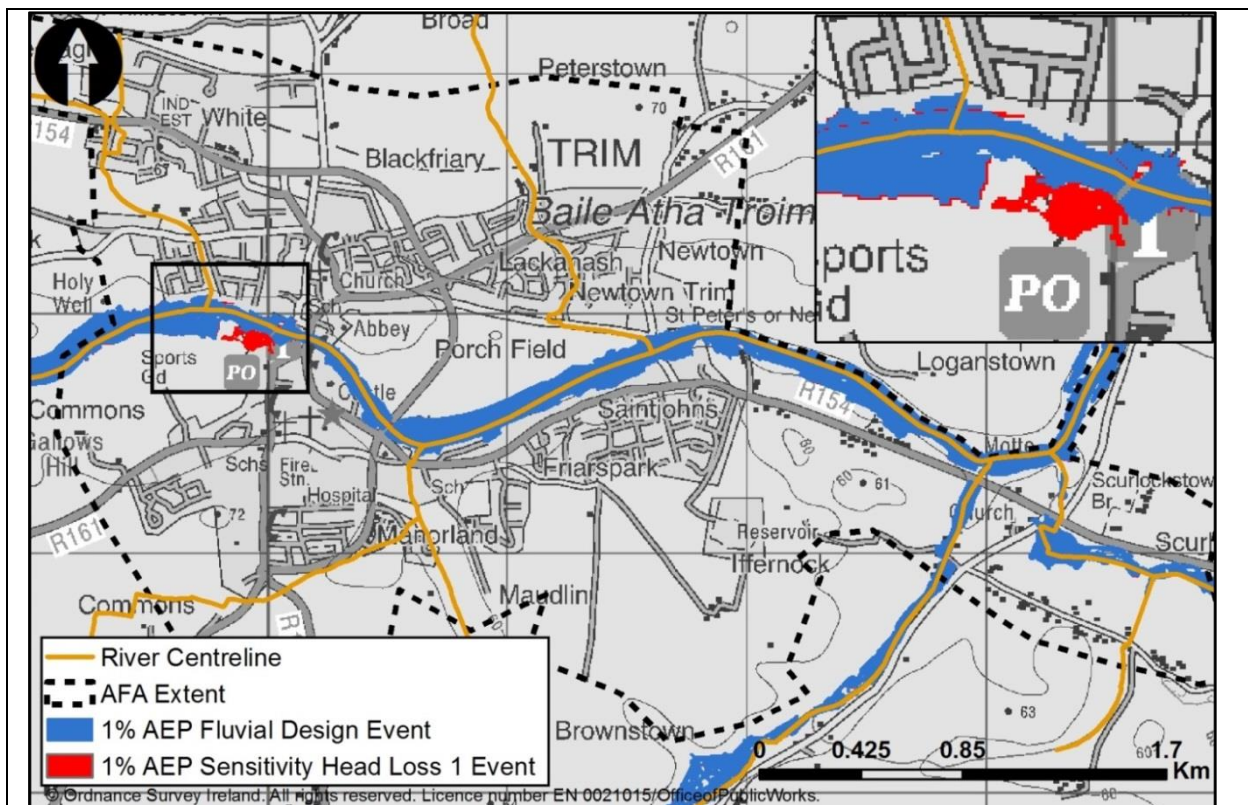


Figure 4.9.29: Comparison between 1% AEP Design Event and 1% AEP Sensitivity
(0701_05898D)

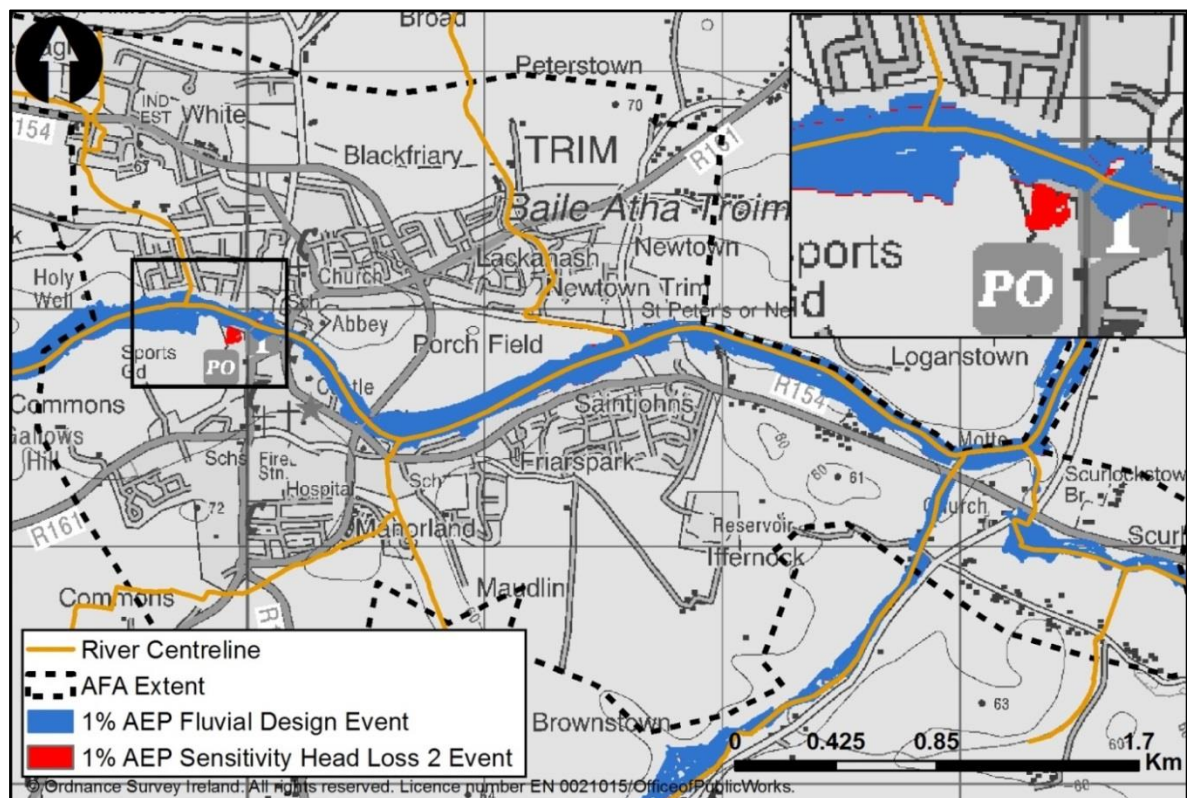


Figure 4.9.30: Comparison between 1% AEP Design Event and 1% AEP Sensitivity
(0720_00043W)

- e) Building representation – Building representation was modelled by adjusting the roughness of cells within the building footprint to a Manning's n of 0.3. The topography within the 2D model domain was based on LIDAR - the cells within building footprints remained 'unblocked'. Figure 4.9.31 shows that the Trim model has low sensitivity to building representation, as revealed by the overall low increase in the 1% AEP flood extent. This negligible change results in no further impact on to receptors located within the AFA.

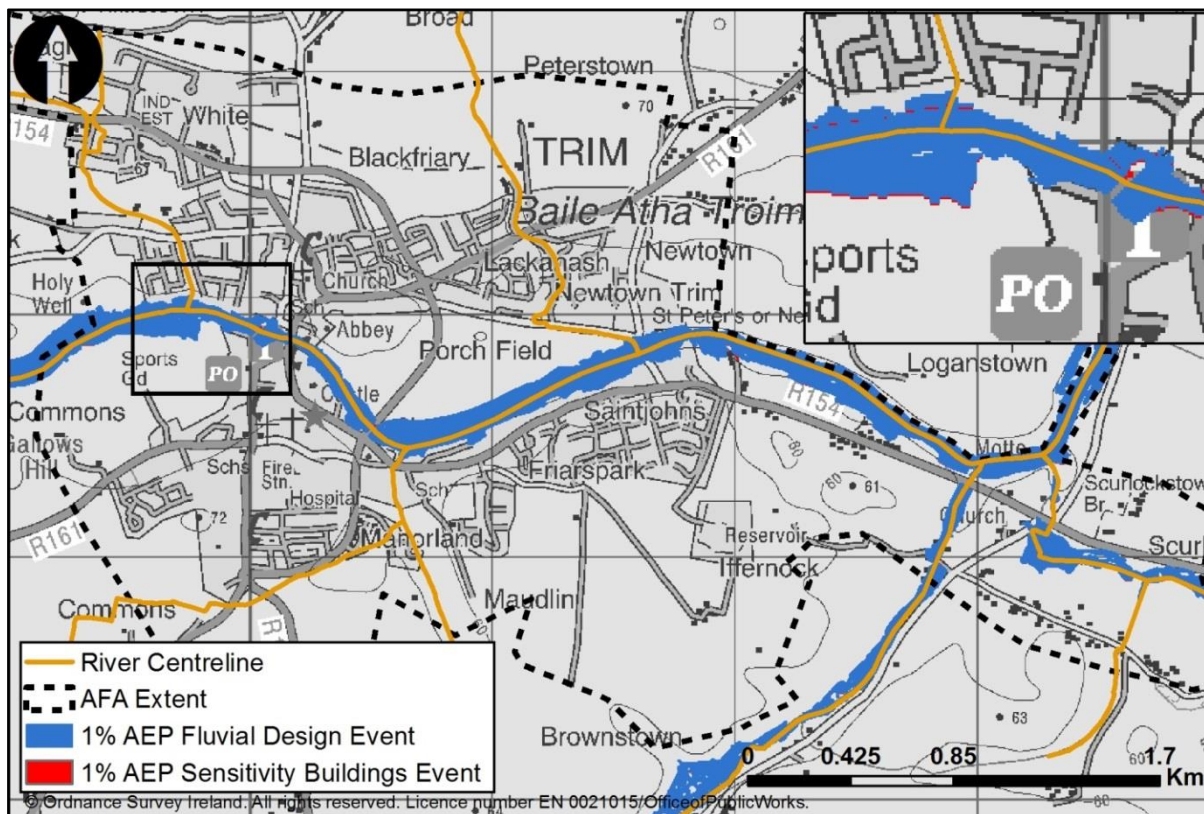


Figure 4.9.31: Comparison between 1% AEP Design Event and 1% AEP Sensitivity Buildings Event

Table 4.9.3 summarises the outcomes of the above sensitivity simulations or evaluations considered for the Trim model. Of these parameters, the model is demonstrated a high sensitivity to an increase in model inflows. Table 8.1 in the Hydrology Report states that there is medium/low uncertainty/sensitivity associated with the hydrological inputs for this model. Head Loss Event 1 (assessing 0701_05898D) also indicates high model sensitivity. The model demonstrates a moderate sensitivity to changing roughness, with the resulting analysis identifying moderate increase to flood extents. This moderate change to the flood extents has resulted in a high impact to properties located within the AFA. In comparison no properties are impacted during the design event. The model illustrates a low sensitivity to building representation and no further impact to properties located within the AFA.

Table 4.9.3: Sensitivity Summary

Sensitivity Simulation	Sensitivity	Impact
1% AEP Sensitivity 1D/2D Roughness Event	Moderate	High
1% AEP Sensitivity Water Level Boundary Event	Low	-
1% AEP Sensitivity Flow Event	High	High
1% AEP Sensitivity Head Loss 1 Event	High	High
1% AEP Sensitivity Head Loss 2 Event	Low	Low
1% AEP Sensitivity Building Representation	Low	Low

4.9.5 Hydraulic Model Calibration and Verification

(1) Key Historical Floods (from IBE0600Rp0004_HA07 Inception Report_F02 unless otherwise specified):

AUG 2008.	<p>Review of the historical data indicated that flooding occurred in Navan, Trim, Ballivor, Athboy and Edenderry in August 2008.</p> <p>At Trim, the heavy rainfall on the Boyne catchment resulted in the River Boyne overtopping its banks. The level reading at Trim Hydrometric Station was the 7th highest on record. Trim Pitch and Putt course flooded; however no further information on properties flooded or resultant damage was available. A peak flow of 129.6m³/s was recorded at Trim Hydrometric Station (07005) during this flood event.</p> <p>This was estimated to be between a 50% and 10% AEP flood event and the model simulations, shown below in Figure 4.9.32, demonstrate flooding of this vicinity during the 10% AEP event.</p> <p>Generally, the model supports the estimated return period. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.</p>
JAN 2005.	<p>Review of the historical data indicated that on 7th & 8th January 2005, the River Boyne overflowed into low lying ground and subsequently flooded the pitch and putt, the children's play area and the surrounding land in Trim. A peak flow of 125.9 m³/s occurred at Trim Hydrometric Station during this flood event.</p> <p>The peak flow recorded is similar to the peak flow recorded during the August 2008 event, with an estimated flood frequency between 50% and 10% AEP. The children's play area is within the 10% AEP flood extent as shown in Figure 4.9.32. Generally, the model supports the estimated return period. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for</p>

model calibration.

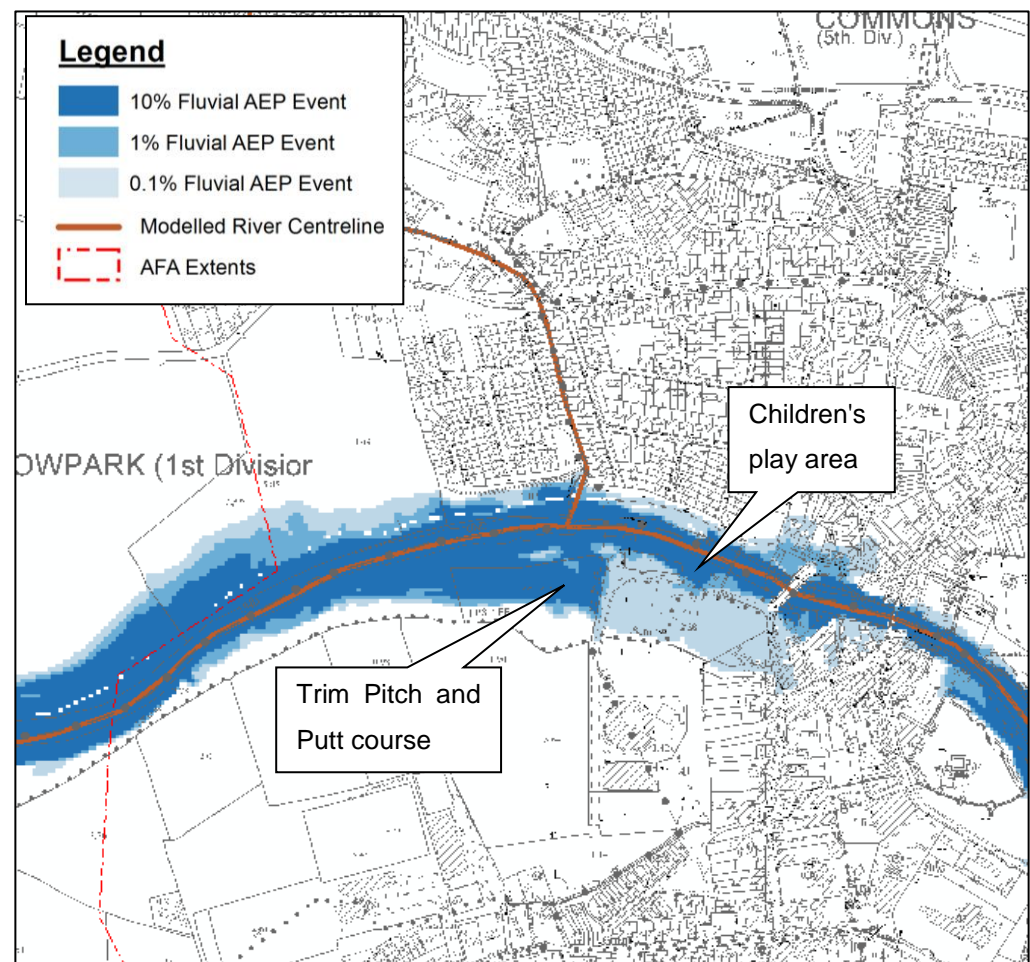


Figure 4.9.32: Simulated Flood Extents at Trim Pitch and Putt for the 1%, 10% and 0.1% AEP Events

NOV 2002.

Information was found for a flood event which occurred in Drogheda, Navan, Trim and Edenderry in November 2002. In Navan and Trim, more extensive fluvial flooding occurred as a result of the River Boyne overflowing in both towns.

There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.

NOV 2000.

Information was found for a flood event which occurred in Drogheda, Navan, Trim, Baltray, Edenderry and Mornington in November 2000. Floods were caused by heavy rain and storm force winds. In Navan and Trim it was reported more extensive flooding occurred as a result of the River Boyne overflowing in both towns.

In Trim, Patrick Street, Loman Street and Watergate Street Bridge were flooded. The bridge remained closed for a period after the flood had passed due to fears regarding its integrity.

The peak flow recorded at station 07041 on 7th November 2000 was approximately 237m³/s, which is beyond the reliable limit for this station.

Watergate Street Bridge was not found to flood during any model design run, as shown in Figure 4.9.33. It should be noted however that this bridge was replaced in 2005, so information regarding the bridge flooding cannot be used for model calibration or verification.

Flooding was only found to occur on Loman Street during design runs of 0.1% AEP, as shown in Figure 4.9.33. It was not possible to achieve model flooding in this area during less severe design runs, so it is possible that flooding at this location during November 2000 was due to an alternative source.

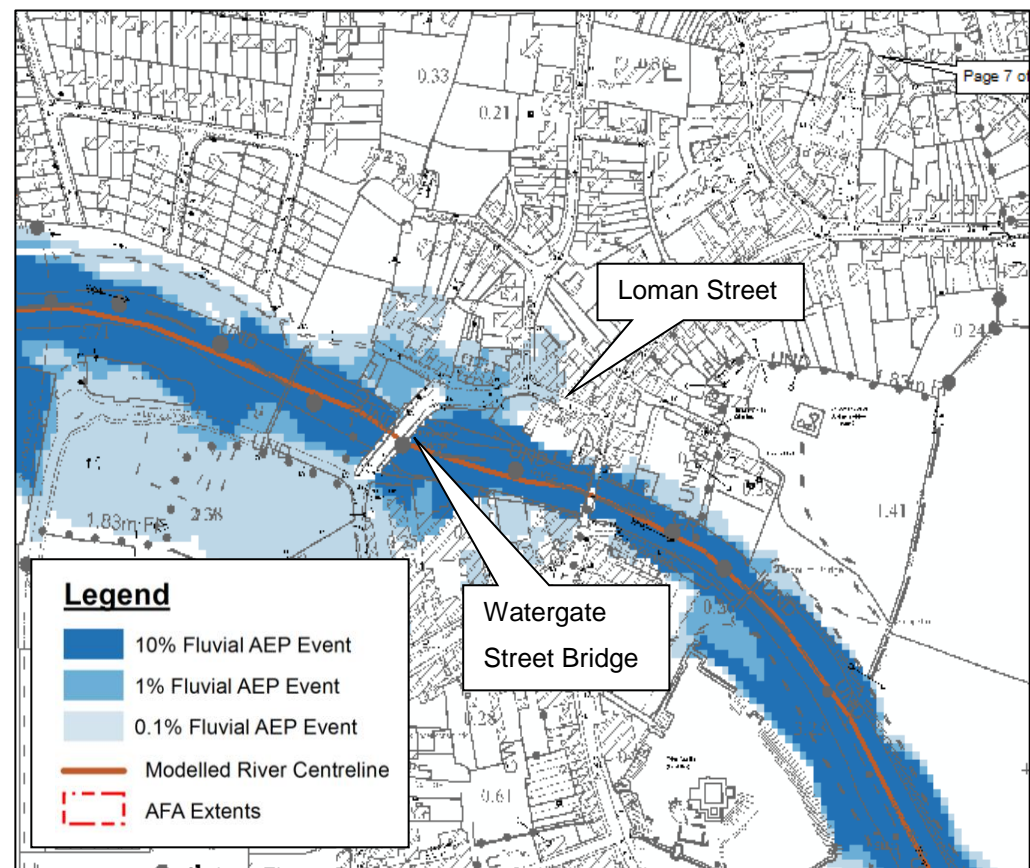


Figure 4.9.33: Simulated Flood Extents at Loman St and Watergate St Bridge, for the 1%, 10% and 0.1% AEP events

Model simulations (see Figure 4.9.34) indicate that Patrick Street is not subject to fluvial flooding from the River Boyne itself. Patrick Street is located a significant distance from all modelled flood extents. Consequently, as it is not reasonably practicable to achieve modelled fluvial flood extents which encompass the Patrick Street, and due to the lack of information on the source of flooding, it is assumed that this flooding is due to a source other than fluvial flooding.

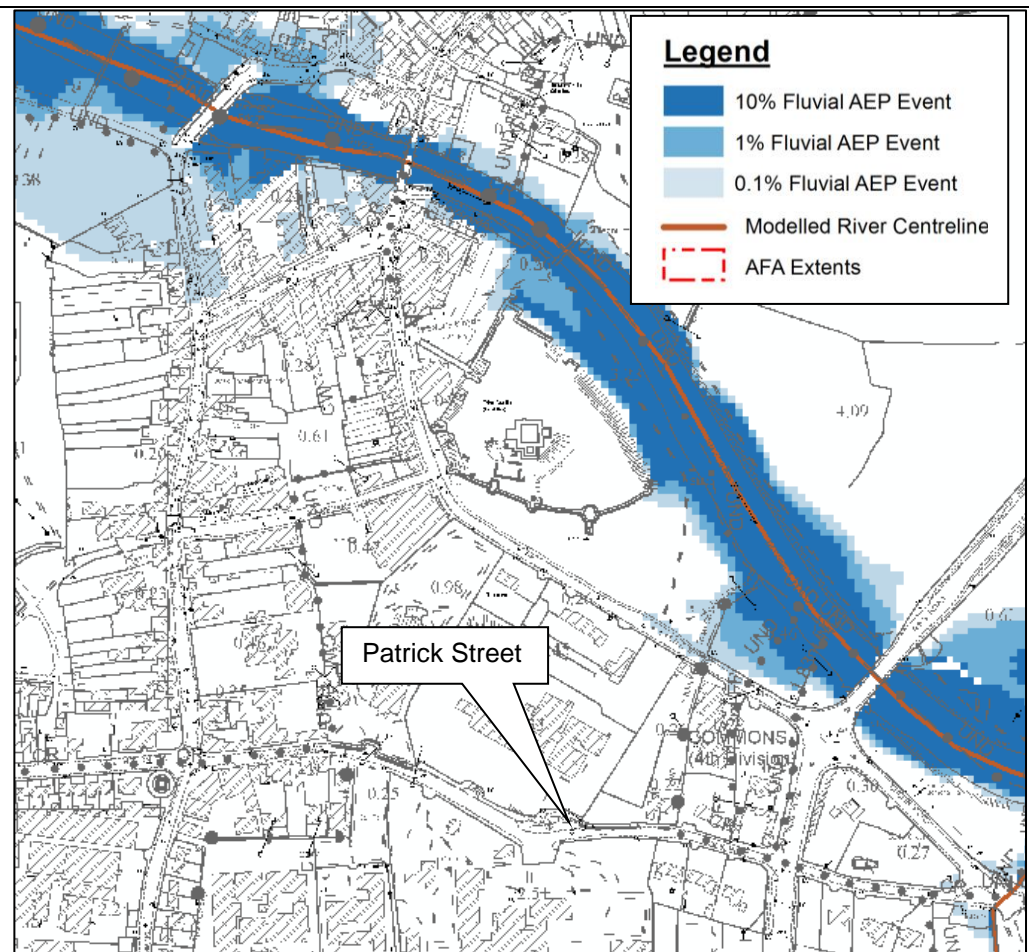


Figure 4.9.34: Simulated Flood Extents Patrick Street, for the 1%, 10% and 0.1% AEP events

There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.

FEB 1995.

Heavy rainfall in February of 1995 led to flooding in Trim and Ballivor. In Trim, the peak flow recorded at Trim Hydrometric Station (07005) for this event was $130.4\text{m}^3/\text{s}$. This equates to a flood frequency of approximately 50% AEP. The River Boyne overflowed its banks and Watergate Street Bridge was closed as a precaution. No further information is available. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.

JUN 1993.

Navan, Trim and Ballivor endured floods in June 1993 following heavy rainfall and fluvial flooding. In Trim, the peak flow for this event was measured at $138\text{m}^3/\text{s}$. The River Boyne overflowed, and Watergate Street Bridge was closed as a precaution as the water level was 2-3 inches below the deck of the bridge. The daily mean water level at Trim Hydrometric Station (as per <http://www.opw.ie/hydro>), which is located just upstream of Watergate Street Bridge, was measured as 56.0mOD Poolbeg, or approximately 53.3mOD Malin.

The peak flow recorded is similar to the peak flow recorded during the August 2008

	<p>event, with an estimated flood frequency between 50% and 10% AEP. During the 10% AEP model simulation, the peak water level at 07005 reaches 53.59 mOD which is 0.167m higher than the measured peak water level during the June 1993 flood event (53.423mOD on 12/06/93). Generally, the model supports the estimated return period. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.</p>
<p>DEC 1978.</p>	<p>Information was found for a flood event which occurred in Drogheda, Navan, Trim, Mornington and Baltray in December 1978. A maximum flow of 130m³/s was recorded at Trim hydrometric station (07005) where the River Boyne overflowed. Griffin Park, Athboy Road, Market Street, Haggard Street, High Street, St. Joseph's home and St. Mary's secondary school were all mentioned as having flooded.</p> <p>The peak flow recorded is similar to the peak flow recorded during the August 2008 event, with an estimated return period of between 50% and 10% AEP. None of the areas described were found to flood during any model design run, as shown in Figure 4.9.35 and Figure 4.9.36. These locations are likely to have been subjected to pluvial flooding rather than fluvial as they are located at a significant distance from any modelled watercourses. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.</p> <div data-bbox="448 1077 1422 1816"> </div> <p>Figure 4.9.35: Simulated Flood Extents at the Athboy Road for the 1%, 10% and 0.1% AEP events</p>

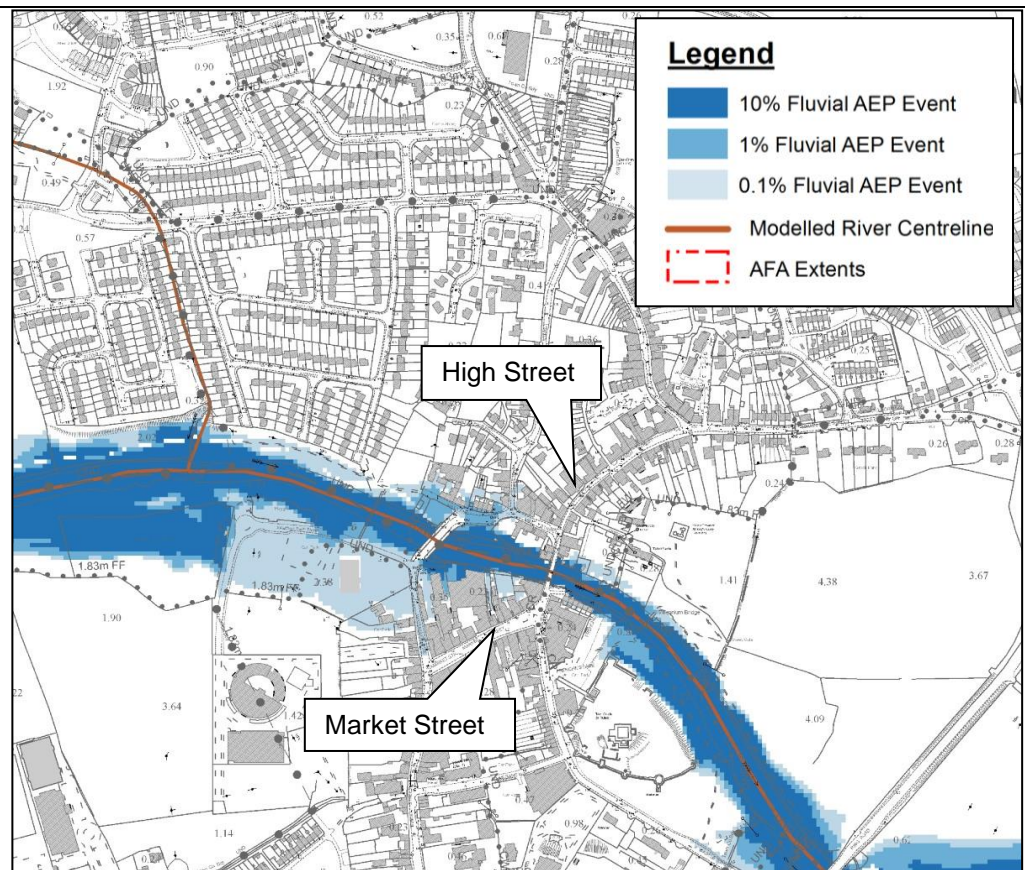
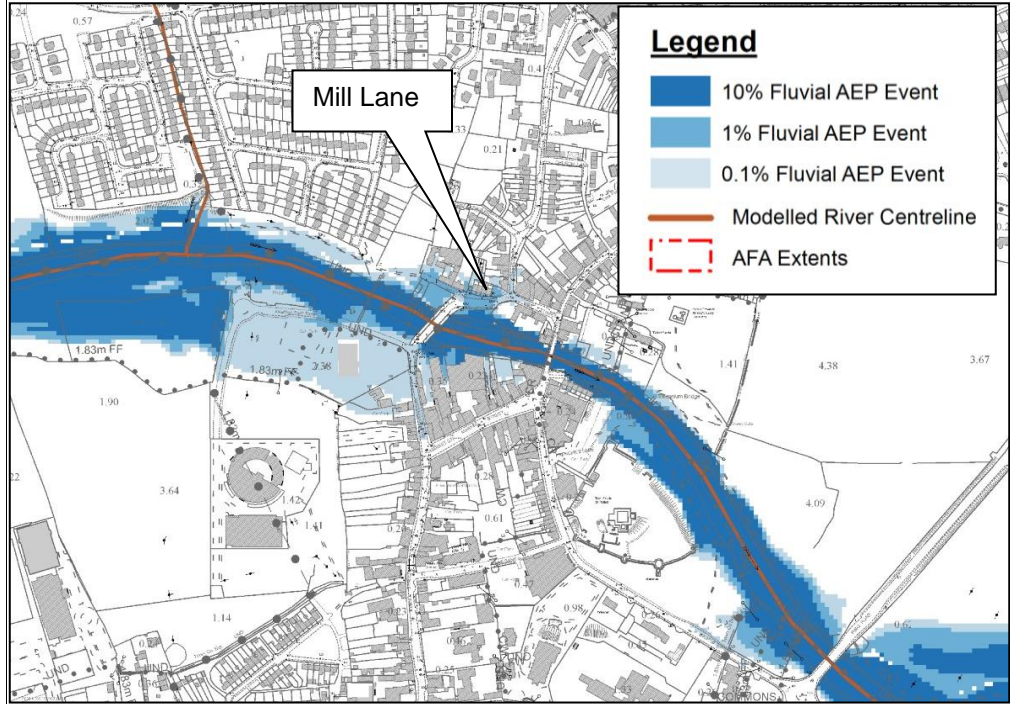


Figure 4.9.36: Simulated Flood Extents at High Street and Market Street for the 1%, 10% and 0.1% AEP Events

DEC 1968.	An OPW report indicates that flooding occurred during December 1968/January 1969 in Navan and Trim when the River Boyne overflowed. The affected areas were at Derrindaly, near Trim, and at Kilcarn, near Navan. However, no exact date or specific details are available for this flood event. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.
NOV 1965.	<p>A flood event was found to have occurred in Drogheda, Navan and Trim in November of 1965. The flooding was caused by heavy rainfall, with the situation in Drogheda compounded due to high tides. In Trim, the River Boyne overtopped its banks. The peak flow recorded at Trim hydrometric station was $186.52\text{m}^3/\text{s}$. Some press articles reported that the level of the River Boyne rose to 6 feet above its normal level (from http://www.opw.ie/hydro, the 95 percentile level at Trim hydrometric station, upstream of Watergate Bridge, is 53.36mOD Poolbeg derived for the period 1975 to 2008), while others reported that the level was an inch above the 1954 flood level (which itself was reported to have risen above the parapets of the New Bridge). The New Bridge was impassable and 3 houses flooded with reported depths of over 3 feet of water.</p> <p>The peak water level reached at the Trim Hydrometric Station is not available. The</p>

	<p>reported peak flow is significantly higher than the reliable limit of the gauge. The reported comments are insufficient in detail in order to be used to calibrate or validate the model - the location of the flooded houses is unknown. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.</p>
JAN 1965.	<p>Flooding occurred in Navan and Trim in January 1965. Flooding was reported in the Moymet area of Trim. This area is not located in the vicinity of any modelled watercourses. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.</p>
DEC 1954.	<p>A flood event was found to have occurred in Drogheda, Mornington, Navan and Trim in December of 1954. In Trim, the River Boyne overflowed its banks and the water level rose above the parapets of the "new bridge". Press article reported flooding of houses on Mill Lane and Athboy Road. During this flood event, the automatic flood gauge in the river was swept away; hence hydrometric data is not available.</p> <p>It is not possible to estimate the flood frequency of this event as gauge data is missing. Model calibration using this data is therefore not possible. Houses on Mill Lane were found to be affected by flooding during design runs of 1% AEP, as shown in Figure 4.9.37, which provides limited model verification.</p>  <p>Figure 4.9.37: Simulated Flood Extents at Mill Lane for Dec 1954 Event for the 1%, 10% and 0.1% AEP Events</p>
MAR 1947.	<p>Information was found for a flood event which occurred in Navan, Trim, Ballivor and Athboy in March 1947. The flooding followed rapid thaw of snow and ice in</p>

	conjunction with heavy rainfall. In Trim, the River Boyne overtopped its banks and press articles report the river rising to 8 feet above its normal level. The bridge in Trim was submerged and families in low lying areas had to abandon their homes. Some roads were impassable. Reports indicate that approaches to one bridge in the town were blocked by 3 feet of water, indicating that the water level reached approximately 58.83mOD Poolbeg. The reported comments are insufficient in detail in order to be used to calibrate or validate the model. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.
AUG 1905.	Flooding occurred in Trim in August 1905 caused by approximately 36 hours of heavy rainfall in the Trim area. Only outline information for this flood event is available from a press report. This states that damage was caused to crops along the River Boyne; however no further details are available. There are no further details of flood extents, levels or the source of flooding and so this event is unable to be used for model calibration.

Summary of Calibration

Water level and flow records from gauging stations 07005, 07007 and 07041 were analysed in order to determine the flood frequency of historical events at the Trim AFA. It should be noted that flow data for flood events is generally beyond the reliable limit for the rating at these stations however and should be treated with caution.

Section 4.9.5(5) provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided and generally good agreement has been observed.

A mass balance calculation was carried out on the model to make sure that the total volume of water entering and leaving the model at the upstream and downstream boundaries balances the quantity of water remaining in the model domain at the end of a simulation. Refer to Chapter 3.11 for details of acceptable limits. The mass error in the 1% AEP design run was found to be 0.29%, which lies well within acceptable limits.

Very little detailed information is available relating to fluvial flooding within the Trim AFA, and a number of changes which affect the hydrodynamics of the modelled watercourses have been made to the area in recent years such as the replacement of Watergate Street Bridge in 2005. As a result, detailed model calibration was not possible. The model Q-h relationship was calibrated to the rating curves at hydrometric gauges 07005, 07007 and 07041 and good agreement at these locations was achieved. Good qualitative support for the model results was also achieved from the limited historical flooding information available. Despite the limited calibration and verification data, the model is considered to be performing satisfactorily for design event simulation.

(2) Post Public Consultation Updates:

All recorded comments were investigated following informal public consultation and formal S.I. public consultation periods in 2015. Following on from the public consultation period, general model updates were applied to refine model resolution and improve model stability, mapping issued as Final reflects these changes.

(3) Standard of Protection of Existing Formal Defences:

None.

(4) Gauging Stations:**(a) Station 07007_RPS, River Boyne**

This gauging station is located at the upstream end of the Trim model. All spot gaugings marked as 'data post logger removal, rating B' (post 1980) fall within 400mm of the modelled Q-h, the required range for a MPW. Data before this point is not regarded for this rating curve match as the change of a data logger due to drainage works leaves an inconsistency in the spot gauge zero and so these spot gaugings were no longer representative. It appears that a low flow control point has been missed in surveying, causing the offset of the modelled graph from the OPW Rating Equations curve (see Figure 4.9.38 below). The limit of reliable rating stated by OPW for this station is 17m³/s from 01/10/72. Therefore, flow values above this have been extrapolated and should be treated with caution. The closest match between the OPW rating curve and the modelled rating curve is up to this flow value, where it can then be seen to vary to a higher degree past this. At flows higher than 40m³/s a very different pattern can be seen, with the model showing hysteresis occurring. There are two bridges upstream of the gauge, one immediately so and a further bridge downstream within close proximity which are the cause of the hysteresis effect due to attenuation of water. As this gauging station lies within a MPW this rating curve match can be accepted.

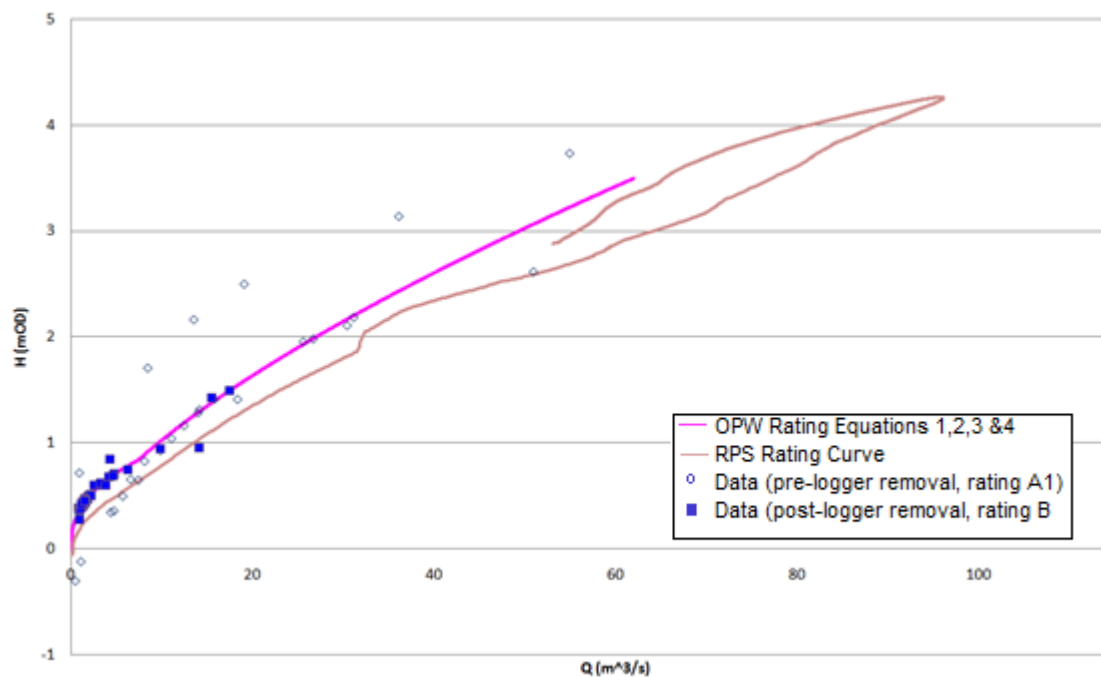


Figure 4.9.38: Comparison of Modelled Rating Curves with Existing Rating and Spot Gaugings, Station 07007

(b) Station 07005_RPS, River Boyne

A gauging station is located in Trim with bridges located immediately upstream and downstream. A rating review was carried out in order to support the calibration of the model. Refer to IBE0600Rp0025_HA07_Hydrology Report_F01 for full details of this rating review. A good correlation was found between the spot gaugings and the model output, however, for low flows simulated the water level is approximately 100mm lower. Limit of reliable rating is $130\text{m}^3/\text{s}$ so flow values above this have been extrapolated and should be treated with caution and from this point a variation can be seen from the RPS curve (see Figure 4.9.39 below). All spot gaugings pre 1977 were excluded as channel dredging in the Boyne had been carried out and these spot gaugings were no longer viewed as being representative.

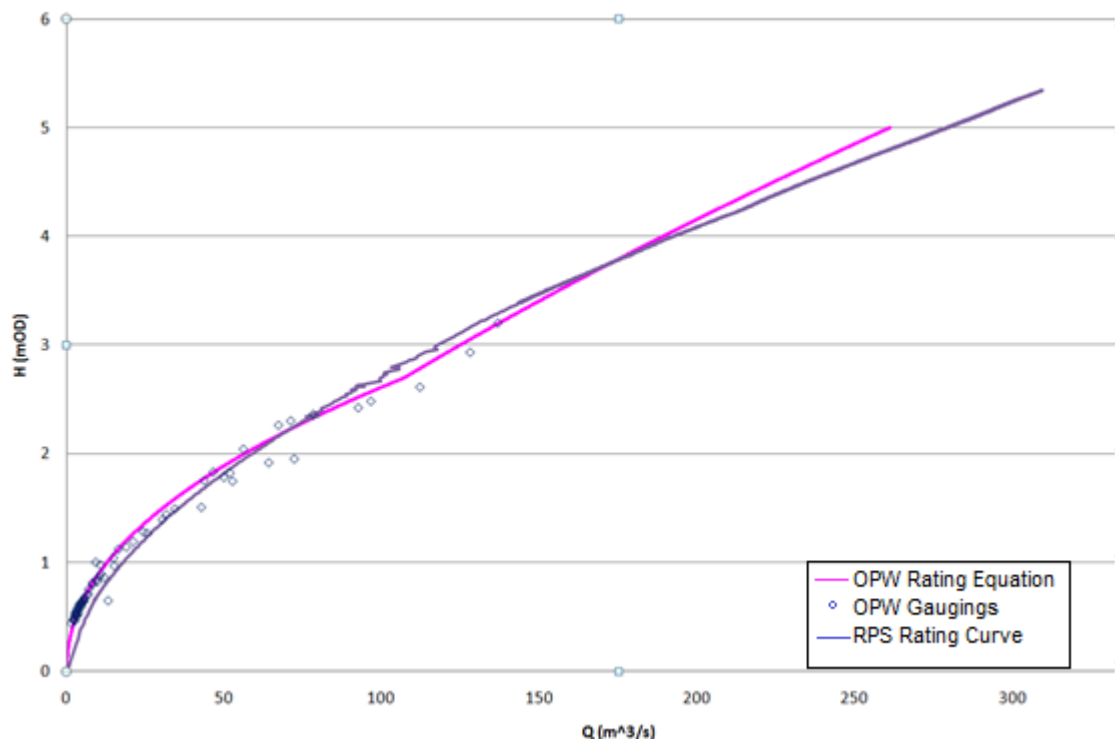


Figure 4.9.39: Comparison of Modelled Rating Curves with Existing Rating and Spot Gaugings, Station 07005

(c) Station 07041_RPS, River Boyne

This gauging station at the downstream end of the Trim model had no spot gauges provided so the model curve was matched only to the EPA Rating Equation curve (see Figure 4.9.40 below). A maximum depth variance of 0.07m can be observed and the model Q-h is a very good match.

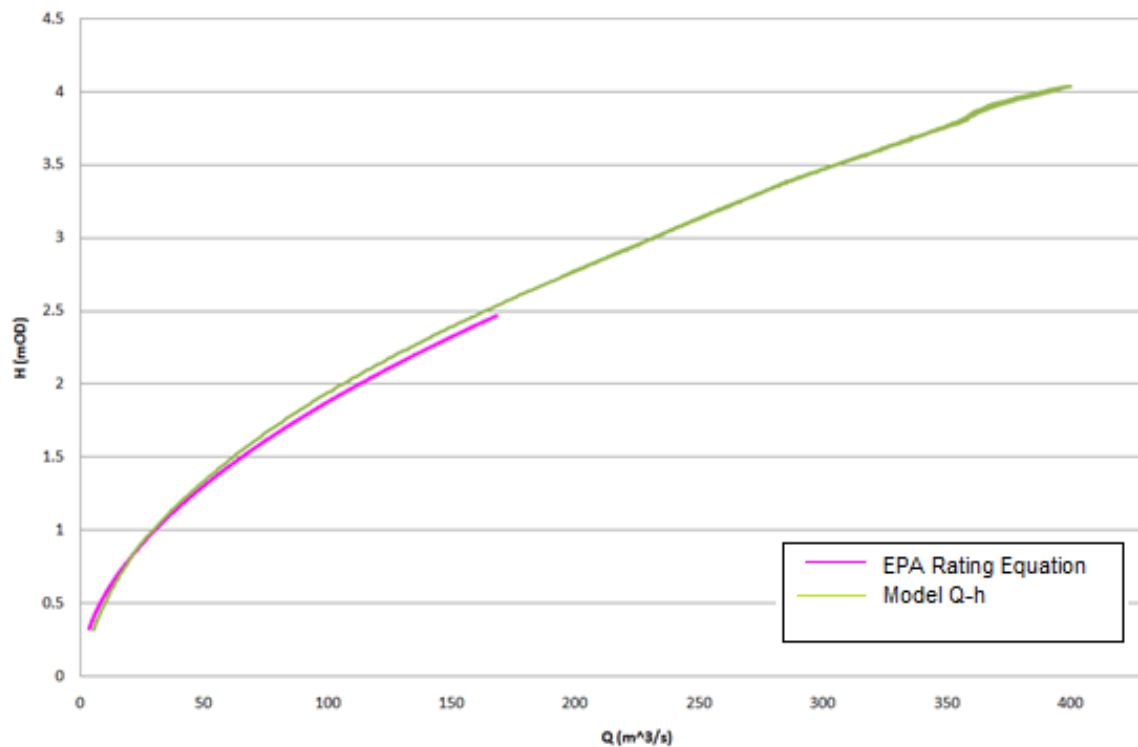


Figure 4.9.40: Comparison of Modelled Rating Curves with Existing Rating, Station 07041

(5) Validation with MIKE NAM:

(a) Trim (07005)

As outlined in Section 3.5.3, NAM modelling provides a further layer of simulated hydrometric data for calibration of the hydraulic models. Flood events which are outside the continuous flow record period at a gauge are available through the simulated time series flow data where this modelling was undertaken. Some key historical flood events recorded in section 4.9.5 (1) have insufficient data to be populated in table 4.9.4, namely the November 2002 flood, the November 2000 flood, the December 1968 flood and any other flood event earlier.

The difference between the modelled peak water level achieved at the peak discharge estimated by MIKE NAM and the observed peak water level (where available) is provided in Table 4.9.4. The difference is less than 545mm for each event. This comparison is provided for information purposes only, and has not been used during the calibration or validation of the model. Model calibration is based on observed data, which takes precedence over the data provided from the NAM hydrological model. Further details on the NAM model are provided in the Hydrology Report.

Table 4.9.4: Comparison of Observed, Hydrologically (NAM) and Hydraulically Modelled Data for Flood Events

Observed Peak			MIKE FLOOD Simulated Peak at Observed WL	MIKE NAM Simulated Peak	MIKE FLOOD Simulated Peak at NAM Discharge	Water Level Difference
Flood Event	Water Level (mOD)	Discharge (m ³ /s)	Discharge (m ³ /s)	Discharge (m ³ /s)	Water Level (mOD)	(m)
18/08/2008 (50% to 10% AEP)	53.211	130.252	123.59	-	-	-
08/01/2005 (50% to 10% AEP)	53.139	125.901	119.194	156.95	53.676	0.537
1/02/1995 (50% AEP)	53.296	130.655	128.753	154.196	53.639	0.343
12/06/1993 (50% to 10% AEP)	53.423	138.389	137.76	180.94	53.968	0.545
28/12/1978 (50% to 10% AEP)	53.23	129.708	124.758	127.12	53.269	0.039

(6) Comparison of Flows:

Table 4.9.5 provides details of the flow in the model at every HEP intermediate check point, modelled tributary and gauging station. These flows have been compared with the hydrology flow estimation and a percentage difference provided.

Table 4.9.5: Modelled Flows and Check Flows

River Name & Chainage	Peak Water Flows			
	AEP	Check Flow (m3/s)	Model Flow (m3/s)	Diff (%)
TRIM TRIBUTARY 1 - BOYNE 2111.26	10%	11.37	11.49	1.08
07_909_3	1%	19.71	18.37	-6.82
	0.1%	32.93	27.35	-16.95
TRIM TRIBUTARY 2 - BOYNE 886.449	10%	0.58	0.45	-21.90
07_1609_3	1%	1.08	0.86	-20.74
	0.1%	1.97	1.55	-21.22
TRIM TRIBUTARY 3 - BOYNE 2932.62	10%	18.22	18.36	0.78
07_908_4	1%	30.36	29.27	-3.60
	0.1%	48.82	48.89	0.14
BLACKFRIARY TRIBUTARY - BOYNE 2809.65	10%	1.82	1.48	-18.46
07_601_6	1%	3.41	2.78	-18.36
	0.1%	6.18	5.05	-18.27
STONEHALL TRIBUTARY 1 - BOYNE 1420.47	10%	1.10	1.31	19.45
07_10000_1	1%	2.06	2.46	19.37
	0.1%	3.74	4.35	16.31
STONEHALL TRIBUTARY 2 - BOYNE 1668.71	10%	0.53	0.59	10.38
07_20000_1	1%	0.99	1.10	11.01
	0.1%	1.80	1.99	10.61
RIVER BOYNE 23836.7	10%	55.25	56.94	3.06
07_1517_5_RPS	1%	78.08	77.74	-0.44
	0.1%	106.78	106.1	-0.64
RIVER BOYNE 45463	10%	187.96	158.67	-15.58
07005_RPS	1%	266.71	227.14	-14.83
	0.1%	366.87	323.83	-11.73

The percentage difference can be seen to progressively increase for the greater return periods at 07_909_3 on Trim Tributary 1 - Boyne. This can be explained by the presence of flooding in this area of the model. There is very little flooding in the 10% AEP, which can be seen by a very close match between check flow and model flow. The variance arises with the greater return periods, which correlates with the fact that flooding increases throughout these different model runs, particularly in the 0.1% AEP as flood

water spills into Trim Tributary 3. This occurs from water leaving Trim tributary 1 and flowing in low lying topography over the floodplain to pour into the downstream end of Trim Tributary 3.

At check point 07_1609_3 on Trim Tributary 2 – Boyne, a substantial percentage difference can be observed, however, this is relative to flows being small and so a variance of only $0.13\text{m}^3/\text{s}$ is the cause of a 21.9% difference. A small degree of flooding at the downstream end of the river has contributed to this also.

A very good match at check point 07_908_4 on Trim Tributary 3 – Boyne is met for the all return periods.

The flows at the downstream boundary on Blackfriary Tributary – Boyne have variances of approximately 18% for each, with a small amount of flooding occurring at the downstream extent of the river. Flow differences of $0.34\text{m}^3/\text{s}$ and $1.13\text{m}^3/\text{s}$ between check flow and model flow produce the 18% variance, a seemingly large percentage difference that is exaggerated by low flow values. 07_1517_5_RPS on the River Boyne is well matched, with a maximum variance of 3.06%. At check point 07_1609_3 on Trim Tributary 2 – Boyne, a substantial percentage difference can be observed, however, this is relative to flows being small and so a variance of only $0.42\text{m}^3/\text{s}$ is the cause of a 21.22% difference. A small degree of flooding at the downstream end of the river has contributed to this also.

At check point 07_10000_1 on Stonehall Tributary 1 – Boyne the percentage difference for the 10% and 1% AEP events is 19.45% and 19.37% respectively, with a 16.31% variance for the 0.1% AEP event. There is a small degree of flooding on the channel for the shorter return periods with the difference between check flow and model flow being exaggerated by the flows being small. For 0.1% AEP the percentage difference is 16.31% as stated but the model flow is actually $4.35\text{m}^3/\text{s}$, which is higher than the $3.74\text{m}^3/\text{s}$ check flow. From the flood maps, flooding occurs at the downstream end of the river which would indicate that the model flow should be less than check flow, however, the River Boyne at the very downstream extent of Stonehall Tributary 1 backs up into it and so this provides the increase in flow.

On River Boyne, at 07005_RPS the percentage difference can be seen to decrease from 15.58% in the 10% AEP event to 11.73% in the 0.1% AEP event. There is a lot of flooding along the River Boyne, increasing with the greater return periods, and this pattern of flow can then be explained by flood waters out of bank being delayed in returning back to the channel and continuing to flow downstream, with more water taking a longer time to do so.

(7) Other Information:

None.

4.9.6 Hydraulic Model Assumptions, Limitations and Handover Notes

(1) Hydraulic Model Assumptions:

- (a) A hot-start file has been used in the 1D model component during all design runs. This hotstart file simulates baseflow conditions in all watercourses within the Trim model.
- (b) Top-up flow between 07_1517_5_RPS and 07005_RPS has been delayed by three hours

changing the peak from occurring at 03:00:00 to 06:00:00 on 2/1/2013, to coincide with the peak of the 07007_RPS inflow in the Boyne and therefore increase the downstream flow for the 07005_RPS gauging station.

- (c) An extremely low initial flow on channel 0725, Stonehall Tributary 2-Boyne, caused unsteadiness in the model. Therefore, up until 1/1/13 10:30:00 at 07_20000_U for the input to this channel, the flow has been increased to 0.1m³/s.
- (d) The in-channel roughness coefficients were selected based on normal bounds using photographs delivered as part of the channel and structure survey.
- (e) Culvert 17I on Butter Stream Trib (0727) was surveyed at the upstream inlet but length was unknown, however the surveyors traced the culvert and confirmed that it flows into reach 0726. Therefore, after observation of data and maps also, the culvert was entered as being approximately 224.6m long with the upstream face double piped, 1m diameter each. The downstream face at 82J was entered as the only cross-section along the culvert with a single pipe of 1.6m diameter. This decision was based on the double piped face being the most limiting to flow and so the larger 1.6m diameter single pipe was included only at the downstream cross-section 0726_00082J at chainage 655.613 on Butter Stream (0726) to simulate the most critical event in this area. There was no interpolated cross-section for the transition, with the cross-section immediately upstream of 82J, section 0727_00017I at chainage 730 on Butter Stream Trib (0727), being the double piped face. The invert level of the pipe at the outlet was also raised from 62.12mOD Malin to 62.5mOD Malin in order to improve model stability. This was considered to be a conservative assumption as it would have the effect of slightly reducing the culvert's capacity.
- (f) For Bridge 8093D at chainage 23577.994 on the River Boyne where the small arch on the far left hand side was removed from the model, the location of this arch (far off to the left hand side from the channel itself) was causing an instability in the MIKE model. This resulted in flickering of the water level affecting the maximum water level reached and removing the arch proved the only solution to provide full stability of the structure. Modelling the presence of the arch was deemed unnecessary as it is not connected to the river channel and at its raised level on the left river bank and the wall and fence built up in front of it, even in extreme flood situations it is unlikely to be flooded, with an assumption of little to no effect if the situation did occur.
- (g) Weir 6159W on the River Boyne is not included in the network file as it was causing instability issues in the model simulations; however the cross-sections are still remaining so the small rise in bed level is still providing the required backwater.
- (h) Three cross-sections have been input to the network file where a rise occurs in the bed level and they were required to be entered as weirs for stability: 0701_05534_acting as a weir, 0701_05570_acting as a weir and 0701_06057_acting as a weir.

(2) Hydraulic Model Limitations and Parameters:

- a) The cross-section and Network files are identical for all design run simulations. The parameters within the HD parameter file are also identical.
- b) Where the watercourses are modelled as 1D-2D, markers have been located at the top of the bank of the cross-sections within the 1D model to denote the point at which water is transferred from the 1D model to the 2D domain. Where the watercourses are modelled as 1D only, the floodplain is contained within the cross-section in the 1D model, as the markers are placed at the cross-section extents.
- c) During model construction, some instability was detected. A review of the model schematisation was conducted in an attempt to remove or reduce the impact of these instabilities. This involved minor adjustments to cross-section locations around structures in order to improve model resolution. Once this process had been completed, remaining instabilities were caused during the transfer of flow from MIKE 11 to MIKE 21. A review of the lateral links was conducted to ensure they were appropriate. After this process was completed, a review of the lateral link parameters was conducted to resolve remaining instabilities, resulting in the selection of the parameters identified in the table below.
- d) An overall time step of 1 second has been selected for all run scenarios.
- e) In HD Parameters the delta value was set to 0.85 to improve stability.
- f) The Zeta Min value was set to 0.2.
- g) Inter1Max value was set to 50.
- h) In the MIKE11 set-up, a hot-start file was created. This file simulates initial baseflow conditions.
- i) At structure 0701_08093D on the River Boyne the small far left arch has been removed for modelling purposes. This will have no effect on the smaller flood events and little to none for the thousand year flood.
- j) A weir structure was removed from the .nwk file (cross section 0701_06160W) at chainage 42911 on the River Boyne (as stated in Section 4.9.3.1), as it was not hydraulically significant and was a source of instability within the model. The drop in bed level over the weir remains part of the .xns file.
- k) Grid spacing of 5m was applied throughout the model. This resolution was selected as it allows the area of interest to be modelled in sufficient detail whilst also maintaining good computational performance of the model.
- l) There are some instabilities within the model, however they do not affect the maximum water level. Following a review of the model, it was not possible to eradicate these instabilities completely, so their significance and impact on the model results was reviewed. The instabilities do not affect the peak water level as stated and no erroneous out-of-bank flooding is caused,

therefore it was concluded that they do not have a significant impact on model results. An example is shown in Figure 4.9.41 during the 0.1% AEP event at chainage 33786 along the River Boyne.

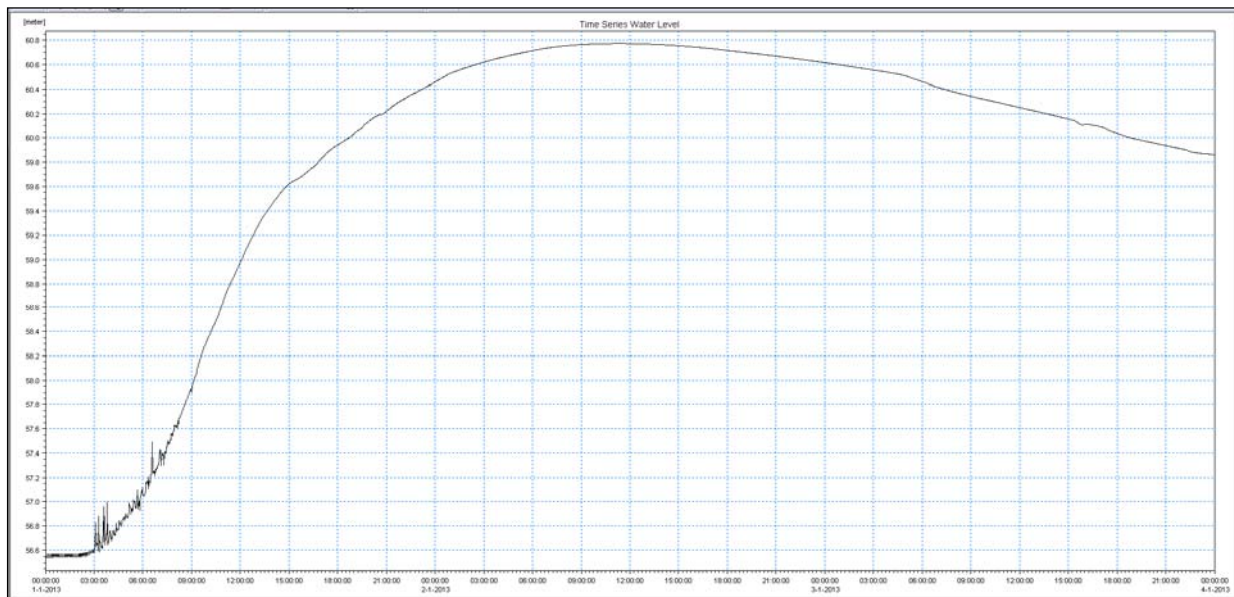


Figure 4.9.41: Water Level Profile for 0.1% AEP Design Run at 0701_07073E

MIKE 11

Timestep (seconds)	1
Wave Approximation	High Order Fully Dynamic
Delta	0.85

MIKE 21

Timestep (seconds)	1
Drying / Flooding depths (metres)	0.02/0.03
Eddy Viscosity (and type)	0.17 (Constant value, Flux based)

MIKE FLOOD

Link Exponential Smoothing Factor (where non-default value used)	1
Lateral Link Depth Tolerance (m) (where non-default value used)	0.1

(3) Design Event Runs & Hydraulic Model Handover Notes:

- a) The topographical survey information provided by Murphy Surveys for HA07 has been supplemented with topographical survey information recorded for the purpose of the Gauging Station Rating Reviews (in order to facilitate model calibration at the gauging station location).
- b) Post-processing of the flood extents to remove those bridges and culverts which do not flood for the modelled return period from the flood extent (as discussed in Chapter 3). There was no further post-processing of the flood extents required.
- c) Trim is influenced only by fluvial sources.
- d) The 10% AEP, 1% AEP and 0.1% AEP fluvial events were simulated to determine the flood risk throughout the Trim AFA. The flood extents show land and property being flooded, particularly on the River Boyne running through Trim town itself and continuing with out of bank flooding downstream of this.
- e) The weir at 6159W on the River Boyne was removed from the network file as a weir for model run stability. The cross-sections remain in the model, so the channel remains the same and water flow will act in the same manner as before.
- f) An assumption was made on the length and shape of the culvert that connects Athboy Tributary 2 with Athboy Tributary 1. This was represented in the model at sections 17I to 82J and has been discussed in Section 4.9.6.1.
- g) Considerable flooding was found to occur from the River Boyne upstream of the Trim AFA during all modelled design runs, as shown in Figure 4.9.42. This flooding is due to insufficient channel capacity. Large areas of agricultural land are affected. The R161 was also found to flood during design runs of 10% AEP or greater, and the R156 was found to flood during design runs of 0.1% AEP. Up to approximately 10 properties were found to be affected during design runs of 0.1% AEP as well.

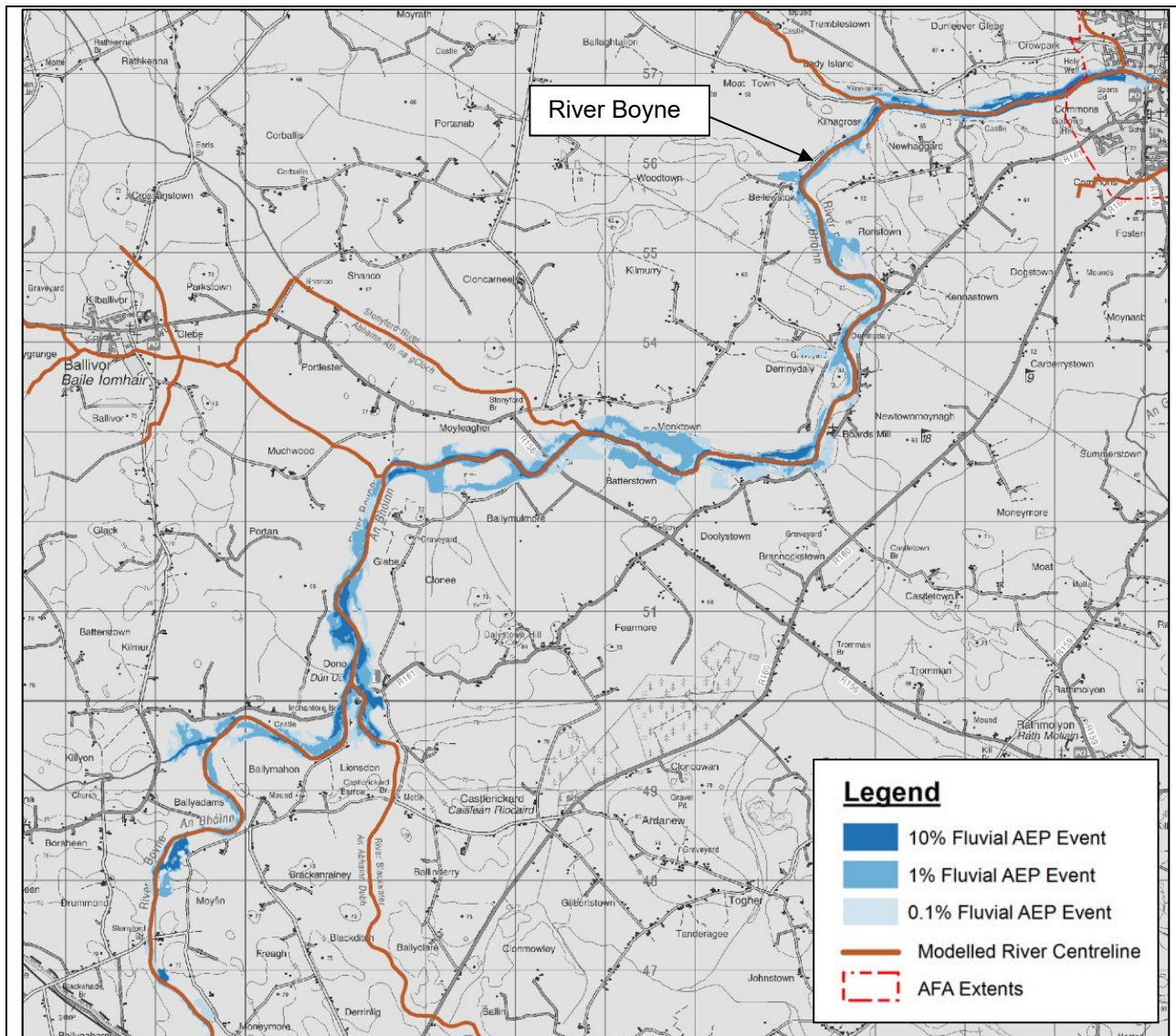


Figure 4.9.42: Modelled Flood Extents of the River Boyne (upstream of the Trim AFA)

- h) Flooding from the River Boyne was found to occur in the Trim AFA due to insufficient channel capacity and the restrictive effect of a number of bridges including 5885D and 5726D. Flooding was found to affect the Pitch and Putt golf course during design runs of 10% AEP or greater, as shown in Figure 4.9.32. A number of properties on Mill Lane were also found to flood during design runs of 1% AEP, as shown in Figure 4.9.37.
- i) Flooding was found to occur on the Knightsbrook River during all model design runs due to insufficient channel capacity and the restrictive effect of culvert 121 at chainage 2953. Flooding affects agricultural land and is most significant during design runs of 1% AEP or greater, as shown in Figure 4.9.43. No properties were found to be affected.

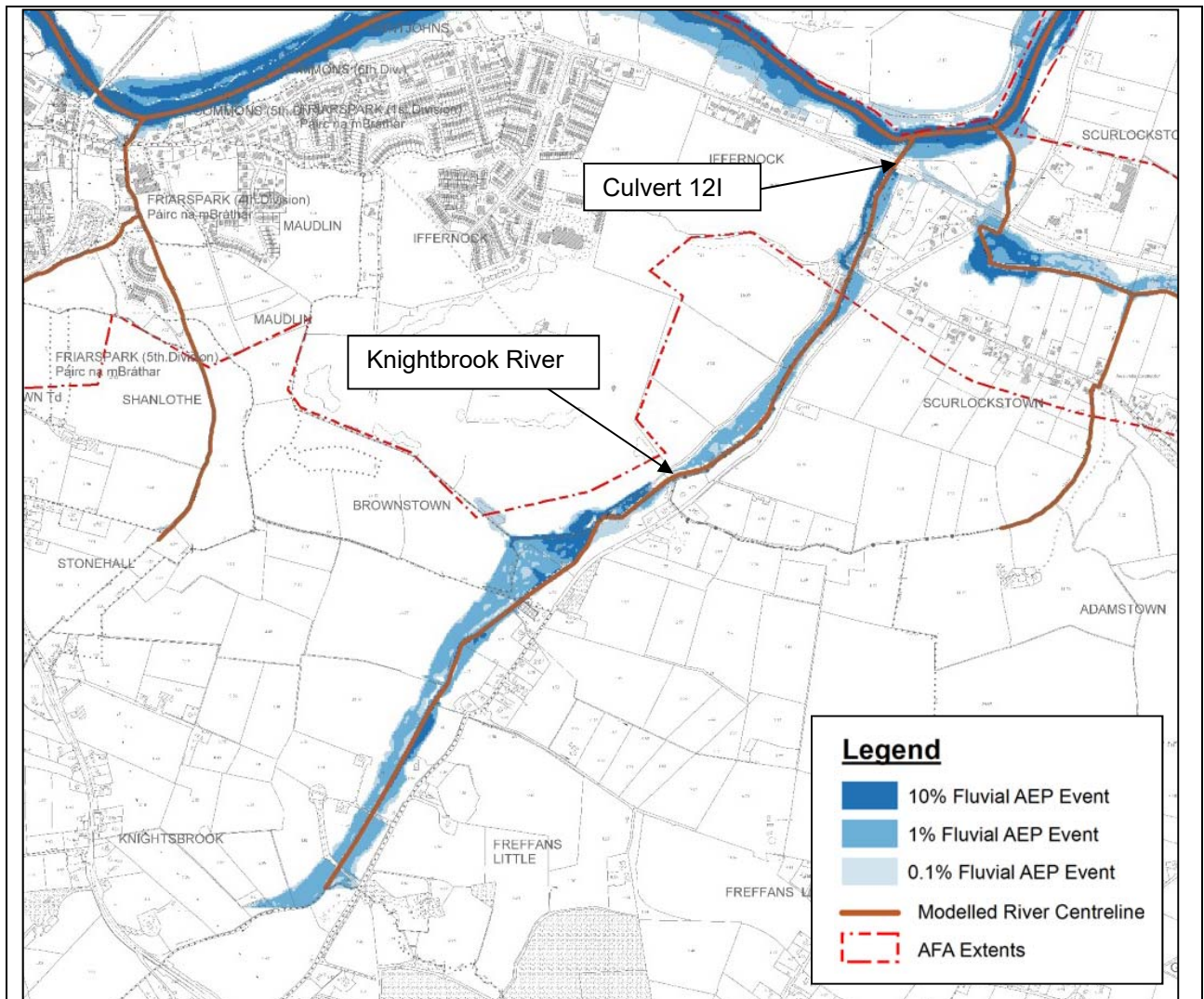


Figure 4.9.43: Modelled Flood Extents of the Knightbrook River

- j) Flooding was found to occur on the Boycetown River during design runs of 10% AEP or greater, as shown in Figure 4.9.44. This flooding is due to insufficient channel capacity and the restrictive effect of culvert 32D which passes under the R154 at chainage 1969. Flooding was mainly found to affect agricultural land, however flooding was also found to affect one property and the R154 during design runs of 0.1% AEP.

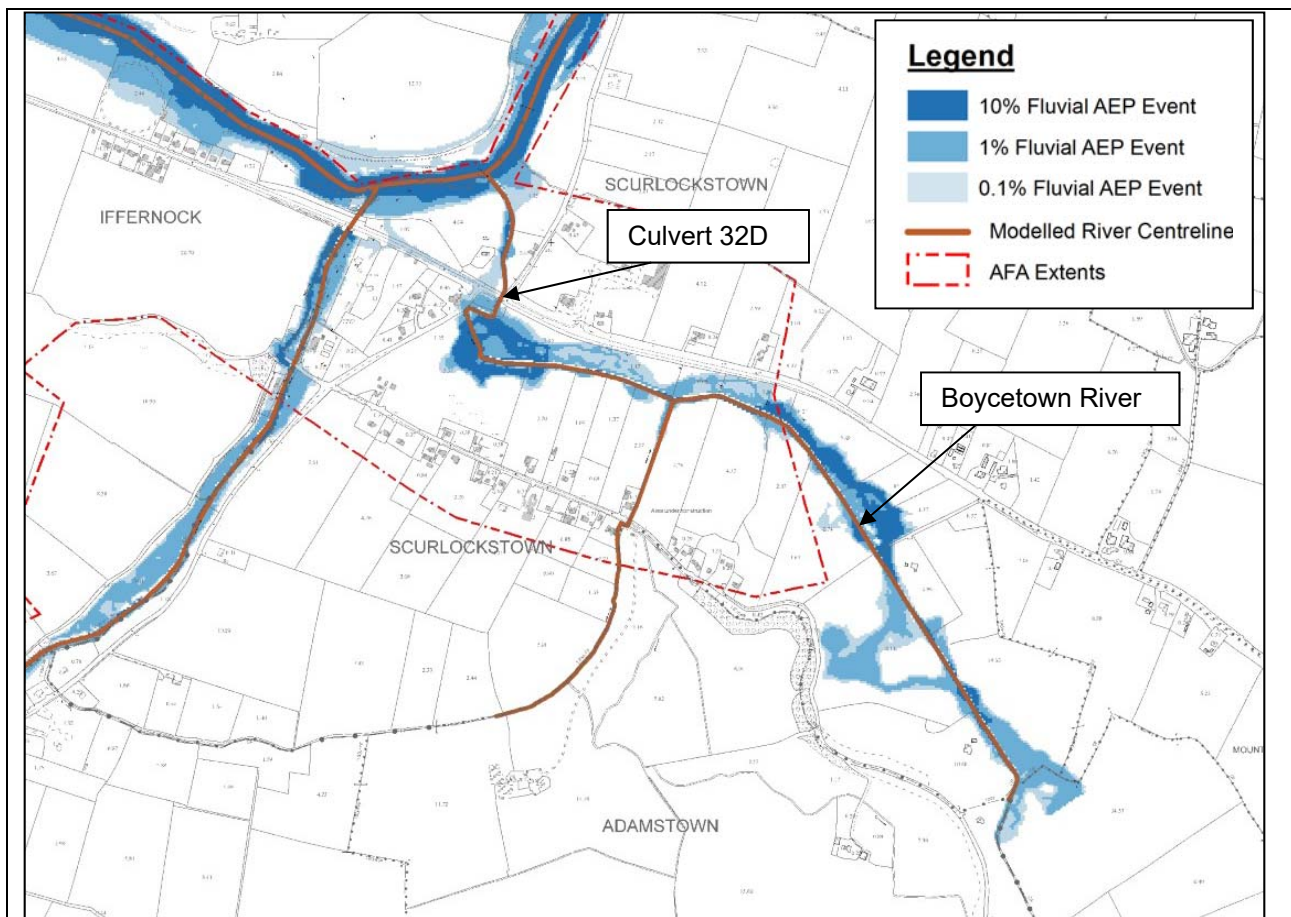


Figure 4.9.44: Modelled Flood Extents of the Boycetown River

- k) In conclusion, the model is considered to be performing well for design event simulation and is supported by historic and hydrometric information. The modelled flood extents cover areas which have a history of flooding and modelled flows correlate well with estimated flows providing confidence that the model adequately represents the hydraulics of the modelled watercourses.
- l) The cross-sections located within the 1D model reaches (between Ch 20043 - 43185 and Ch 50690 - 59096) have been reviewed and extended using the NDHM (in order to prevent 'glass walls' from affecting peak water levels and flows).

(4) Hydraulic Model Deliverables:

Please see Appendix I for a list of all model files provided with this report.

(5) Quality Assurance:

Model Constructed by:	Maria Nixon
Model Reviewed by:	Stephen Patterson
Model Approved by:	Andrew Jackson

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Hydraulic analysis was undertaken in order to identify the location and frequency of flooding within the extents of the HA07 modelled watercourses. The analysis utilised MIKE and Infoworks ICM computational modelling software informed by detailed topographical survey information (channel sections, in-channel/flood defence structures, bathymetric and floodplain), combined with hydrological inputs (riverine inflows and sea levels) and water-level control parameters (such as channel-roughness), to determine flood hazard. A series of flood extent, zone, depth, velocity and risk-to-life maps known collectively as flood hazard maps were generated based on the model results.

The influence of coastal water levels has been modelled by applying an appropriate water level boundary profile to the downstream extent of all coastal river models. Tidal data has been taken from the Irish Coastal Protection Strategy Study (ICPSS). The effects of the sea levels are propagated upstream by the modelling software allowing the interaction of river flows and coastal water levels to be modelled accurately. Model tests included variation in fluvial-tidal joint probability and temporal variations, along with parameters such as eddy viscosity and bed resistance. In some AFAs, relative timings between fluvial and coastal peaks were adjusted to establish the worst case flood outlines, for a particular combination of events.

Key flood events were used in the calibration of each model whereby the model was reviewed in order to make sure historic flooding is accurately represented; the principal model parameters that are reviewed and amended during the model calibration process are:

- Bed and floodplain roughness coefficients;
- Structure roughness and head loss coefficients;
- Timing of hydrographs;
- Magnitude of hydrographs;
- Incorporation of additional survey information (e.g. additional cross-sections or missed structures).

There is good confidence in both the hydrology and hydraulics of the Navan, Trim, Drogheda and Baltray AFAs due to the presence of gauging stations and flood extent verification events.

There is moderate confidence in both the hydrology and hydraulics of the Johnstown Bridge, and Longwood AFAs due to the presence of a gauging station within each model. However, larger flood events should be recorded at the gauging station in order to improve this confidence. There is also moderate confidence in the hydrology and hydraulics of the Mornington AFA. The Mornington River is

not gauged however the Boyne River is and there is good calibration / validation data available in relation recent flood events.

The lack of historical flooding information within the Athboy, Ballivor and Edenderry AFAs gives little quantitative data to calibrate the model to the larger flood events. Whilst the models were calibrated to available spot gaugings and anecdotal information to the best extent possible, overall there is little or poor data to calibrate the model to and observation of more events would be necessary to reduce the uncertainty in model results.

The accuracy of the models representing existing conditions in terms of flood level, depth, extent and flow velocity allows potential flood options to be meaningfully assessed, enabling the appropriate actions/decisions to be taken. The calibrated models were used to simulate present day and future flood hazard conditions and potential options to facilitate the appraisal of possible flood risk management actions and measures.

There were no defence failure scenarios required. Sensitivity tests have been conducted for each model, and reported within Chapter 4. The parameters selected for the sensitivity analysis were dependent on the specific model but generally included:

- roughness coefficients
- critical structure coefficients
- flow inputs
- operation of dynamic structures
- downstream boundary conditions
- representation of buildings in 2D model domain
- timing of tributaries
- flow volume

It was concluded from the sensitivity analysis that the model parameters with the greatest influence is the peak discharge and a change to the 1D/2D roughness coefficients, with all fluvial models being moderately or highly sensitive to these parameters. In addition, the Drogheda, Baltray & Mornington model is sensitive to a change in the downstream boundary condition, the Edenderry and Longwood models are sensitive to hydrograph volume and the Trim model is sensitive to the head loss parameter at a critical structure.

Future potential changes which may affect the outputs of the CFRAM Study were also assessed. Urbanisation and afforestation allowances are applied on a case by case basis as required, the factors

themselves having been derived during the hydrology analysis by looking at historic urbanisation growth indicators and estimating appropriate growth factors for MRFS and HEFS.

There are inherent assumptions, limitations and uncertainty associated with hydraulic modelling, which are detailed for each hydraulic model within Chapter 4. The issues addressed include:

- schematisation decisions regarding out-of-bank flow routes;
- culvert/bridge schematisation (including skew angle considerations);
- sweetening flow assumptions;
- comments and notes throughout to reflect data sources; changes to parameters from default;
- explanation of parameters used that are outside of the expected ranges; and
- any other atypical assumptions made.

The objective of hydraulic analysis is to gain a detailed understanding of the Study area's flood response and mechanisms to assess both flood risk and determine flood risk management solutions. Given the detailed hydraulic modelling analysis of historic flood events, and estimation of design and future flood level, depth, velocity and extent conditions for each AFA within this study, it is concluded that no further hydrodynamic modelling or analysis is required to satisfy the requirements of the project brief. The accuracy of the models representing existing conditions in terms of flood level, depth, extent and flow velocity has allowed the possible benefits of flood options to be meaningfully assessed, allowing the appropriate actions/decisions to be taken.

5.2 RECOMMENDATIONS

Future users of each hydraulic model should be fully aware of the assumptions, limitations, sensitivity and uncertainty (as discussed within this report) when assessing the output from hydraulic model simulations.

It is recommended that should an extreme flood occur in the future, then a comprehensive post-flood survey is completed. The data collected should be used during a review of the hydraulic analysis in order to determine if any model updates are required (and further improve the calibration / validation of the model).

There is poor or moderate confidence in the Johnstown Bridge, Longwood, Mornington, Athboy, Ballivor and Edenderry hydraulic analysis. All of these AFAs, with the exception of Mornington, have been shown to have very low or no flood risk (following completion of the flood risk assessment). In order to improve the confidence in each model, it is recommended a review is undertaken to identify where improvements to the hydrometric network could be made. This would lead to increased data availability which could be used in future hydraulic analysis.

For the Navan AFA, it is recommended that the interaction of fluvial flooding with surface water flooding and the urban drainage network is investigated.

6 REFERENCES

1. EC Directive on the Assessment and Management of Flood Risks (2007/60/EC)
 2. S.I. No. 122/2010 - European Communities (Assessment and Management of Flood Risks) Regulations 2010
 3. Eastern CFRAM Study , Previous Study Review - Phase 1 – Flood Mapping Addendum: Fingal East Meath FRAM Study, Tolka Flood Study and Dodder FRAM Study, IBE0600Rp0017 (RPS, 2013)
 4. Eastern CFRAM Study, HA07 Inception Report, IBE0600Rp0004 (RPS, 2012)
 5. Eastern CFRAM Study, HA07 Hydrology Report, IBE0600Rp0012 (RPS, 2013)
 6. Flood Studies Update Programme – Work Package 1.2 – Estimation of Point Rainfall Frequencies – prepared by Met Eireann for Office of Public Works (October 2007)
 7. Engineers Australia, Australian Rainfall and Runoff, Revision Project 15, Two Dimensional Simulations in Urban Areas, Representation of Buildings in 2D Numerical Flood Models, (Water Research Laboratory, University of New South Wales, 2012)
 8. Flood Studies Update Programme – Work Package 3.4 – Guidance for River Basin Modelling – prepared by JBA for Office of Public Works (May 2010)
 9. Culvert Design and Operation Guide - R168 (CIRIA, 1997)
 10. EurOtop (2007). Wave Overtopping of Sea Defences and Related Structures – Assessment Manual. Eds. Pullen, T., N.W.H Allsop, T. Bruce, , A. Kortenhaus, H. Schüttrumpf & J.W. van der Meer. www.overtopping-manual.com.
 11. Fluvial Design Guide (Environment Agency, 2010)
-

Appendix A

Athboy AFA

Additional Information

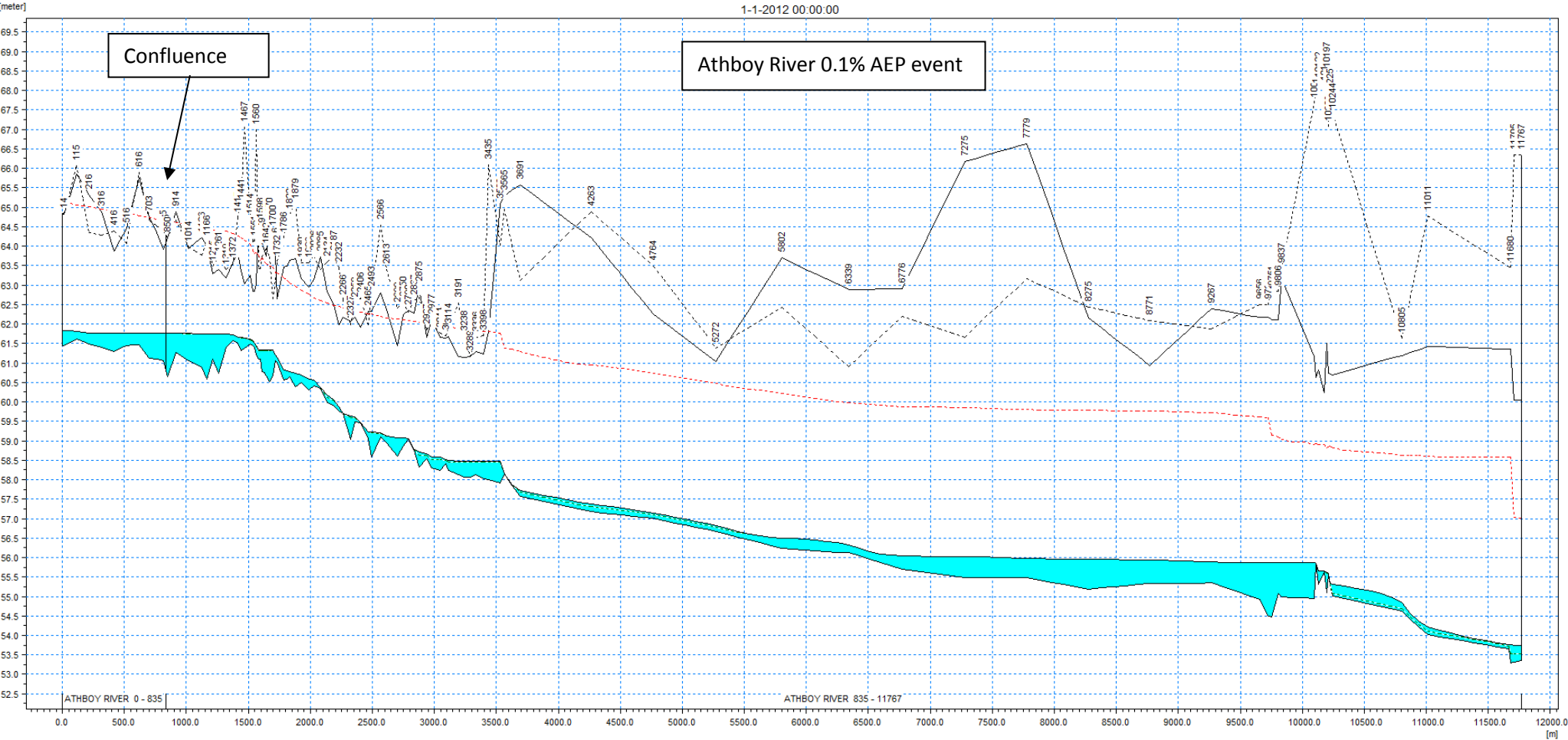
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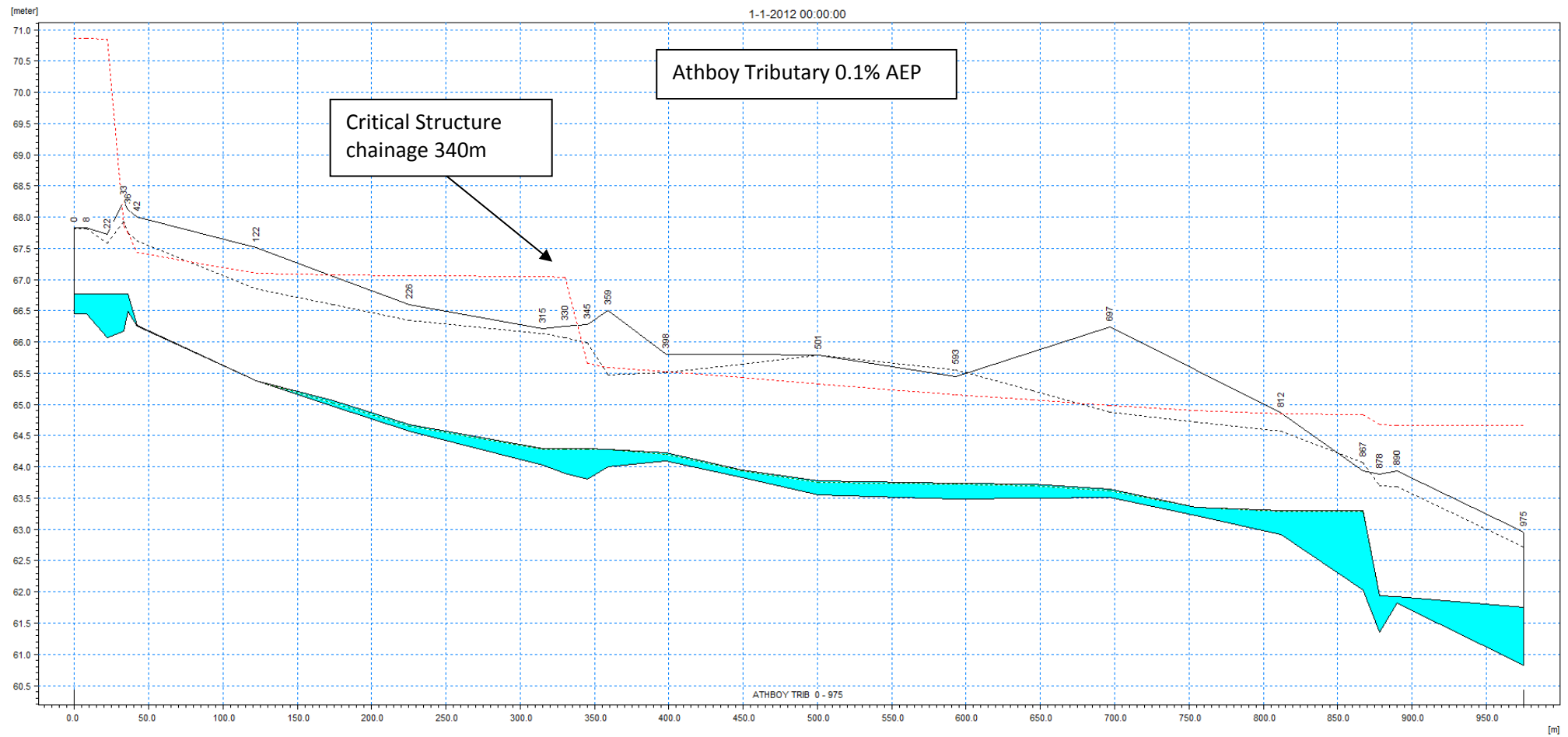
- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

1D Structures

	Branch	Chainage	ID	Length (m)	Opening Shape	Height (m)	Width (m)	Spring Height from invert (m)	Mannings n
1	ATHBOY RIVER	1572.552	0728_01020D_Bridge	15.99	Circular	2.84	6.5	1.76	0.035
2	ATHBOY RIVER	3549.255	0728_00821E_bridge	5.77	Cross Section DB	4.14	5.62	2.89	0.035
3	ATHBOY RIVER	9726.007	0728_00204D_bridge	5.5	Cross Section DB	5.84	7.22	4.63	0.035
4	ATHBOY TRIB	340.769	0729_00066I_culvert	6.64	Cross Section DB	1	-	-	0.02
5	ATHBOY RIVER	11683.5	0728_00007D_bridge	7.22	Cross Section DB	3.65	6.93	2.32	0.075
6	ATHBOY TRIB	27.37	0729_00097I_culvert	10.577	Circular	1	-	-	0.02

River Long Section Profile





Final Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_ATHB3_MF_DES_2_Q2	HA07_ATHB3_M21_DES_2_Q2		
HA07_ATHB3_MF_DES_2_Q5	HA07_ATHB3_M21_DES_2_Q5		
HA07_ATHB3_MF_DES_2_Q10	HA07_ATHB3_M21_DES_2_Q10		
HA07_ATHB3_MF_DES_2_Q20	HA07_ATHB3_M21_DES_2_Q20		
HA07_ATHB3_MF_DES_2_Q50	HA07_ATHB3_M21_DES_2_Q50		
HA07_ATHB3_MF_DES_2_Q100_1	HA07_ATHB3_M21_DES_2_Q100_1		
HA07_ATHB3_MF_DES_2_Q200	HA07_ATHB3_M21_DES_2_Q200		
HA07_ATHB3_MF_DES_2_Q1000_1	HA07_ATHB3_M21_DES_2_Q1000		
HA07_ATHB3_MF_DES_2_Q2_MRFS	HA07_ATHB3_M21_DES_2_Q2_MRFS		
HA07_ATHB3_MF_DES_2_Q5_MRFS	HA07_ATHB3_M21_DES_2_Q5_MRFS		
HA07_ATHB3_MF_DES_2_Q10_MRFS	HA07_ATHB3_M21_DES_2_Q10_MRFS		
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HA07_ATHB3_MF_DES_2_Q50_MRFS	HA07_ATHB3_M21_DES_2_Q50_MRFS		
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HA07_ATHB3_MF_DES_2_Q200_MRFS	HA07_ATHB3_M21_DES_2_Q200_MRFS		
HA07_ATHB3_MF_DES_2_Q1000_1_MRFS	HA07_ATHB3_M21_DES_2_Q1000_MRFS		
HA07_ATHB3_MF_DES_2_Q10_HEFS	HA07_ATHB3_M21_DES_2_Q10_HEFS		
HA07_ATHB3_MF_DES_2_Q100_1_HEFS	HA07_ATHB3_M21_DES_2_Q100_1_HEFS		
HA07_ATHB3_MF_DES_2_Q1000_1_HEFS	HA07_ATHB3_M21_DES_2_Q1000_HEFS		
	BlockedBuildings&Rivers_2		
	Corine1		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_ATHB3_M11_DES_2_Q2	HA07_ATHB3_NWK_DES_07	HA07_ATHB3_XNS_DES_16	HA07_ATHB3_BND_DES_2_Q2
HA07_ATHB3_M11_DES_2_Q5	HA07_ATHB3_NWK_DES_07_1	HA07_ATHB3_XNS_DES_17	HA07_ATHB3_BND_DES_2_Q5
HA07_ATHB3_M11_DES_2_Q10		HA07_ATHB3_XNS_DES_17_Q1000HEFS	HA07_ATHB3_BND_DES_2_Q10
HA07_ATHB3_M11_DES_2_Q20			HA07_ATHB3_BND_DES_2_Q20
HA07_ATHB3_M11_DES_2_Q50			HA07_ATHB3_BND_DES_2_Q50
HA07_ATHB3_M11_DES_2_Q100_1			HA07_ATHB3_BND_DES_2_Q100_1
HA07_ATHB3_M11_DES_2_Q200			HA07_ATHB3_BND_DES_2_Q200
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HA07_ATHB3_M11_DES_2_Q1000_HEFS			HA07_ATHB3_BND_DES_2_Q1000_HEFS

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
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Athboy DS Boundary_1_MRFS			
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Final Model Files - Sensitivity

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MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
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GIS Deliverables - Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u>	Water Level and Flows	<u>Fluvial</u>	<u>Fluvial</u>	<u>Fluvial</u>
E02EXFCD500F0	<u>Fluvial</u>	E02DPFCD500F0	E02VLFCDD500F0	E02RLFCD100F0
E02EXFCD200F0	E02NFCDF0	E02DPFCD200F0	E02VLFCDD200F0	E02RLFCD010F0
E02EXFCD100F0	E02NFMDF0	E02DPFCD100F0	E02VLFCDD100F0	E02RLFCD001F0
E02EXFCD050F0	E02NFHDF0	E02DPFCD050F0	E02VLFCDD050F0	
E02EXFCD020F0		E02DPFCD020F0	E02VLFCDD020F0	
E02EXFCD010F0	Flood Defences (Shapefiles)	E02DPFCD010F0	E02VLFCDD010F0	
E02EXFCD005F0	N/A	E02DPFCD005F0	E02VLFCDD005F0	
E02EXFCD001F0		E02DPFCD001F0	E02VLFCDD001F0	
				Flood Zones (Shapefiles)
	Defended Areas (Shapefiles)			E02ZNA_FCDF0
E02EXFMD500F0	N/A	E02DPFMD500F0		E02ZNB_FCDF0
E02EXFMD200F0		E02DPFMD200F0		
E02EXFMD100F0	Wave Overtopping (Shapefiles)	E02DPFMD100F0		E02ZNA_FMDF0
E02EXFMD050F0	N/A	E02DPFMD050F0		E02ZNB_FMDF0
E02EXFMD020F0		E02DPFMD020F0		
E02EXFMD010F0		E02DPFMD010F0		
E02EXFMD005F0		E02DPFMD005F0		
E02EXFMD001F0		E02DPFMD001F0		
E02EXFHD100F0		E02DPFHD100F0		
E02EXFHD010F0		E02DPFHD010F0		
E02EXFHD001F0		E02DPFHD001F0		
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables - Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
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General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix B

Ballivor AFA

Additional Information

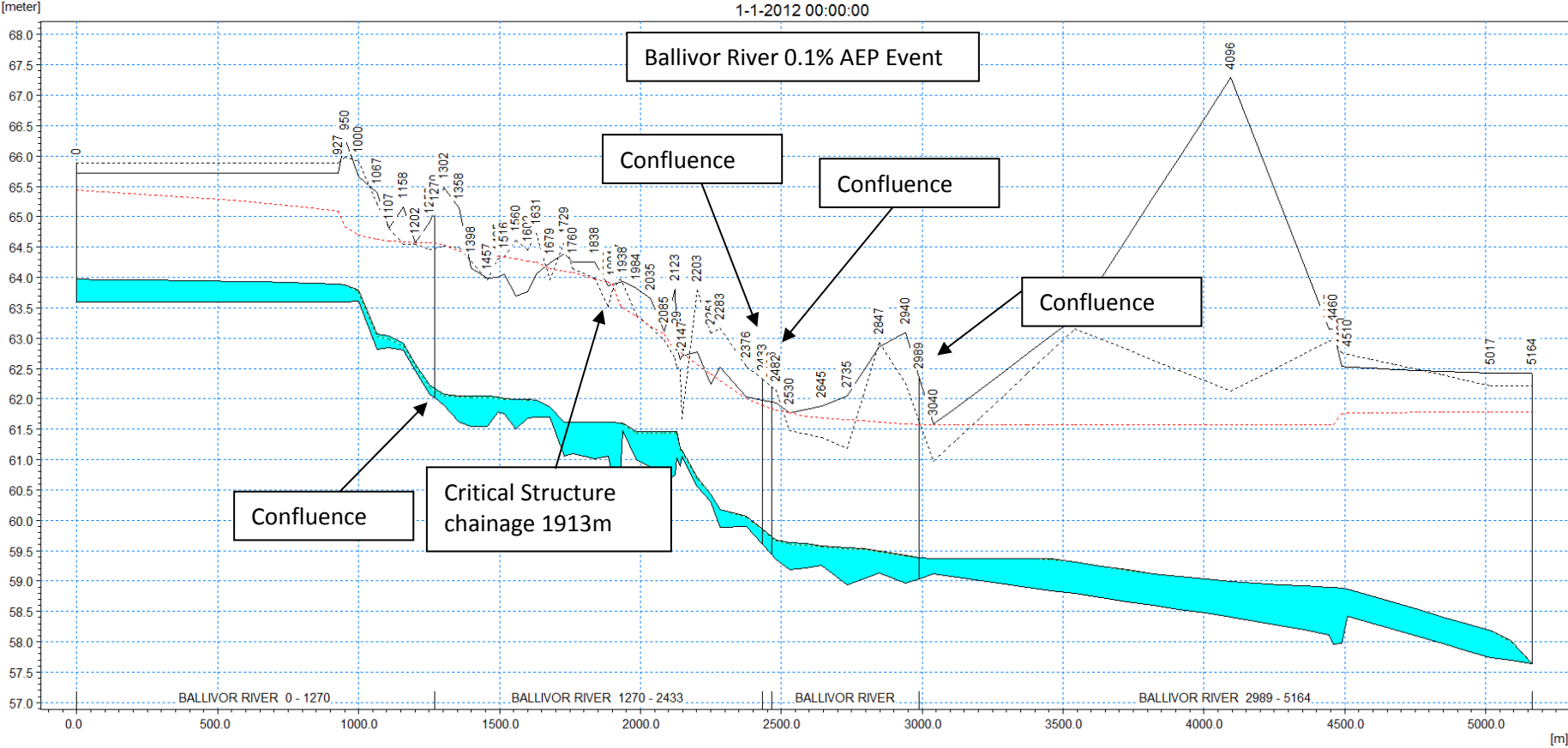
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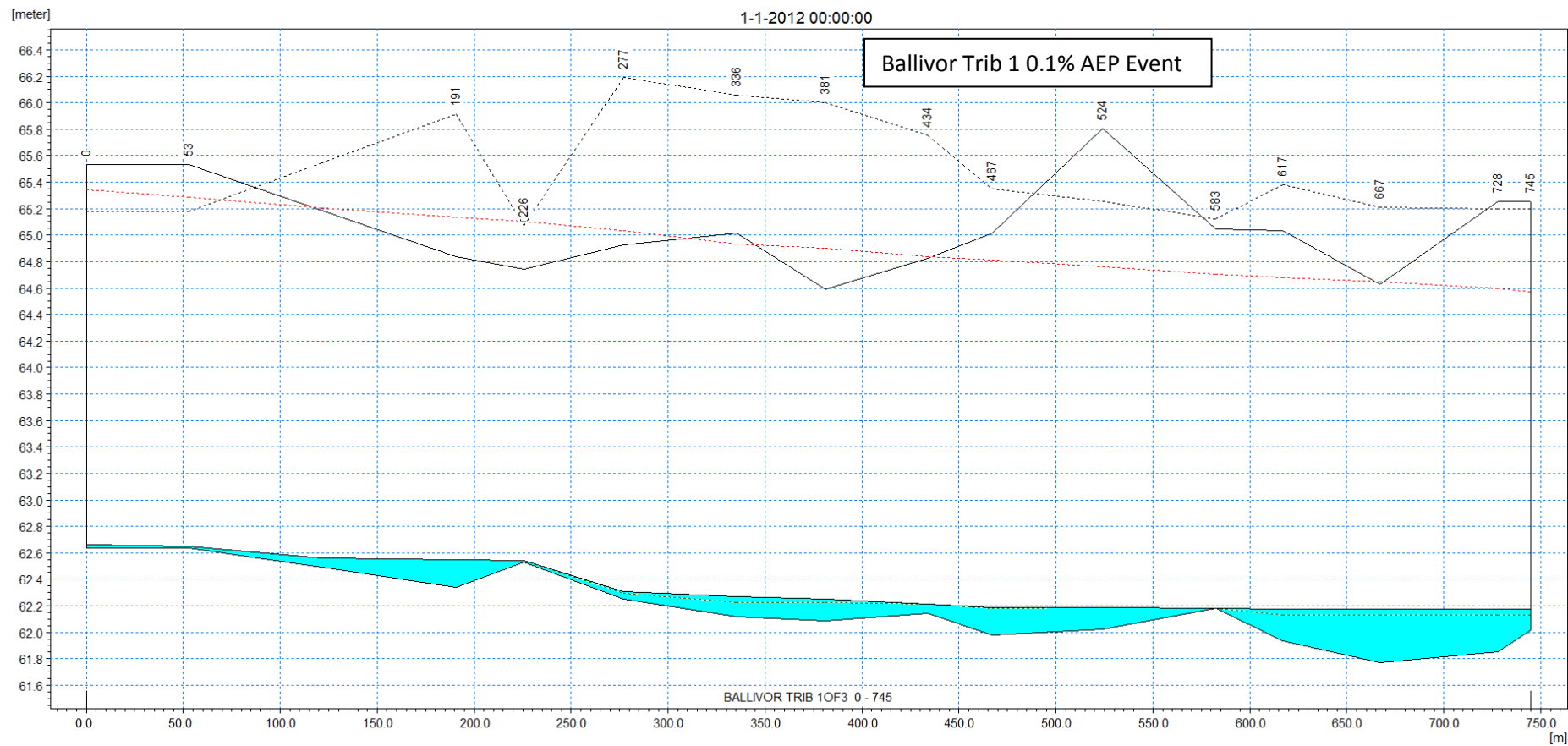
- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

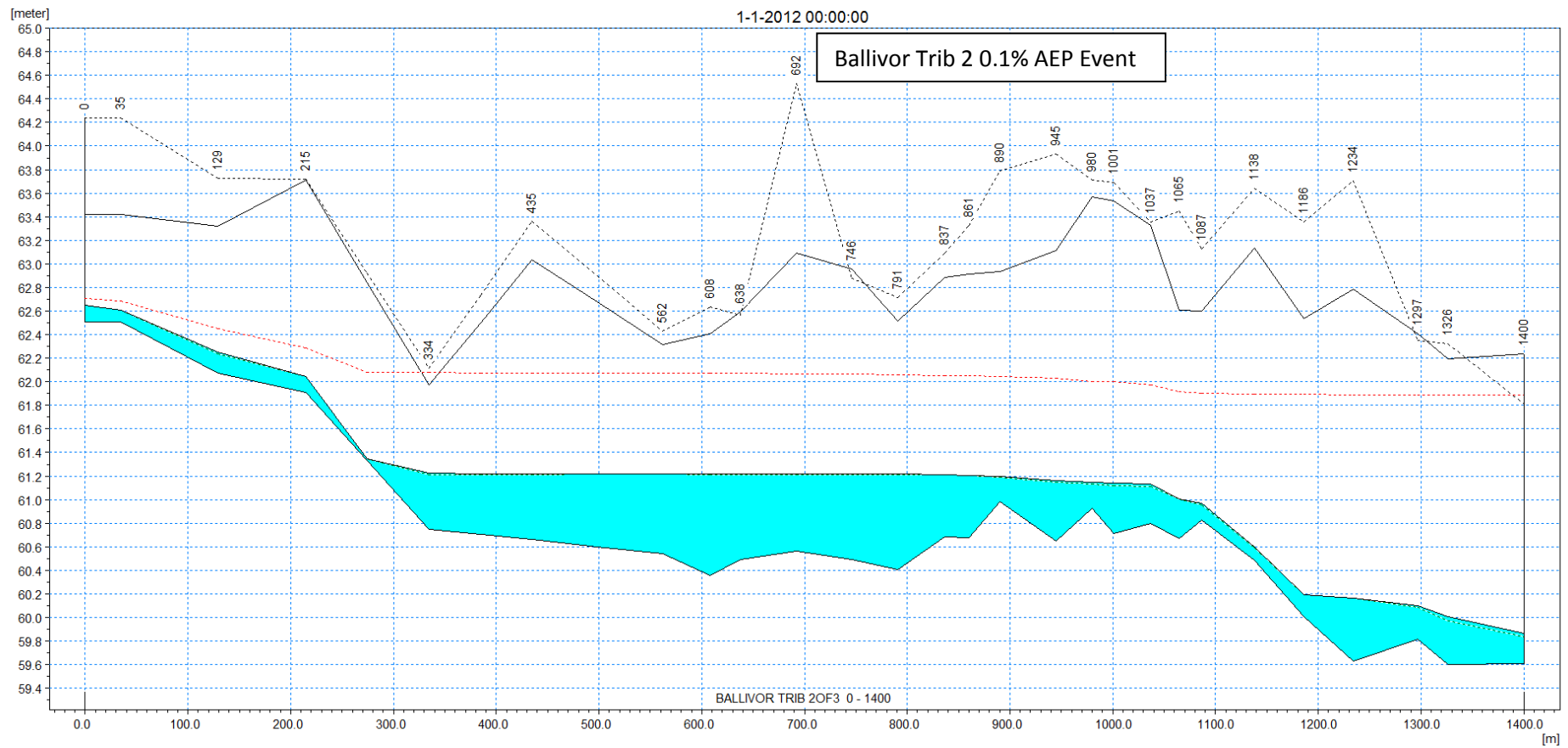
1D Structures

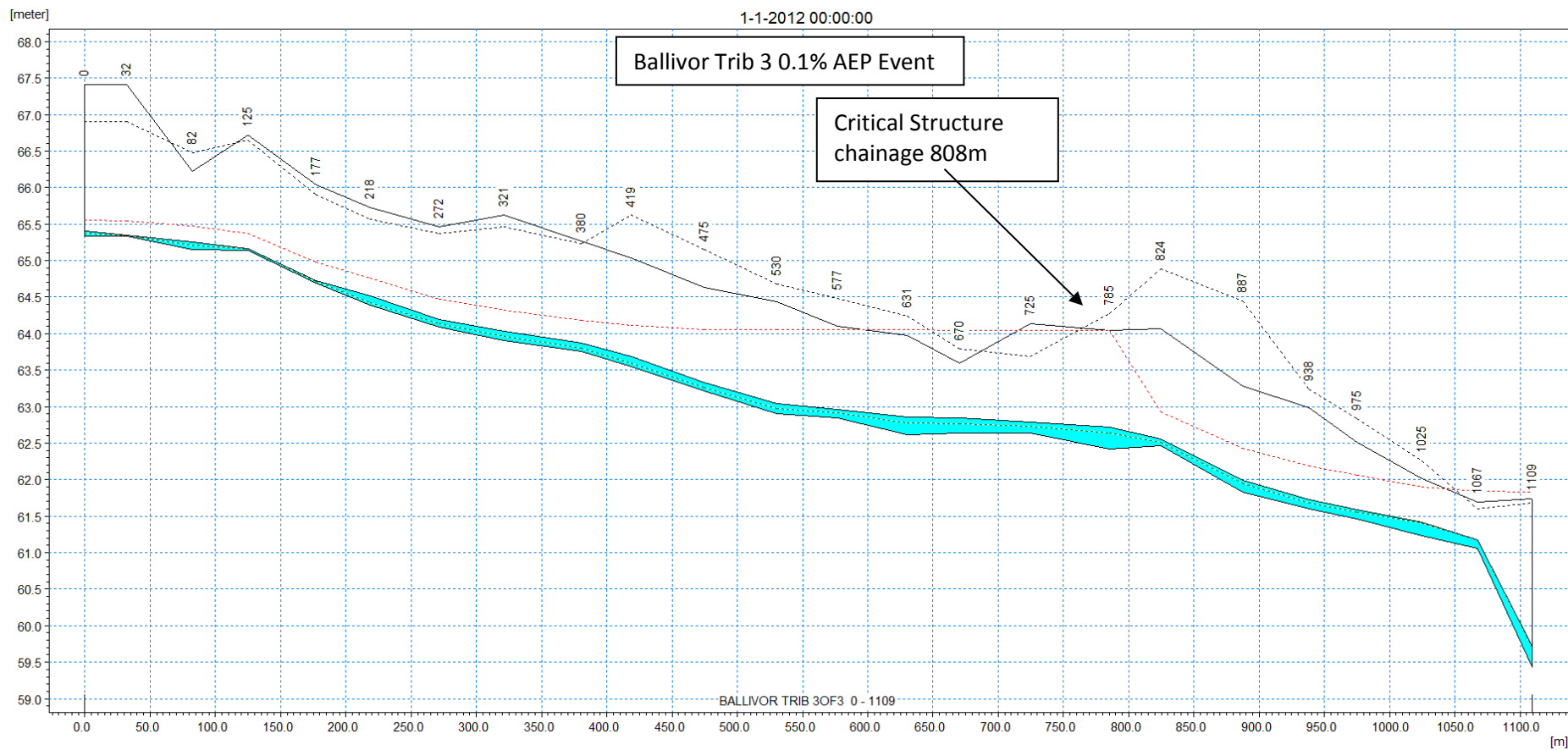
	Branch	Chainage	ID	Length (m)	Opening Shape	Height (m)	Width (m)	Spring Height from invert (m)	Mannings n
1	BALLIVOR RIVER	927.55	0730_00424D	3.98	Cross Section DB	1.67	1.19	1.12	0.05
2	BALLIVOR RIVER	1609.42	0730_00355D	1.08	Cross Section DB	2.42	8.33	-	0.05
3	BALLIVOR RIVER	1913.27	0730_00325D	10.89	Cross Section DB	2.83	2.67	-	0.05
4	BALLIVOR RIVER	4472.61	0730_00069D	7.71	Cross Section DB	2.97	2.31	-	0.05
5	BALLIVOR SPLIT & STONEYFORD RIVER	782.17	0741_00442D	11.68	Cross Section DB	2.68	1.99	-	0.05
6	BALLIVOR SPLIT & STONEYFORD RIVER	1307	0741_00390D	7.12	Cross Section DB	4.52	7.46	2.84	0.035
7	BALLIVOR SPLIT & STONEYFORD RIVER	4422.72	0741_00078D	6.6	Cross Section DB	4.94	7.65	2.97	0.04
8	BALLIVOR TRIB 2OF3	839.52	0731_00059D	2.38	Cross Section DB	1.75	2.28	-	0.05
9	BALLIVOR TRIB 2OF3	968	0731_00047I	21.36	Circular	1.2	-	-	0.013
10	BALLIVOR TRIB 2OF3	1024	0731_00041I	18.38	Circular	1.4	-	-	0.013
11	BALLIVOR TRIB 2OF3	1048.95	0731_00039D	15.67	Cross Section DB	1.625	2.62	-	0.035
12	BALLIVOR TRIB 3OF3	808.28	0732_00032Em	12.49	Cross Section DB	0.29	0.5	-	0.035

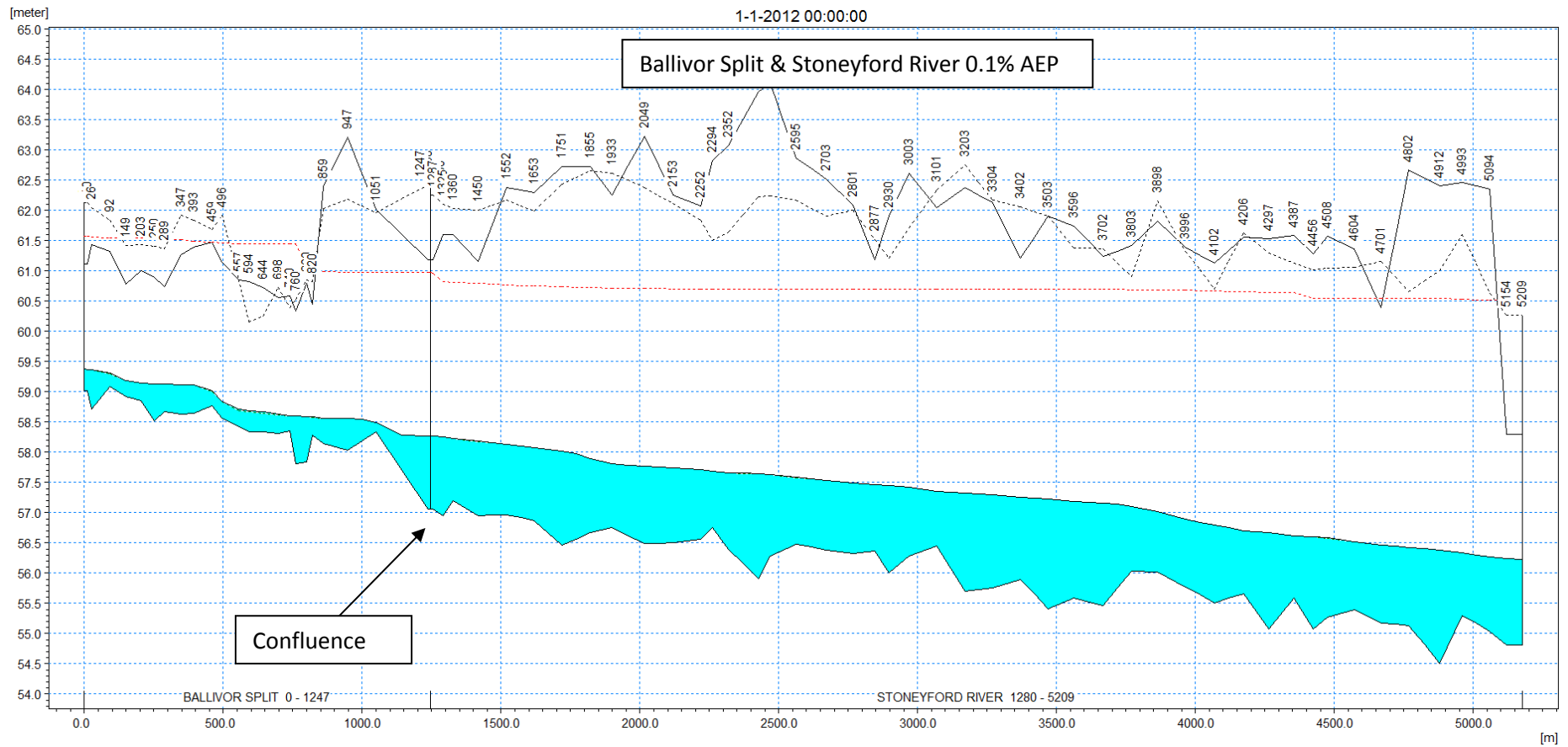
River Long Section Profile











Final Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_BALL2_MF_DES_Q2	HA07_BALL2_M21_DES_Q2		N/A
HA07_BALL2_MF_DES_Q5	HA07_BALL2_M21_DES_Q5		
HA07_BALL2_MF_DES_Q10	HA07_BALL2_M21_DES_Q10		
HA07_BALL2_MF_DES_Q20	HA07_BALL2_M21_DES_Q20		
HA07_BALL2_MF_DES_2_Q50	HA07_BALL2_M21_DES_Q50		
HA07_BALL2_MF_DES_Q100	HA07_BALL2_M21_DES_Q100		
HA07_BALL2_MF_DES_Q200	HA07_BALL2_M21_DES_Q200		
HA07_BALL2_MF_DES_Q1000	HA07_BALL2_M21_DES_Q1000		
HA07_BALL2_MF_DES_Q2_MRFS	HA07_BALL2_M21_DES_Q2_MRFS		
HA07_BALL2_MF_DES_Q5_MRFS	HA07_BALL2_M21_DES_Q5_MRFS		
HA07_BALL2_MF_DES_Q10_MRFS	HA07_BALL2_M21_DES_Q10_MRFS		
HA07_BALL2_MF_DES_Q20_MRFS	HA07_BALL2_M21_DES_Q20_MRFS		
HA07_BALL2_MF_DES_2_Q50_MRFS	HA07_BALL2_M21_DES_Q50_MRFS		
HA07_BALL2_MF_DES_Q100_MRFS	HA07_BALL2_M21_DES_Q100_MRFS		
HA07_BALL2_MF_DES_Q200_MRFS	HA07_BALL2_M21_DES_Q200_MRFS		
HA07_BALL2_MF_DES_Q1000_MRFS	HA07_BALL2_M21_DES_Q1000_MRFS		
HA07_BALL2_MF_DES_Q10_HEFS	HA07_BALL2_M21_DES_Q10_HEFS		
HA07_BALL2_MF_DES_Q100_HEFS	HA07_BALL2_M21_DES_Q100_HEFS		
HA07_BALL2_MF_DES_Q1000_HEFS_1	HA07_BALL2_M21_DES_Q1000_HEFS		
	HA07_BALL2_DFS_BATHY		
	HA07_BALL2_DFS_FPR		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_BALL2_M11_DES_Q2	HA07_BALL2_NWK_DES	HA07_BALL2_XNS_DES	HA07_BALL2_BND_DES_Q2_1
HA07_BALL2_M11_DES_Q5		HA07_BALL2_XNS_DES_1	HA07_BALL2_BND_DES_Q5_1
HA07_BALL2_M11_DES_Q10			HA07_BALL2_BND_DES_Q10_1
HA07_BALL2_M11_DES_Q20			HA07_BALL2_BND_DES_Q20_1
HA07_BALL2_M11_DES_Q50			HA07_BALL2_BND_DES_Q50_1
HA07_BALL2_M11_DES_Q100			HA07_BALL2_BND_DES_Q100_1
HA07_BALL2_M11_DES_Q200			HA07_BALL2_BND_DES_Q200_1
HA07_BALL2_M11_DES_Q1000			HA07_BALL2_BND_DES_Q1000_1
HA07_BALL2_M11_DES_Q2_MRFS			HA07_BALL2_BND_DES_Q2_MRFS
HA07_BALL2_M11_DES_Q5_MRFS			HA07_BALL2_BND_DES_Q5_MRFS
HA07_BALL2_M11_DES_Q10_MRFS			HA07_BALL2_BND_DES_Q10_MRFS
HA07_BALL2_M11_DES_Q20_MRFS			HA07_BALL2_BND_DES_Q20_MRFS
HA07_BALL2_M11_DES_Q50_MRFS			HA07_BALL2_BND_DES_Q50_MRFS
HA07_BALL2_M11_DES_Q100_MRFS			HA07_BALL2_BND_DES_Q100_MRFS
HA07_BALL2_M11_DES_Q200_MRFS			HA07_BALL2_BND_DES_Q200_MRFS
HA07_BALL2_M11_DES_Q1000_MRFS			HA07_BALL2_BND_DES_Q1000_MRFS
HA07_BALL2_M11_DES_Q10_HEFS			HA07_BALL2_BND_DES_Q10_HEFS
HA07_BALL2_M11_DES_Q100_HEFS			HA07_BALL2_BND_DES_Q100_HEFS
HA07_BALL2_M11_DES_Q1000_HEFS			HA07_BALL2_BND_DES_Q1000_HEFS

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_BALL2_DFS0_Q2	HA07_BALL2_HD_DES_Q2	HA07_BALL2_M11_DES_Q2	HA07_BALL2_M11_DES_Q2
HA07_BALL2_DFS0_Q5	HA07_BALL2_HD_DES_Q5	HA07_BALL2_M11_DES_Q5	HA07_BALL2_M11_DES_Q5
HA07_BALL2_DFS0_Q10_2	HA07_BALL2_HD_DES_Q10	HA07_BALL2_M11_DES_Q10	HA07_BALL2_M11_DES_Q10
HA07_BALL2_DFS0_Q20	HA07_BALL2_HD_DES_Q20	HA07_BALL2_M11_DES_Q20	HA07_BALL2_M11_DES_Q20
HA07_BALL2_DFS0_Q50	HA07_BALL2_HD_DES_Q50	HA07_BALL2_M11_DES_2_Q50	HA07_BALL2_M11_DES_2_Q50
HA07_BALL2_DFS0_Q100_2	HA07_BALL2_HD_DES_Q100	HA07_BALL2_M11_DES_Q100	HA07_BALL2_M11_DES_Q100
HA07_BALL2_DFS0_Q200	HA07_BALL2_HD_DES_Q200	HA07_BALL2_M11_DES_Q200	HA07_BALL2_M11_DES_Q200
HA07_BALL2_DFS0_Q1000_2	HA07_BALL2_HD_DES_Q1000	HA07_BALL2_M11_DES_Q1000	HA07_BALL2_M11_DES_Q1000
HA07_BALL2_DFS0_Q2_MRFS	HA07_BALL2_HD_DES_Q2_MRFS	HA07_BALL2_M11_DES_Q2_MRFS	HA07_BALL2_M11_DES_Q2_MRFS
HA07_BALL2_DFS0_Q5_MRFS	HA07_BALL2_HD_DES_Q5_MRFS	HA07_BALL2_M11_DES_Q5_MRFS	HA07_BALL2_M11_DES_Q5_MRFS
HA07_BALL2_DFS0_Q10_1_MRFS	HA07_BALL2_HD_DES_Q10_MRFS	HA07_BALL2_M11_DES_Q10_MRFS	HA07_BALL2_M11_DES_Q10_MRFS
HA07_BALL2_DFS0_Q20_MRFS	HA07_BALL2_HD_DES_Q20_MRFS	HA07_BALL2_M11_DES_Q20_MRFS	HA07_BALL2_M11_DES_Q20_MRFS
HA07_BALL2_DFS0_Q50_MRFS	HA07_BALL2_HD_DES_Q50_MRFS	HA07_BALL2_M11_DES_2_Q50_MRFS	HA07_BALL2_M11_DES_2_Q50_MRFS
HA07_BALL2_DFS0_Q100_1_MRFS	HA07_BALL2_HD_DES_Q100_MRFS	HA07_BALL2_M11_DES_Q100_MRFS	HA07_BALL2_M11_DES_Q100_MRFS
HA07_BALL2_DFS0_Q200_MRFS	HA07_BALL2_HD_DES_Q200_MRFS	HA07_BALL2_M11_DES_Q200_MRFS	HA07_BALL2_M11_DES_Q200_MRFS
HA07_BALL2_DFS0_Q1000_1_MRFS	HA07_BALL2_HD_DES_Q1000_MRFS	HA07_BALL2_M11_DES_Q1000_MRFS	HA07_BALL2_M11_DES_Q1000_MRFS
HA07_BALL2_DFS0_Q10_1_HEFS	HA07_BALL2_HD_DES_Q10_HEFS	HA07_BALL2_M11_DES_Q10_HEFS	HA07_BALL2_M11_DES_Q10_HEFS
HA07_BALL2_DFS0_Q100_1_HEFS	HA07_BALL2_HD_DES_Q100_HEFS	HA07_BALL2_M11_DES_Q100_HEFS	HA07_BALL2_M11_DES_Q100_HEFS
HA07_BALL2_DFS0_Q1000_1_HEFS	HA07_BALL2_HD_DES_Q1000_HEFS	HA07_BALL2_M11_DES_Q1000_HEFS_1	HA07_BALL2_M11_DES_Q1000_HEFS_1
HA07_BALL2_DFS0_BoyneWL_Q2_1	ballivorHDmap		
HA07_BALL2_DFS0_BoyneWL_Q5_1			
HA07_BALL2_DFS0_BoyneWL_Q10			
HA07_BALL2_DFS0_BoyneWL_Q20_1			
HA07_BALL2_DFS0_BoyneWL_Q50_1			
HA07_BALL2_DFS0_BoyneWL_Q100			
HA07_BALL2_DFS0_BoyneWL_Q200_1			
HA07_BALL2_DFS0_BoyneWL_Q1000			
HA07_BALL2_DFS0_BoyneWL_Q2_MRFS			

MIKE 11 - DFS0 FILE Continued	MIKE 11 – HD FILE Continued	MIKE 11 - RESULTS FILE Continued	HD RESULTS FILE Continued
HA07_BALL2_DFS0_BoyneWL_Q5_MRFS HA07_BALL2_DFS0_BoyneWL_Q10_MRFS HA07_BALL2_DFS0_BoyneWL_Q20_MRFS HA07_BALL2_DFS0_BoyneWL_Q50_MRFS HA07_BALL2_DFS0_BoyneWL_Q100_MRFS HA07_BALL2_DFS0_BoyneWL_Q200_MRFS HA07_BALL2_DFS0_BoyneWL_Q1000_MRFS HA07_BALL2_DFS0_BoyneWL_Q10_HEFS HA07_BALL2_DFS0_BoyneWL_Q100_HEFS HA07_BALL2_DFS0_BoyneWL_Q1000_HEFS			

Final Model Files - Sensitivity

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_BALL2_MF_SEN_1_Q100_flow HA07_BALL2_MF_SEN_Q100_fpr HA07_BALL2_MF_SEN_Q100_fv HA07_BALL2_MF_SEN_Q100_hl_1 HA07_BALL2_MF_SEN_Q100_rough HA07_BALL2_MF_SEN_Q100_wlbnd	HA07_BALL2_M21_SEN_1_Q100_flow HA07_BALL2_M21_SEN_Q100_fpr HA07_BALL2_M21_SEN_Q100_fv HA07_BALL2_M21_SEN_Q100_hl_1 HA07_BALL2_M21_SEN_Q100_rough HA07_BALL2_M21_SEN_Q100_wlbnd HA07_BALL2_DFS_BATHY HA07_BALL2_DFS_FPR HA07_BALL2_DFS_FPR_rough		HA07_BALL2_M21_SEN_1_Q100_flow HA07_BALL2_M21_SEN_Q100_fpr HA07_BALL2_M21_SEN_Q100_fv HA07_BALL2_M21_SEN_Q100_hl_1 HA07_BALL2_M21_SEN_Q100_rough HA07_BALL2_M21_SEN_Q100_wlbnd

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_BALL2_M11_SEN_1_Q100_flow HA07_BALL2_M11_SEN_Q100_fpr HA07_BALL2_M11_SEN_Q100_fv HA07_BALL2_M11_SEN_Q100_hl_1 HA07_BALL2_M11_SEN_Q100_rough HA07_BALL2_M11_SEN_Q100_wlbnd	HA07_BALL2_NWK_DES HA07_BALL2_NWK_SEN_1_Q100_flow HA07_BALL2_NWK_SEN_hl_1	HA07_BALL2_XNS_DES HA07_BALL2_XNS_SEN_1_Q100_flow HA07_BALL2_XNS_SEN_hl_1 HA07_BALL2_XNS_SEN_rough	HA07_BALL2_BND_DES_Q100_1 HA07_BALL2_BND_SEN_1_Q100_flow HA07_BALL2_BND_SEN_Q100_1_fv HA07_BALL2_BND_SEN_Q100_1_wlbnd

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_BALL2_DFS0_Q100_2 HA07_BALL2_DFS0_SEN_Q100_flow HA07_BALL2_DFS0_SEN_Q100_fv HA07_BALL2_DFS0_BoyneWL_Q100 HA07_BALL2_DFS0_BoyneWL_Q100_MRFS HA07_BALL2_DFS0_SEN_BoyneWL_Q100_fv	HA07_BALL2_HD_SEN_1_Q100_flow HA07_BALL2_HD_SEN_Q100_fpr HA07_BALL2_HD_SEN_Q100_fv HA07_BALL2_HD_SEN_Q100_hl_1 HA07_BALL2_HD_SEN_Q100_rough HA07_BALL2_HD_SEN_Q100_wlbnd	HA07_BALL2_M11_SEN_1_Q100_flow HA07_BALL2_M11_SEN_Q100_fpr HA07_BALL2_M11_SEN_Q100_fv HA07_BALL2_M11_SEN_Q100_hl_1 HA07_BALL2_M11_SEN_Q100_rough HA07_BALL2_M11_SEN_Q100_wlbnd	N/A

GIS Deliverables - Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u>	Water Level and Flows	<u>Fluvial</u>	<u>Fluvial</u>	<u>Fluvial</u>
E06EXFCD500F0	<u>Fluvial</u>	E06DPFCD500F0	E06VLFC0500F0	E06RLFCD100F0
E06EXFCD200F0	E06NFCDF0	E06DPFCD200F0	E06VLFC0200F0	E06RLFCD010F0
E06EXFCD100F0	E06NFMDF0	E06DPFCD100F0	E06VLFC0100F0	E06RLFCD001F0
E06EXFCD050F0	E06NFHDF0	E06DPFCD050F0	E06VLFC0050F0	
E06EXFCD020F0		E06DPFCD020F0	E06VLFC0020F0	
E06EXFCD010F0	Flood Defences (Shapefiles)	E06DPFCD010F0	E06VLFC0010F0	
E06EXFCD005F0	N/A	E06DPFCD005F0	E06VLFC0005F0	
E06EXFCD001F0		E06DPFCD001F0	E06VLFC0001F0	
	Defended Areas (Shapefiles)			Flood Zones (Shapefiles)
E06EXFMD500F0	N/A	E06DPFMD500F0		E06ZNA_FCDF0
E06EXFMD200F0		E06DPFMD200F0		E06ZNB_FCDF0
E06EXFMD100F0	Wave Overtopping (Shapefiles)	E06DPFMD100F0		
E06EXFMD050F0	N/A	E06DPFMD050F0		E06ZNA_F MDF0
E06EXFMD020F0		E06DPFMD020F0		E06ZNB_F MDF0
E06EXFMD010F0		E06DPFMD010F0		
E06EXFMD005F0		E06DPFMD005F0		
E06EXFMD001F0		E06DPFMD001F0		
E06EXFHD100F0		E06DPFHD100F0		
E06EXFHD010F0		E06DPFHD010F0		
E06EXFHD001F0		E06DPFHD001F0		
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables - Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E03RIFCD100F0 E03RIFCD010F0 E03RIFCD001F0 E03RIFMD100F0 E03RIFMD010F0 E03RIFMD001F0	<u>Fluvial</u> E07_RTFC001_F0 E07_RTMD001_F0	<u>Fluvial</u> E03RDFCD001F0 E03RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix C

Drogheda & Baltray AFA Additional Information

List of background information included:

- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

1D Structures

Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
Boyne	32580	0701_03258	9.9	ARCH	6.29	9.09	3.77	0.045
			9.9	ARCH	6.16	9.3	3.77	0.045
			9.9	ARCH	6.3	9.22	3.77	0.045
			9.9	ARCH	6.19	9.36	3.77	0.045
			9.9	ARCH	6.41	9.13	3.77	0.045
			9.9	ARCH	6.12	8.51	3.77	0.045
Boyne	26200	0701_02620	9.9	ARCH	4.71	6	2.5	0.045
			9.9	ARCH	2.79	4.8	1.34	0.045
			9.9	ARCH	3.47	4.85	1.62	0.045
			9.9	ARCH	4.2	5.68	1.75	0.045
			9.9	ARCH	4.2	5.95	1.75	0.045
			9.9	ARCH	3.86	4.73	1.7	0.045
			9.9	ARCH	3.82	4.79	1.68	0.045
			9.9	ARCH	3.99	5.31	1.65	0.045
			9.9	ARCH	3.91	5.33	1.74	0.045
			9.9	ARCH	2.92	3.71	1.64	0.045
Boyne	18310	0701_01831	9.5	RECT	7.06	64	NA	0.045
Boyne	12780	0701_01278	8.3	RECT	8.38	36.3	NA	0.045
Boyne	8510	0701_00851	25.3	RECT	12.303	38.36	NA	0.045
			25.3	RECT	12.303	33.5	NA	0.045
Boyne	8290	0701_00829	14.8	RECT	6.71	10.73	NA	0.045
			14.8	RECT	7.61	11.44	NA	0.045
			14.8	RECT	7.3	11.32	NA	0.045
			14.8	RECT	6.26	11.07	NA	0.045
Boyne	8060	0701_00806	15.8	RECT	6.45	7.74	NA	0.045
			15.8	RECT	9.85	12.4	NA	0.045

Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
			15.8	RECT	10.39	17.6	NA	0.045
			15.8	RECT	6.91	12.3	NA	0.045
Boyne	7840	0701_00784	14.9	RECT	8.55	16.3	NA	0.045
			14.9	RECT	8.65	15.48	NA	0.045
Boyne	7710	0701_00771	2.7	ARCH	9.82	40.94	8.64	0.045
Boyne	6990	0701_00699	11.2	RECT	25	39.22	NA	0.045
			11.2	RECT	34.03	77.86	NA	0.045
			11.2	RECT	30	39.5	NA	0.045
Baltray	60	0702_00006	8	ARCH	1.78	1.58	1.13	0.05
		0702_00006	8	ARCH	2.04	1.97	1.33	0.05
		0702_00006	8	ARCH	2.07	1.45	1.47	0.05
Unknown	5650	0703_00565	3.2	RECT	1.84	5	NA	0.06
Unknown	5490	0703_00549	5.9	RECT	1.43	6	NA	0.06
Unknown	3230	0703_00323	1.6	RECT	0.76	1.68	NA	0.07
Unknown	3150	0703_00315	1.3	RECT	1.32	1.39	NA	0.07
			1.3	RECT	1.32	1.39	NA	0.07
Unknown	2300	0703_00230	4.6	ARCH	1.12	0.79	0.94	0.07
Unknown	1830	0703_00183	5.9	ARCH	1.58	1.3	1.03	0.05
Unknown	1680	0703_00168	1.6	RECT	1	2.6	NA	0.06
Unknown	390	0703_00039	7.4	ARCH	2.6	2.86	1.45	0.06
Newtown Stalaban	5350	0704_00535	7.6	ARCH	1.96	1.83	1.05	0.05
Newtown Stalaban	2750	0704_00275	15.9	ARCH	1.07	1.62	0.75	0.05
Newtown Stalaban	2320	0704_00232	8.7	ARCH	2.35	2.15	1.41	0.05
			8.7	ARCH	2.35	2.15	1.41	0.05
Beaulieu	1050	0705_00105	5.1	RECT	1.34	1.7	NA	0.05
Dry Bridge	2250	0706_00225	6.2	RECT	1.59	2.84	NA	0.05

Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
Dry Bridge	1680	0706_00168	13.5	ARCH	3.21	4.52	1.83	0.05
Dry Bridge	830	0706_00083	9.1	ARCH	1.46	2.65	NA	0.055
Unknown	70	0708_00007	3.7	ARCH	0.91	1.45	0.77	0.05
Unknown	1430	0708_00143	424.6	CIRC	1200	1200	NA	1.5
Unknown	900	0708_00090	16.4	CIRC	700	700	NA	1.5
Unknown	840	0708_00084	32.8	RECT	640	860	NA	1.5
Unknown	810	0708_00081	36.7	CIRC	600	600	NA	1.5
Unknown	440	0708_00044	24.9	CIRC	1200	1200	NA	1.5
Unknown	410	0708_MH4004	249.8	CIRC	1200	1200	NA	1.5
Dry Bridge	2720	0706_00272	3.3	CIRC	750	750	NA	1.5
Dry Bridge	2720	0706_00272	3.3	CIRC	750	750	NA	1.5
Dry Bridge	1910	0706_00191	5.3	CIRC	750	750	NA	1.5
Dry Bridge	1420	0706_00142	114	RECT	2960	3370	NA	1.5
Dry Bridge	1730	0707_00173	5.3	CIRC	600	600	NA	3
Dry Bridge	1570	0707_00157	96	CIRC	200	200	NA	1.5
Dry Bridge	1570	0707_00157	96	CIRC	300	300	NA	1.5
Dry Bridge	1570	0707_00157	95.9	CIRC	300	300	NA	1.5
Dry Bridge	1570	0707_00157	8.3	CIRC	600	600	NA	1.5
Dry Bridge	1310	0707_00131	34.5	CIRC	900	900	NA	1.5
Dry Bridge	1160	0707_00116	153.3	CIRC	1000	1000	NA	1.5

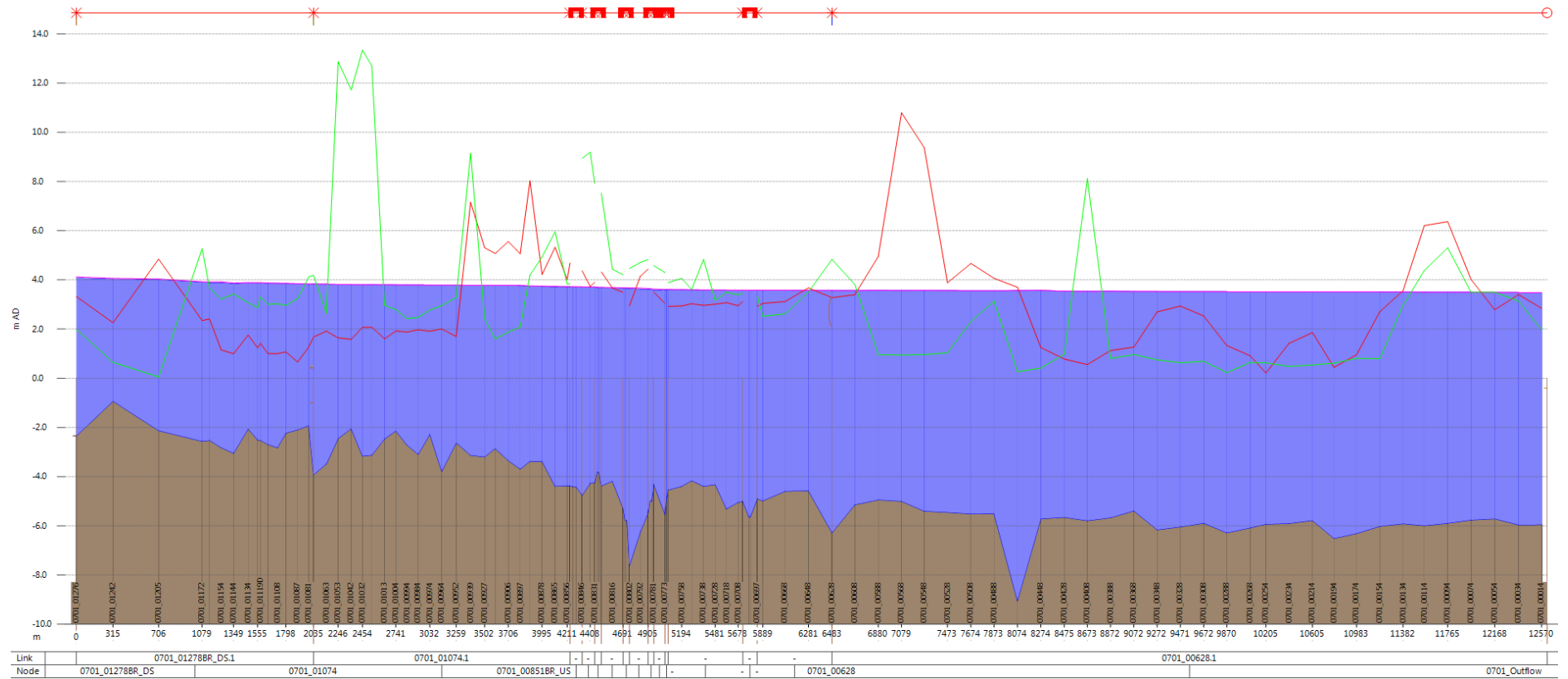
Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
Dry Bridge	670	0707_00067	206	CIRC	600	600	NA	1.5
Dry Bridge	320	0707_00032	145.4	CIRC	200	200	NA	1.5
Dry Bridge	320	0707_00032	144.7	CIRC	200	200	NA	1.5
Dry Bridge	320	0707_00032	10.4	CIRC	600	600	NA	1.5
Dry Bridge	80	0707_00008	19	RECT	600	900	NA	1.5
Newtown Stalaban	6760	0704_00676	10.5	RECT	1210	1260	NA	6
Newtown Stalaban	5330	0704_00533	6.3	CIRC	1000	1000	NA	1.5
Newtown Stalaban	4470	0704_00447	8.4	RECT	910	1190	NA	6
Newtown Stalaban	4470	0704_00447	8.3	RECT	910	1190	NA	6
Newtown Stalaban	4160	0704_00416	26.4	RECT	1470	1620	NA	1.5
Newtown Stalaban	3740	0704_00374	4.7	CIRC	1200	1200	NA	1.5
Newtown Stalaban	3170	0704_00317	4.6	RECT	1100	1110	NA	1.5
Newtown Stalaban	2150	0704_00215	8	RECT	2540	1000	NA	6
Newtown Stalaban	430	0704_00043	9.9	RECT	2930	2340	NA	1.5
Unknown	8400	0703_00840	131.8	CIRC	1160	1160	NA	1.5
Unknown	8090	0703_00809	177.5	CIRC	350	350	NA	1.5
Unknown	7890	0703_00789	29.9	CIRC	1200	1200	NA	1.5
Unknown	7760	0703_00776	3.9	CIRC	1200	1200	NA	1.5
Unknown	7590	0703_00759	4.7	CIRC	600	600	NA	1.5
Unknown	6870	0703_00687	4.1	CIRC	600	600	NA	1.5

Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
Unknown	6520	0703_00652	3.8	CIRC	900	900	NA	1.5
Unknown	6370	0703_00637	5.2	CIRC	900	900	NA	1.5
Unknown	6030	0703_00603	10.3	CIRC	900	900	NA	1.5
Unknown	3770	0703_00377	208.3	CIRC	900	900	NA	1.5
Unknown	3310	0703_00331	2	ARCHSPR UNG	1050	960	NA	1.5
Unknown	3280	0703_00328	2.2	RECT	1660	1060	NA	1.5
Unknown	3220	0703_00322	3.1	CIRC	225	225	NA	1.5
Unknown	3160	0703_00316	1.4	RECT	940	1000	NA	1.5
Unknown	3120	0703_00312	141.8	ARCHSPR UNG	1110	1550	NA	1.5
Unknown	2970	0703_00297	6.1	RECT	1200	1450	NA	1.5
Unknown	2850	0703_00285	4.3	CIRC	1050	1050	NA	1.5
Unknown	2620	0703_00262	4.8	ARCHSPR UNG	1050	950	NA	30
Unknown	2180	0703_00218	8.4	CIRC	1050	1050	NA	1.5
Unknown	2180	0703_00218	8.6	CIRC	1050	1050	NA	1.5
Unknown	1720	0703_00172	1.4	ARCH	1340	680	NA	1.5
Unknown	1660	0703_00166	1.1	RECT	580	1000	NA	6
Unknown	390	0703_00039	1.4	RECT	2860	1200	NA	6
Beaulieu	4230	0705_8913	16	CIRC	1350	1350	NA	1.5
Beaulieu	4235	0705_8913	15.8	CIRC	1350	1350	NA	1.5
Beaulieu	4220	0705_MH8908	21.4	RECT	2510	910	NA	1.5
Beaulieu	2395	0705_MH8907	25.2	CIRC	1350	1350	NA	1.5
Beaulieu	2395	0705_MH8907	25.5	CIRC	1350	1350	NA	1.5
Beaulieu	2370	0705_MH8810	49.6	CIRC	1350	1350	NA	1.5
Beaulieu	2370	0705_MH8813	50.2	CIRC	1350	1350	NA	1.5
Beaulieu	2320	0705_MH8814	52.9	CIRC	1350	1350	NA	1.5
Beaulieu	2320	0705_MH8815	49.6	CIRC	1350	1350	NA	1.5
Beaulieu	2270	0705_MH7808	19.1	CIRC	1350	1350	NA	1.5
Beaulieu	2270	0705_MH7809	22.8	CIRC	1350	1350	NA	1.5
Beaulieu	2250	0705_MH7712	32.9	CIRC	1350	1350	NA	1.5
Beaulieu	2250	0705_MH7711	29.6	CIRC	1350	1350	NA	1.5
Beaulieu	2215	0705_MH8706	71.7	CIRC	900	900	NA	1.5

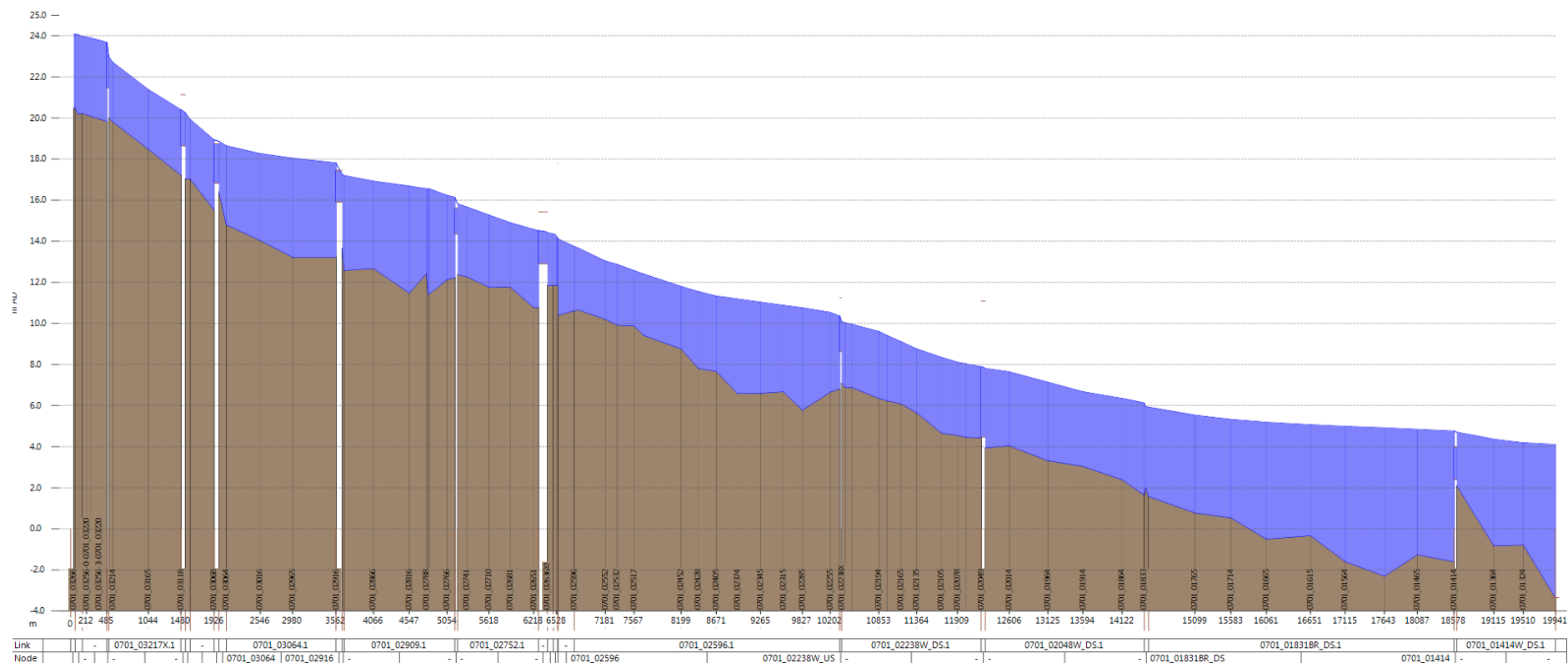
Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
Beaulieu	2215	0705_MH8706	71.8	CIRC	900	900	NA	1.5
Beaulieu	2145	0705_MH8602	164	CIRC	1200	1200	NA	1.5
Beaulieu	2145	0705_MH8602	163.4	CIRC	1200	1200	NA	1.5
Beaulieu	1980	0705_MH9688	147.4	CIRC	1200	1200	NA	1.5
Beaulieu	1980	0705_MH9688	148.7	CIRC	1200	1200	NA	1.5
Beaulieu	1830	0705_MH9499	81.4	CIRC	1200	1200	NA	1.5
Beaulieu	1830	0705_MH9499	82.5	CIRC	1200	1200	NA	1.5
Beaulieu	1750	0705_MH0464	67	CIRC	1200	1200	NA	1.5
Beaulieu	1750	0705_MH0464	68.6	CIRC	1200	1200	NA	1.5
Beaulieu	1685	0705_MH0333	76.4	CIRC	1200	1200	NA	1.5
Beaulieu	1685	0705_MH0333	76.8	CIRC	1200	1200	NA	1.5
Beaulieu	1610	0705_MH1303	28.2	CIRC	900	900	NA	1.5
Beaulieu	1610	0705_MH1303	28.5	CIRC	900	900	NA	1.5
Beaulieu	1610	0705_MH1303	28.7	CIRC	900	900	NA	1.5
Beaulieu	1580	0705_MH1302	128.1	CIRC	1200	1200	NA	1.5
Beaulieu	1580	0705_MH1302	128.9	CIRC	1200	1200	NA	1.5
Beaulieu	1380	0705_INLET3305	58.9	CIRC	1200	1200	NA	1.5
Beaulieu	1380	0705_INLET3305	59.2	CIRC	1200	1200	NA	1.5
Beaulieu	1320	0705_MH4304	55.7	CIRC	1200	1200	NA	1.5
Beaulieu	1320	0705_MH4304	56.5	CIRC	1200	1200	NA	1.5
Beaulieu	1265	0705_MH4307	16.9	CIRC	1200	1200	NA	1.5
Beaulieu	1265	0705_MH4307	17.1	CIRC	1200	1200	NA	1.5
Beaulieu	1245	0705_MH4305	26.6	CIRC	1200	1200	NA	1.5
Beaulieu	1245	0705_MH4305	26.8	CIRC	1200	1200	NA	1.5
Beaulieu	70	0705_MH2607	25.8	RECT	1190	950	NA	1.5
Beaulieu	45	0705_MH2621	6.9	RECT	1190	950	NA	1.5
Beaulieu	38	0705_MH2622	19.4	RECT	1190	950	NA	1.5
Beaulieu	18	0705_MH3603	182.1	RECT	2350	1250	NA	1.5
Beaulieu	150	0705_00015	79.1	RECT	1190	950	NA	1.5
Beaulieu	5120	0705_00512	3.8	RECT	663	1320	NA	1.5
Beaulieu	5110	0705_00511	7.8	CIRC	750	750	NA	1.5
Beaulieu	3930	0705_00393	3.1	CIRC	450	450	NA	1.5
Beaulieu	3930	0705_00393	3.1	CIRC	450	450	NA	1.5

Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING' S n/ Colebrook White
Beaulieu	3400	0705_00340	168.7	CIRC	1200	1200	NA	1.5
Beaulieu	3010	0705_00301	7.5	CIRC	500	500	NA	1.5
Beaulieu	1218	0705_MH5203	11.4	CIRC	1000	1000	NA	1.5
Beaulieu	1218	0705_MH5203	11.3	CIRC	1000	1000	NA	1.5
Beaulieu	1210	0705_0121	19.7	ARCHSPR UNG	1200	1340	NA	1.5
Beaulieu	830	0705_00083	28	CIRC	1300	1300	NA	1.5
Beaulieu	830	0705_00083	28.1	CIRC	1300	1300	NA	1.5
Beaulieu	440	0705_00044	126.5	ARCH	2620	1570	NA	15
Beaulieu	1460	0705_MH2206	8.8	CIRC	900	900	NA	1.5
Beaulieu	1460	0705_MH2206	8.2	CIRC	900	900	NA	1.5
Beaulieu	1460	0705_MH2206	8.7	CIRC	900	900	NA	1.5

River Long Section Profile

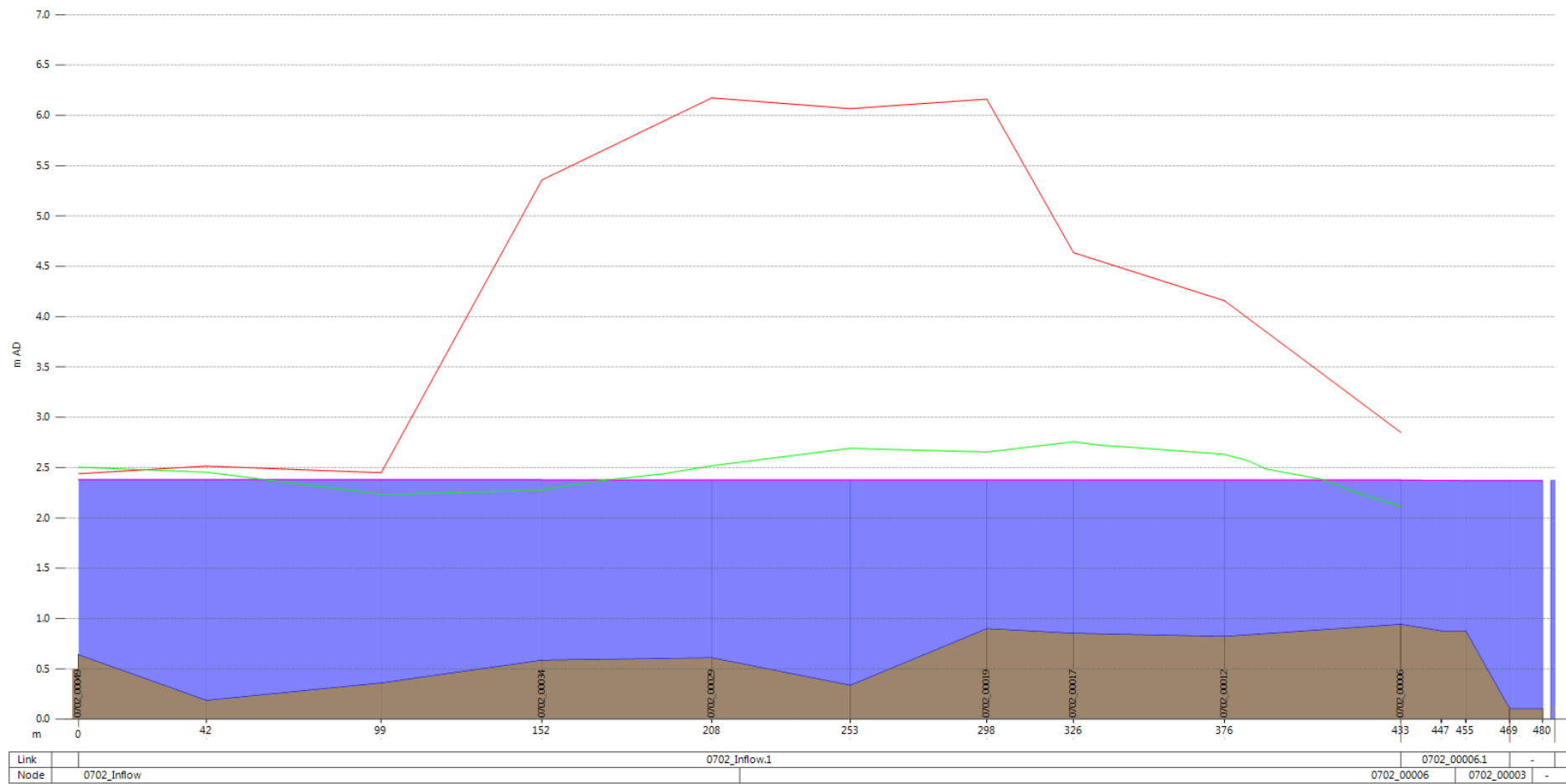


Boyne HPW 1% AEP



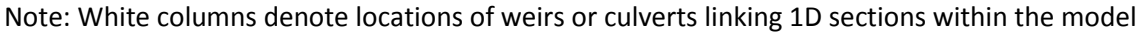
Note: White columns denote locations of weirs linking 1D sections within the model

Boyne MPW 1% AEP

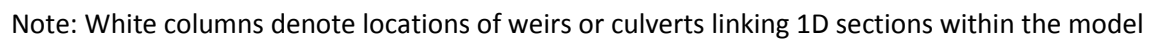


Note: White columns denote locations of weirs or culverts linking 1D sections within the model

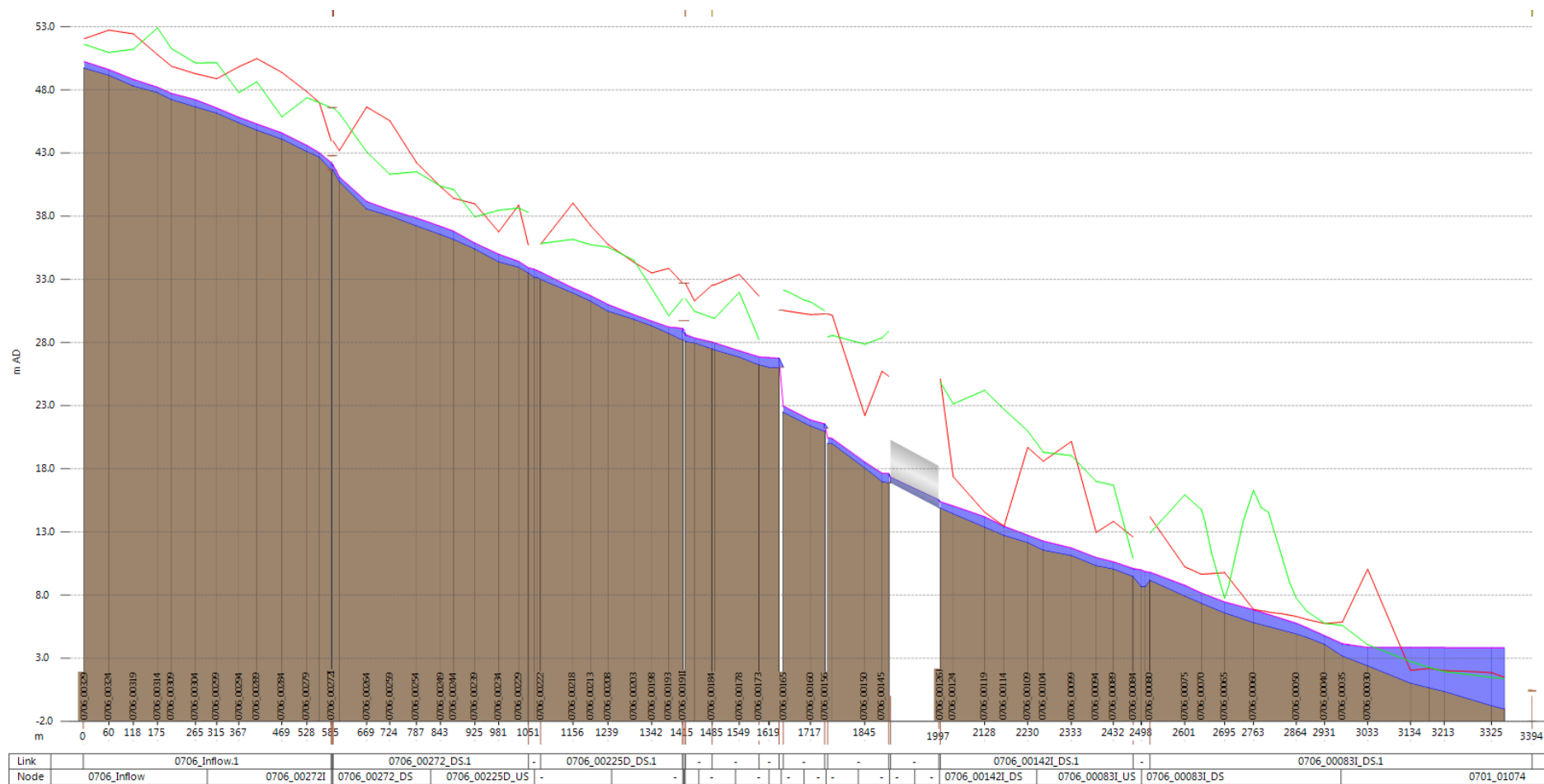
0702 - Baltray River 1% AEP



C

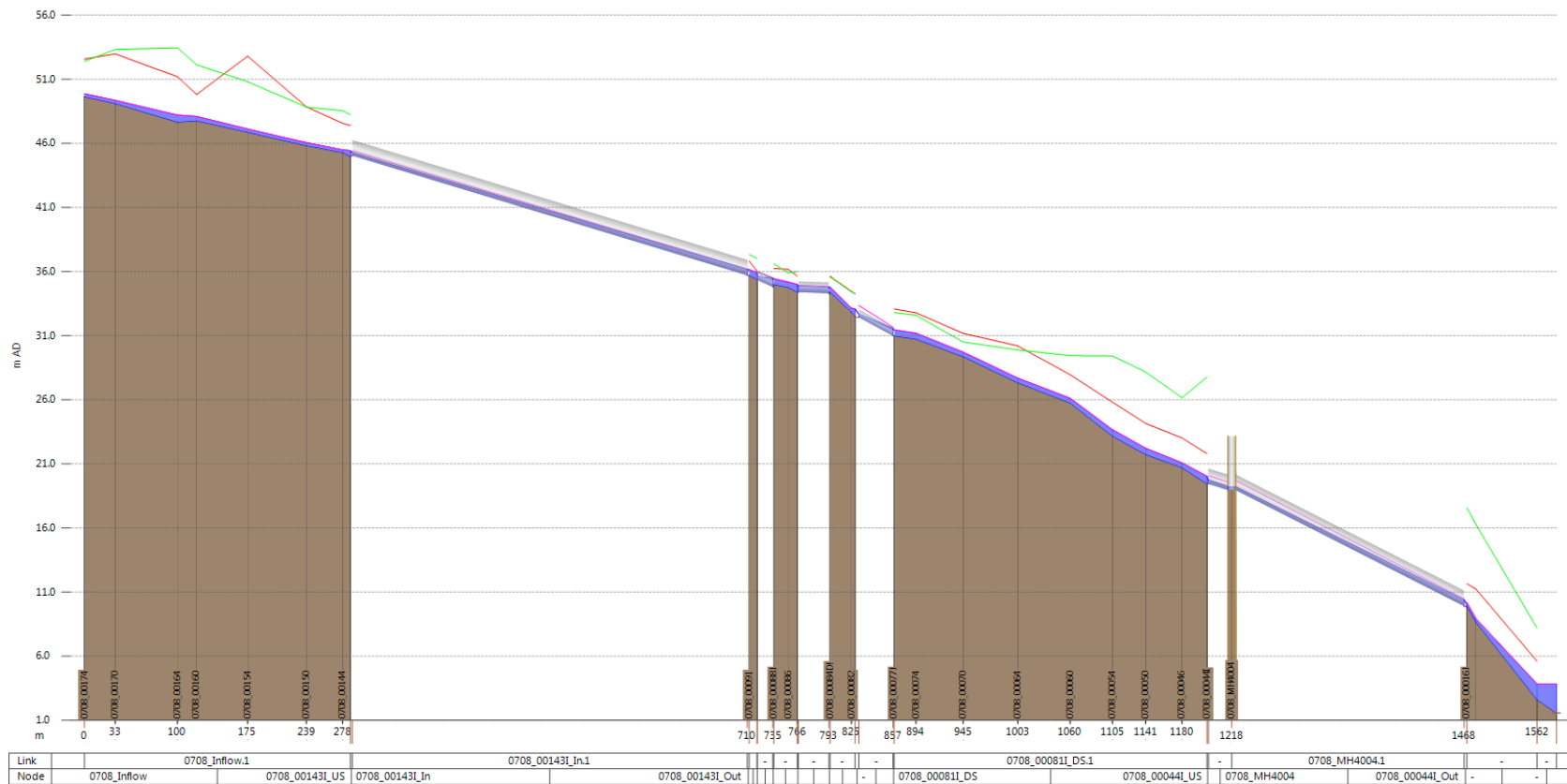


C



Note: White columns denote locations of weirs or culverts linking 1D sections within the model

0706 - Dry Bridge Stream 1% AEP



Note: White columns denote locations of weirs or culverts linking 1D sections within the model

0708 - Unknown 1% AEP

Fluvial and Mechanism 1 Tidal Model Files

ICM Transportable Database containing all relevant model files linked by simulation file for each required run – HA07_DROG07_F02.ICMT

GIS Deliverables – Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<div><div><div>Fluvial</div><div>E16EXFCD500F0</div><div>E16EXFCD200F0</div><div>E16EXFCD100F0</div><div>E16EXFCD050F0</div><div>E16EXFCD020F0</div><div>E16EXFCD010F0</div><div>E16EXFCD005F0</div><div>E16EXFCD001F0</div></div><div><div>Water Level and Flows</div><div>Fluvial</div><div>E16NFCDF0</div><div>E16NFMDF0</div><div>E16NFHDF0</div></div><div><div>Flood Defences (Shapefiles)</div><div>N/A</div></div><div><div>Defended Areas (Shapefiles)</div><div>N/A</div></div><div><div>E16EXFMD500F0</div><div>E16EXFMD200F0</div><div>E16EXFMD100F0</div><div>E16EXFMD050F0</div><div>E16EXFMD020F0</div><div>E16EXFMD010F0</div><div>E16EXFMD005F0</div><div>E16EXFMD001F0</div></div><div><div>E16EXFHD100F0</div><div>E16EXFHD010F0</div><div>E16EXFHD001F0</div></div></div> <td><div><div>Fluvial</div><div>E16DPFCD500F0</div><div>E16DPFCD200F0</div><div>E16DPFCD100F0</div><div>E16DPFCD050F0</div><div>E16DPFCD020F0</div><div>E16DPFCD010F0</div><div>E16DPFCD005F0</div><div>E16DPFCD001F0</div></div><div><div>E16DPFMD500F0</div><div>E16DPFMD200F0</div><div>E16DPFMD100F0</div><div>E16DPFMD050F0</div><div>E16DPFMD020F0</div><div>E16DPFMD010F0</div><div>E16DPFMD005F0</div><div>E16DPFMD001F0</div></div><div><div>E16DPFHD100F0</div><div>E16DPFHD010F0</div><div>E16DPFHD001F0</div></div><td><div><div>Fluvial</div><div>E16VLFCF500F0</div><div>E16VLFCF200F0</div><div>E16VLFCF100F0</div><div>E16VLFCF050F0</div><div>E16VLFCF020F0</div><div>E16VLFCF010F0</div><div>E16VLFCF005F0</div><div>E16VLFCF001F0</div></div><td><div><div>Fluvial</div><div>E16RLFCD100F0</div><div>E16RLFCD010F0</div><div>E16RLFCD001F0</div></div><div><div>Flood Zones (Shapefiles)</div><div>E16ZNA_MCDF0</div><div>E16ZNB_MCDF0</div><div>E16ZNA_MMDF0</div><div>E16ZNB_MMDF0</div></div></td></td></td>	<div><div>Fluvial</div><div>E16DPFCD500F0</div><div>E16DPFCD200F0</div><div>E16DPFCD100F0</div><div>E16DPFCD050F0</div><div>E16DPFCD020F0</div><div>E16DPFCD010F0</div><div>E16DPFCD005F0</div><div>E16DPFCD001F0</div></div> <div><div>E16DPFMD500F0</div><div>E16DPFMD200F0</div><div>E16DPFMD100F0</div><div>E16DPFMD050F0</div><div>E16DPFMD020F0</div><div>E16DPFMD010F0</div><div>E16DPFMD005F0</div><div>E16DPFMD001F0</div></div> <div><div>E16DPFHD100F0</div><div>E16DPFHD010F0</div><div>E16DPFHD001F0</div></div> <td><div><div>Fluvial</div><div>E16VLFCF500F0</div><div>E16VLFCF200F0</div><div>E16VLFCF100F0</div><div>E16VLFCF050F0</div><div>E16VLFCF020F0</div><div>E16VLFCF010F0</div><div>E16VLFCF005F0</div><div>E16VLFCF001F0</div></div><td><div><div>Fluvial</div><div>E16RLFCD100F0</div><div>E16RLFCD010F0</div><div>E16RLFCD001F0</div></div><div><div>Flood Zones (Shapefiles)</div><div>E16ZNA_MCDF0</div><div>E16ZNB_MCDF0</div><div>E16ZNA_MMDF0</div><div>E16ZNB_MMDF0</div></div></td></td>	<div><div>Fluvial</div><div>E16VLFCF500F0</div><div>E16VLFCF200F0</div><div>E16VLFCF100F0</div><div>E16VLFCF050F0</div><div>E16VLFCF020F0</div><div>E16VLFCF010F0</div><div>E16VLFCF005F0</div><div>E16VLFCF001F0</div></div> <td><div><div>Fluvial</div><div>E16RLFCD100F0</div><div>E16RLFCD010F0</div><div>E16RLFCD001F0</div></div><div><div>Flood Zones (Shapefiles)</div><div>E16ZNA_MCDF0</div><div>E16ZNB_MCDF0</div><div>E16ZNA_MMDF0</div><div>E16ZNB_MMDF0</div></div></td>	<div><div>Fluvial</div><div>E16RLFCD100F0</div><div>E16RLFCD010F0</div><div>E16RLFCD001F0</div></div> <div><div>Flood Zones (Shapefiles)</div><div>E16ZNA_MCDF0</div><div>E16ZNB_MCDF0</div><div>E16ZNA_MMDF0</div><div>E16ZNB_MMDF0</div></div>	
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)
<div><div><div>Coastal</div><div>E16EXCCD500F0</div><div>E16EXCCD200F0</div><div>E16EXCCD100F0</div><div>E16EXCCD050F0</div><div>E16EXCCD020F0</div><div>E16EXCCD010F0</div><div>E16EXCCD005F0</div><div>E16EXCCD001F0</div></div><div><div>Water Level and Flows</div><div>Coastal</div><div>E16NCCDF0</div><div>E16NCMDF0</div><div>E16NCHDF0</div></div><div><div>Flood Defences (Shapefiles)</div><div>Refer to DAD</div></div><div><div>Defended Areas (Shapefiles)</div><div>E16DFCCD010F0</div></div><div><div>Wave Overtopping (Shapefiles)</div><div>N/A</div></div><div><div>E16EXCMD500F0</div><div>E16EXCMD200F0</div><div>E16EXCMD100F0</div><div>E16EXCMD050F0</div><div>E16EXCMD020F0</div><div>E16EXCMD010F0</div><div>E16EXCMD005F0</div><div>E16EXCMD001F0</div></div><div><div>E16EXCHD100F0</div><div>E16EXCHD010F0</div><div>E16EXCHD001F0</div></div></div>	<div><div>Coastal</div><div>E16DPCCD500F0</div><div>E16DPCCD200F0</div><div>E16DPCCD100F0</div><div>E16DPCCD050F0</div><div>E16DPCCD020F0</div><div>E16DPCCD010F0</div><div>E16DPCCD005F0</div><div>E16DPCCD001F0</div></div> <div><div>E16DPCMD500F0</div><div>E16DPCMD200F0</div><div>E16DPCMD100F0</div><div>E16DPCMD050F0</div><div>E16DPCMD020F0</div><div>E16DPCMD010F0</div><div>E16DPCMD005F0</div><div>E16DPCMD001F0</div></div> <div><div>E16DPCHD100F0</div><div>E16DPCHD010F0</div><div>E16DPCHD001F0</div></div>	<div><div>Coastal</div><div>E16VLCCD500F0</div><div>E16VLCCD200F0</div><div>E16VLCCD100F0</div><div>E16VLCCD050F0</div><div>E16VLCCD020F0</div><div>E16VLCCD010F0</div><div>E16VLCCD005F0</div><div>E16VLCCD001F0</div></div> <div><div>Risk to Life Function (Raster)</div><div>Coastal</div><div>E16RLCCD001F0</div><div>E16RLCCD005F0</div><div>E16RLCCD100F0</div></div>	
Defence Failure Scenario			
Extent	Depth	Velocity	
N/A	N/A	N/A	

GIS Deliverables - Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E16RIFCD100F0 E16RIFCD010F0 E16RIFCD001F0 E16RIFMD100F0 E16RIFMD010F0 E16RIFMD001F0	<u>Fluvial</u> E07_RTFCDD001_F0 E07_RTFCMD001_F0	<u>Fluvial</u> E16RDFCD001F0 E16RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Coastal</u> E16RICCD100F0 E16RICCD005F0 E16RICCD001F0 E16RICMD100F0 E16RICMD005F0 E16RICMD001F0	<u>Coastal</u> E07_RTCCDD001_F0 E07_RTCMD001_F0	<u>Coastal</u> E16RDCCD001F0 E16RDCMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix D

Edenderry AFA Additional Information

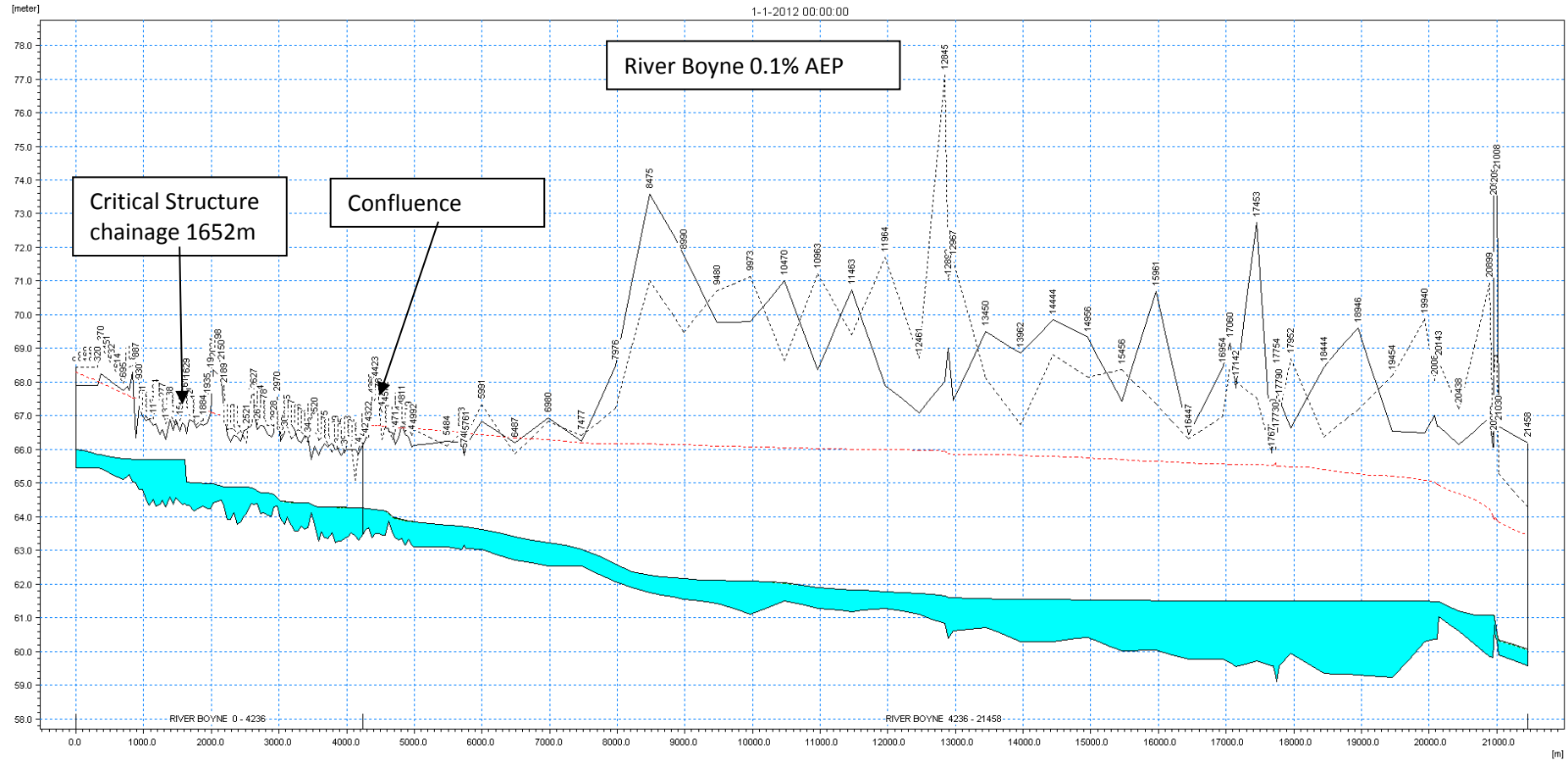
List of background information included:

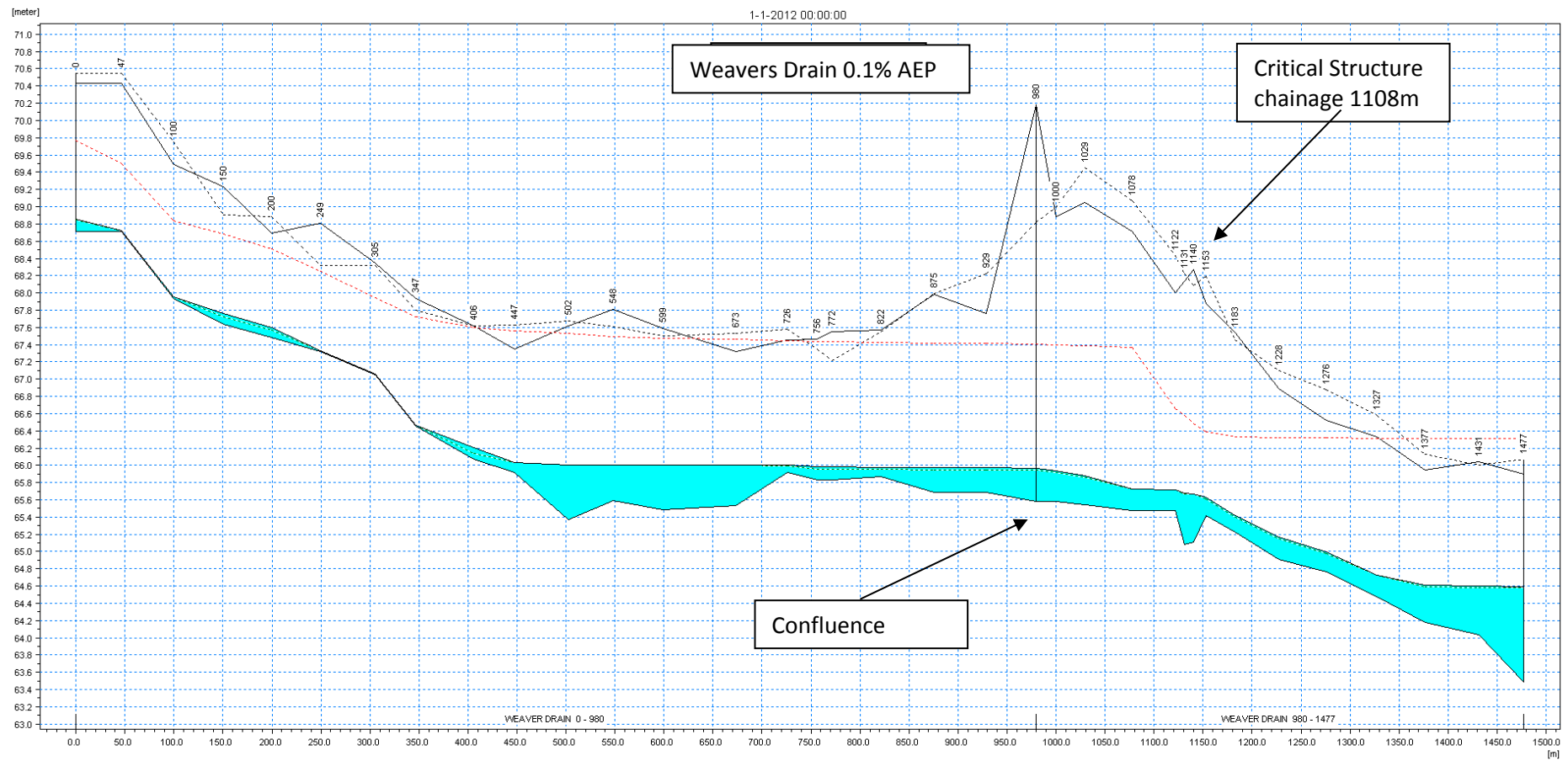
- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

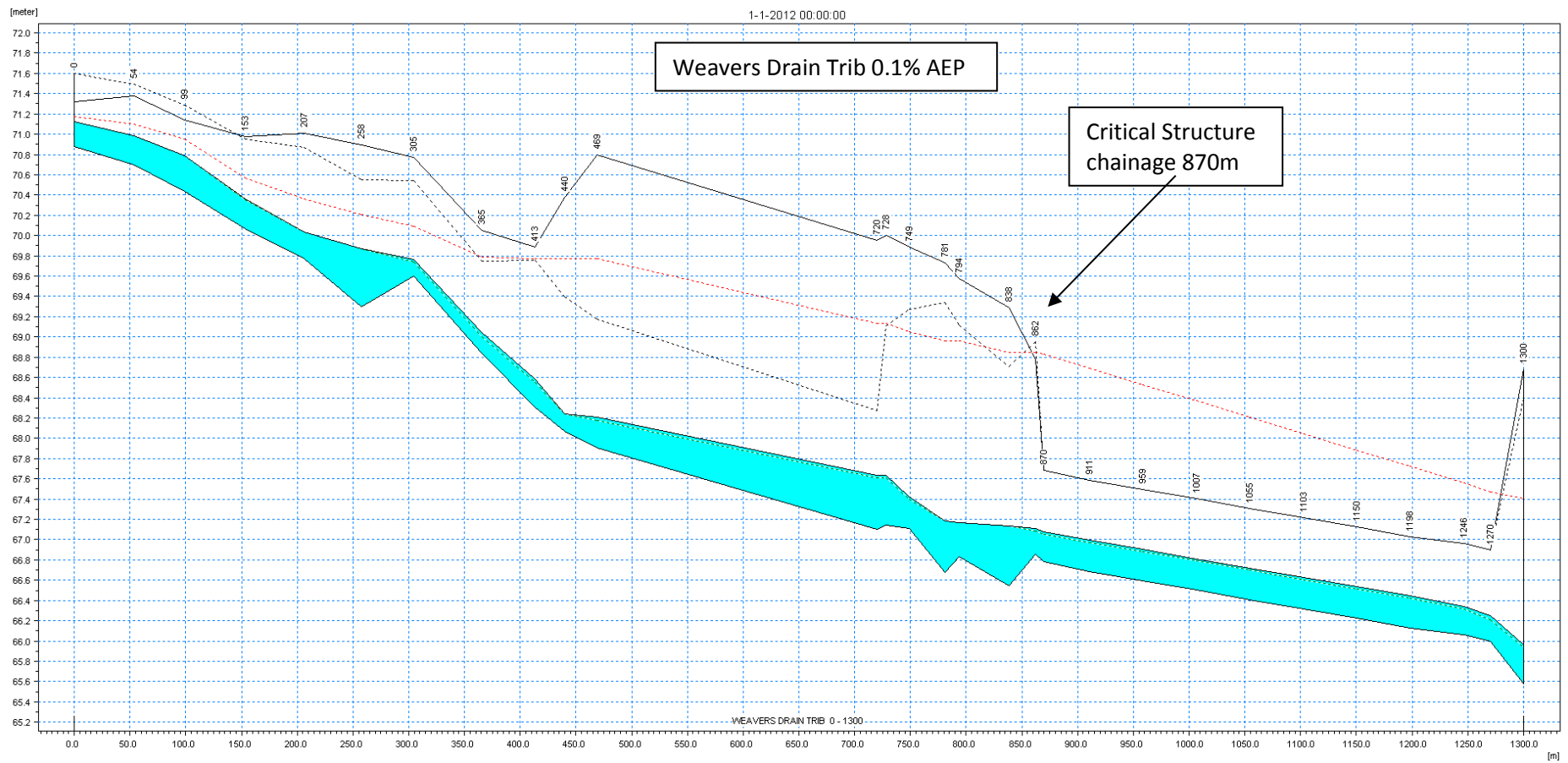
1D Structures

	Branch	Chainage	ID	Length (m)	Opening Shape	Height (m)	Width (m)	Spring Height from invert (m)	Mannings n
1	RIVER BOYNE	346.16	0701_10416D	3.58	Cross Section DB	2.72	4.42	-	0.02
2	RIVER BOYNE	728.57	0701_10379D	5.92	Cross Section DB	2.57	3.73	-	0.02
3	RIVER BOYNE	1652	0701_10288I_midpoint	34	Cross Section DB	3.46	6.1	1.35	0.02
4	RIVER BOYNE	3143.24	0701_10135D_copy	7.93	Cross Section DB	4.65	4.63	3.77	0.02
5	RIVER BOYNE	5717.57	0701_09877D_copy	7.66	Cross Section DB	4.97	5.24	3.83	0.02
6	RIVER BOYNE	12861.49	0701_09156D	7.54	Cross Section DB	5.95	9.07	4.28	0.02
7	RIVER BOYNE	17079.81	0701_08733D_copy	34.22	Cross Section DB	6.54	28.76	-	0.02
8	RIVER BOYNE	17716.71	0701_08669D_copy	10.15	Cross Section DB	7.5	27.72	5.3	0.02
9	RIVER BOYNE	20100.67	0701_08431D_copy	6.39	Cross Section DB	5.84	28.46	3.7	0.02
10	RIVER BOYNE	20984.42	0701_08343D_copy	12.27	Cross Section DB	9.91	35.98	6.53	0.02
11	WEAVER DRAIN	1108.11	0738_00039I_copy	6.02	Circular	0.9	-	-	0.013
12	WEAVERS DRAIN TRIB	870	0739_00031I	400	Circular	0.9	-	-	0.022
13	WEAVERS DRAIN TRIB	796.06	0739_00039I_copy	3.96	Circular	1	-	-	0.022
14	WEAVERS DRAIN TRIB	751.06	0739_00042I_copy	3.86	Circular	1	-	-	0.022
15	WEAVERS DRAIN TRIB	740.15	0739_00043I_copy	4.23	Circular	1	-	-	0.022
16	WEAVERS DRAIN TRIB	590	0739_00071I	250	Circular	1	-	-	0.022

River Long Section Profile







Final Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_EDEN1_MF_DES_Q2	HA07_EDEN1_M21_DES_Q2		N/A
HA07_EDEN1_MF_DES_Q5	HA07_EDEN1_M21_DES_Q5		
HA07_EDEN1_MF_DES_Q10	HA07_EDEN1_M21_DES_Q10		
HA07_EDEN1_MF_DES_Q20	HA07_EDEN1_M21_DES_Q20_1		
HA07_EDEN1_MF_DES_Q50	HA07_EDEN1_M21_DES_Q50		
HA07_EDEN1_MF_DES_Q100	HA07_EDEN1_M21_DES_Q100		
HA07_EDEN1_MF_DES_Q200	HA07_EDEN1_M21_DES_Q200		
HA07_EDEN1_MF_DES_Q1000_1	HA07_EDEN1_M21_DES_Q1000_1		
HA07_EDEN1_MF_DES_Q2_MRFS	HA07_EDEN1_M21_DES_Q2_MRFS		
HA07_EDEN1_MF_DES_Q5_MRFS	HA07_EDEN1_M21_DES_Q5_MRFS		
HA07_EDEN1_MF_DES_Q10_MRFS	HA07_EDEN1_M21_DES_Q10_MRFS		
HA07_EDEN1_MF_DES_Q20_MRFS_1	HA07_EDEN1_M21_DES_Q20_MRFS_1		
HA07_EDEN1_MF_DES_Q50_MRFS	HA07_EDEN1_M21_DES_Q50_MRFS		
HA07_EDEN1_MF_DES_Q100_MRFS	HA07_EDEN1_M21_DES_Q100_MRFS		
HA07_EDEN1_MF_DES_Q200_MRFS	HA07_EDEN1_M21_DES_Q200_MRFS		
HA07_EDEN1_MF_DES_Q1000_3_MRFS	HA07_EDEN1_M21_DES_Q1000_1_MRFS		
HA07_EDEN1_MF_DES_Q10_HEFS	HA07_EDEN1_M21_DES_Q10_HEFS		
HA07_EDEN1_MF_DES_Q100_HEFS	HA07_EDEN1_M21_DES_Q100_HEFS		
HA07_EDEN1_MF_DES_Q1000_1_HEFS	HA07_EDEN1_M21_DES_Q1000_1_HEFS		
	HA07_EDEN_1_Rech_BLDGS_Blocked_WCs_included_Edit141016_1		
	ha07_edenderry_mannings_m_crop2_extent_1.tmp		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_EDEN1_M11_DES_Q2 HA07_EDEN1_M11_DES_Q5 HA07_EDEN1_M11_DES_Q10 HA07_EDEN1_M11_DES_Q20_1 HA07_EDEN1_M11_DES_Q50 HA07_EDEN1_M11_DES_Q100 HA07_EDEN1_M11_DES_Q200 HA07_EDEN1_M11_DES_Q1000_1 HA07_EDEN1_M11_DES_Q2_MRFS HA07_EDEN1_M11_DES_Q5_MRFS HA07_EDEN1_M11_DES_Q10_MRFS HA07_EDEN1_M11_DES_Q20_MRFS_1 HA07_EDEN1_M11_DES_Q50_MRFS HA07_EDEN1_M11_DES_Q100_MRFS HA07_EDEN1_M11_DES_Q200_MRFS HA07_EDEN1_M11_DES_Q1000_1_MRFS HA07_EDEN1_M11_DES_Q10_HEFS HA07_EDEN1_M11_DES_Q100_HEFS HA07_EDEN1_M11_DES_Q1000_1_HEFS	HA07_EDEN1_NWK_DES_04	HA07_EDEN1_XNS_DES_01	HA07_EDEN1_BND_DES_Q2 HA07_EDEN1_BND_DES_Q5 HA07_EDEN1_BND_DES_Q10 HA07_EDEN1_BND_DES_Q20 HA07_EDEN1_BND_DES_Q50 HA07_EDEN1_BND_DES_Q100 HA07_EDEN1_BND_DES_Q200 HA07_EDEN1_BND_DES_Q1000 HA07_EDEN1_BND_DES_Q2_MRFS HA07_EDEN1_BND_DES_Q5_MRFS HA07_EDEN1_BND_DES_Q10_MRFS HA07_EDEN1_BND_DES_Q20_MRFS HA07_EDEN1_BND_DES_Q50_MRFS HA07_EDEN1_BND_DES_Q100_MRFS HA07_EDEN1_BND_DES_Q200_MRFS HA07_EDEN1_BND_DES_Q1000_MRFS HA07_EDEN1_BND_DES_Q10_HEFS HA07_EDEN1_BND_DES_Q100_HEFS HA07_EDEN1_BND_DES_Q1000_HEFS

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_EDEN1_DFS0_Q2	HA07_EDEN1_HD_DES_Q2	HA07_EDEN1_M11_DES_Q2	HA07_EDEN1_HDMaps_DES_Q2
HA07_EDEN1_DFS0_Q5	HA07_EDEN1_HD_DES_Q5	HA07_EDEN1_M11_DES_Q5	HA07_EDEN1_HDMaps_DES_Q5
HA07_EDEN1_DFS0_Q10	HA07_EDEN1_HD_DES_Q10	HA07_EDEN1_M11_DES_Q10	HA07_EDEN1_HDMaps_DES_Q10
HA07_EDEN1_DFS0_Q20	HA07_EDEN1_HD_DES_Q20_1	HA07_EDEN1_M11_DES_Q20_1	HA07_EDEN1_HDMaps_DES_Q20_1
HA07_EDEN1_DFS0_Q50	HA07_EDEN1_HD_DES_Q50	HA07_EDEN1_M11_DES_Q50	HA07_EDEN1_HDMaps_DES_Q50
HA07_EDEN1_DFS0_Q100	HA07_EDEN1_HD_DES_Q100	HA07_EDEN1_M11_DES_Q100	HA07_EDEN1_HDMaps_DES_Q100
HA07_EDEN1_DFS0_Q200	HA07_EDEN1_HD_DES_Q200	HA07_EDEN1_M11_DES_Q200	HA07_EDEN1_HDMaps_DES_Q200
HA07_EDEN1_DFS0_Q1000	HA07_EDEN1_HD_DES_Q1000_1	HA07_EDEN1_M11_DES_Q1000_1	HA07_EDEN1_HDMaps_DES_Q1000_1
HA07_EDEN1_DFS0_Q2_MRFS	HA07_EDEN1_HD_DES_Q2_MRFS	HA07_EDEN1_M11_DES_Q2_MRFS	HA07_EDEN1_HDMaps_DES_Q2_MRFS
HA07_EDEN1_DFS0_Q5_MRFS	HA07_EDEN1_HD_DES_Q5_MRFS	HA07_EDEN1_M11_DES_Q5_MRFS	HA07_EDEN1_HDMaps_DES_Q5_MRFS
HA07_EDEN1_DFS0_Q10_MRFS	HA07_EDEN1_HD_DES_Q10_MRFS	HA07_EDEN1_M11_DES_Q10_MRFS	HA07_EDEN1_HDMaps_DES_Q10_MRFS
HA07_EDEN1_DFS0_Q20_MRFS	HA07_EDEN1_HD_DES_Q20_MRFS_1	HA07_EDEN1_M11_DES_Q20_MRFS_1	HA07_EDEN1_HDMaps_DES_Q20_MRFS_1
HA07_EDEN1_DFS0_Q50_MRFS	HA07_EDEN1_HD_DES_Q50_MRFS	HA07_EDEN1_M11_DES_Q50_MRFS	HA07_EDEN1_HDMaps_DES_Q50_MRFS
HA07_EDEN1_DFS0_Q100_MRFS	HA07_EDEN1_HD_DES_Q100_MRFS	HA07_EDEN1_M11_DES_Q100_MRFS	HA07_EDEN1_HDMaps_DES_Q100_MRFS
HA07_EDEN1_DFS0_Q200_MRFS	HA07_EDEN1_HD_DES_Q200_MRFS	HA07_EDEN1_M11_DES_Q200_MRFS	HA07_EDEN1_HDMaps_DES_Q200_MRFS
HA07_EDEN1_DFS0_Q1000_MRFS	HA07_EDEN1_HD_DES_Q1000_1_MRFS	HA07_EDEN1_M11_DES_Q1000_2_MRFS	HA07_EDEN1_HDMaps_DES_Q1000_2_MRFS
HA07_EDEN1_DFS0_Q10_HEFS	HA07_EDEN1_HD_DES_Q10_HEFS	HA07_EDEN1_M11_DES_Q10_HEFS	HA07_EDEN1_HDMaps_DES_Q10_HEFS
HA07_EDEN1_DFS0_Q100_HEFS	HA07_EDEN1_HD_DES_Q100_HEFS	HA07_EDEN1_M11_DES_Q100_HEFS	HA07_EDEN1_HDMaps_DES_Q100_HEFS
HA07_EDEN1_DFS0_Q1000_HEFS	HA07_EDEN1_HD_DES_Q1000_1_HEFS	HA07_EDEN1_M11_DES_Q1000_1_HEFS	HA07_EDEN1_HDMaps_DES_Q1000_1_HEFS
	ey_ndhm		

Final Model Files - Sensitivity

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_EDEN1_MF_SEN_1_Q100_flow HA07_EDEN1_MF_SEN_1_Q100_tt HA07_EDEN1_MF_SEN_Q100_HEFS_fv HA07_EDEN1_MF_SEN_Q100_HEFS_rough HA07_EDEN1_MF_SEN_Q100_hl_1	HA07_EDEN1_M21_SEN_1_Q100_flow HA07_EDEN1_M21_SEN_1_Q100_fv HA07_EDEN1_M21_SEN_1_Q100_hl_1 HA07_EDEN1_M21_SEN_1_Q100_rough HA07_EDEN1_M21_SEN_1_Q100_tt HA07_EDEN_1_Rech_BLDGS_Blocked_WCs_included_Edit141016_1 ha07_edenderry_mannings_m_crop2_extent_1.tmp ha07_edenderry_mannings_m_crop2_extent_1_rough		HA07_EDEN1_M21_SEN_1_Q100_flow HA07_EDEN1_M21_SEN_1_Q100_rough HA07_EDEN1_M21_SEN_1_Q100_tt HA07_EDEN1_M21_SEN_Q100_fv HA07_EDEN1_M21_SEN_Q100_hl_1

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_EDEN1_M11_SEN_1_Q100_flow HA07_EDEN1_M11_SEN_1_Q100_fv HA07_EDEN1_M11_SEN_1_Q100_hl_1 HA07_EDEN1_M11_SEN_1_Q100_rough HA07_EDEN1_M11_SEN_1_Q100_tt	HA07_EDEN1_NWK_DES_04 HA07_EDEN1_NWK_SEN_1_hl_1 HA07_EDEN1_NWK_SEN_1_Q100_flow	HA07_EDEN1_XNS_DES_01 HA07_EDEN1_XNS_SEN_1_hl_1 HA07_EDEN1_XNS_SEN_1_Q100_flow HA07_EDEN1_XNS_SEN_1_rough	HA07_EDEN1_BND_DES_Q100 HA07_EDEN1_BND_SEN_1_Q100_flow HA07_EDEN1_BND_SEN_1_Q100_fv HA07_EDEN1_BND_SEN_1_Q100_tt

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_EDEN1_DFS0_Q100 HA07_EDEN1_DFS0_SEN_Q100_flow HA07_EDEN1_DFS0_SEN_Q100_fv HA07_EDEN1_DFS0_SEN_Q100_tt	HA07_EDEN1_HD_SEN_1_Q100_flow HA07_EDEN1_HD_SEN_1_Q100_fv HA07_EDEN1_HD_SEN_1_Q100_hl_1 HA07_EDEN1_HD_SEN_1_Q100_rough HA07_EDEN1_HD_SEN_1_Q100_tt	HA07_EDEN1_M11_SEN_1_Q100_flow HA07_EDEN1_M11_SEN_1_Q100_rough HA07_EDEN1_M11_SEN_1_Q100_tt HA07_EDEN1_M11_SEN_Q100_fv HA07_EDEN1_M11_SEN_Q100_hl_1	N/A

GIS Deliverables – Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u>	Water Level and Flows	<u>Fluvial</u>	<u>Fluvial</u>	<u>Fluvial</u>
E17EXFCD500F0	<u>Fluvial</u>	E17DPFCD500F0	E17VLFCFCD500F0	E17RLFCFCD100F0
E17EXFCD200F0	E17NFCDF0	E17DPFCD200F0	E17VLFCFCD200F0	E17RLFCFCD010F0
E17EXFCD100F0	E17NFMDF0	E17DPFCD100F0	E17VLFCFCD100F0	E17RLFCFCD001F0
E17EXFCD050F0	E17NFHDF0	E17DPFCD050F0	E17VLFCFCD050F0	
E17EXFCD020F0		E17DPFCD020F0	E17VLFCFCD020F0	
E17EXFCD010F0	Flood Defences (Shapefiles)	E17DPFCD010F0	E17VLFCFCD010F0	
E17EXFCD005F0	N/A	E17DPFCD005F0	E17VLFCFCD005F0	
E17EXFCD001F0		E17DPFCD001F0	E17VLFCFCD001F0	
	Defended Areas (Shapefiles)			Flood Zones (Shapefiles)
E17EXFMD500F0	N/A	E17DPFMD500F0		E17ZNA_FCDF0
E17EXFMD200F0		E17DPFMD200F0		E17ZNB_FCDF0
E17EXFMD100F0	Wave Overtopping (Shapefiles)	E17DPFMD100F0		E17ZNA_FMDF0
E17EXFMD050F0	N/A	E17DPFMD050F0		E17ZNB_FMDF0
E17EXFMD020F0		E17DPFMD020F0		
E17EXFMD010F0		E17DPFMD010F0		
E17EXFMD005F0		E17DPFMD005F0		
E17EXFMD001F0		E17DPFMD001F0		
E17EXFHD100F0		E17DPFHD100F0		
E17EXFHD010F0		E17DPFHD010F0		
E17EXFHD001F0		E17DPFHD001F0		
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables – Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E17RIFCD100F0 E17RIFCD010F0 E17RIFCD001F0 E17RIFMD100F0 E17RIFMD010F0 E17RIFMD001F0	<u>Fluvial</u> E07_RTFCD001_F0 E07_RTFMD001_F0	<u>Fluvial</u> E17RDFCD001F0 E17RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix E

Johnstown Bridge AFA Additional Information

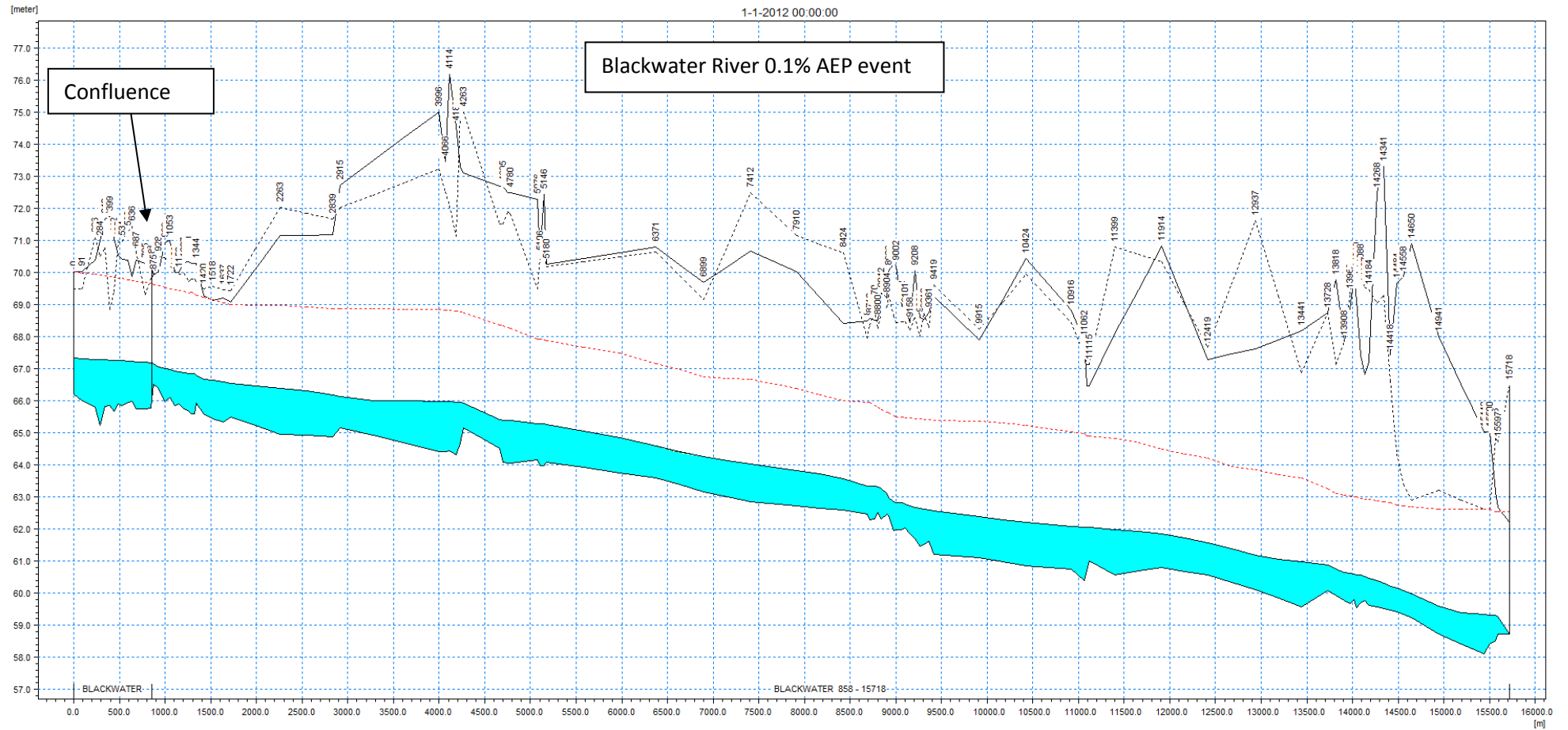
List of background information included:

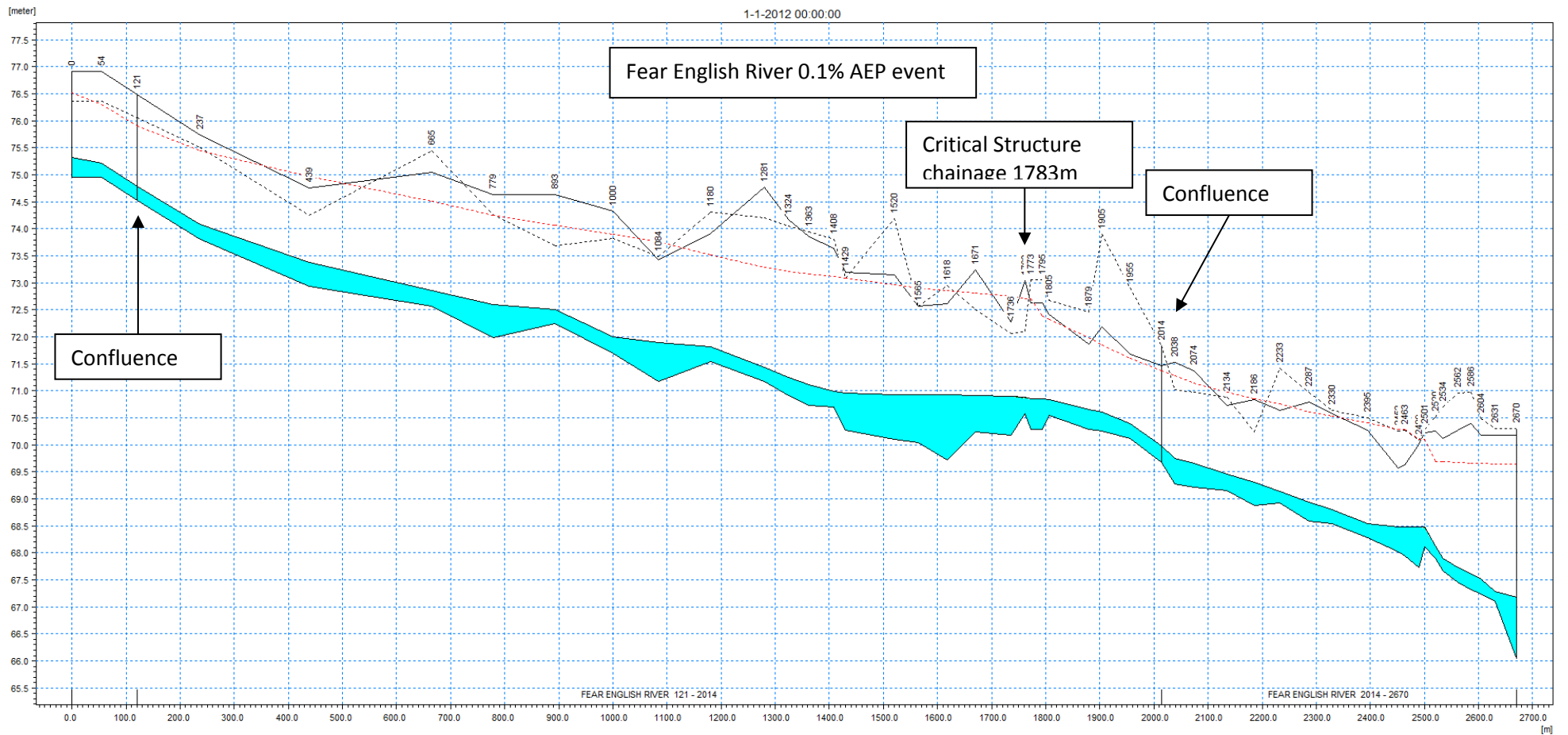
- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

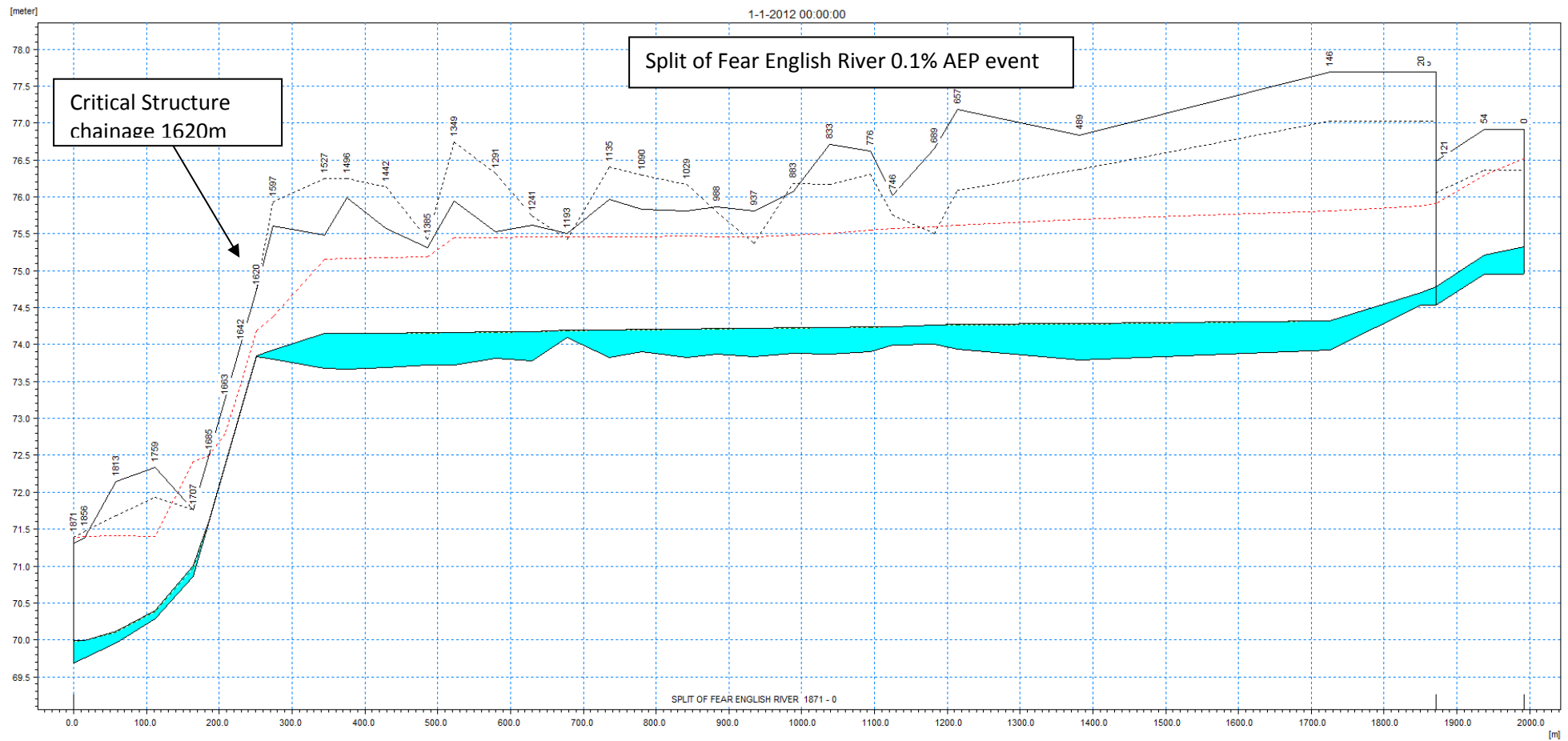
1D Structures

	Branch	Chainage	ID	Length (m)	Opening Shape	Height (m)	Width (m)	Spring Height from invert (m)	Mannings n
1	FEAR ENGLISH RIVER	2511.175	0737_00016I_copy	16.47	Cross Section DB	2.49	4.089	1.47	0.035
2	FEAR ENGLISH RIVER	2477.21	0737_00018	0.21	Cross Section DB	1.98	4.9	-	0.035
3	FEAR ENGLISH RIVER	2427.5	0737_00024Jeditmove	6.12	Cross Section DB	2.2	4.13	-	0.028
4	FEAR ENGLISH RIVER	1783.5	0737_00089D	8.78	Cross Section DB	1.91	3	0.9	0.035
5	SPLIT OF FEAR ENGLISH RIVER	1357.35	0740_00051I	4.05	Circular	0.8	-	-	0.013
6	SPLIT OF FEAR ENGLISH RIVER	1561.72	0740_00033I	47.49	Circular	0.9	-	-	0.013
7	SPLIT OF FEAR ENGLISH RIVER	1619.87	0740_00026I	87.24	Circular	0.9	-	-	0.011
8	BLACKWATER	1036.5	0735_01465E	12.1	Cross Section DB	3.4	7.65	-	0.035
9	BLACKWATER	1036.5	0735_01465Epipe	0.2	Irregular, Level-Width Table	1.25	6.911	-	0.035
10	BLACKWATER	1304.2	0735_01439D	3.64	Cross Section DB	2.8	7.3	-	0.035
11	BLACKWATER	1304.2	0735_01439Dpipe	0.2	Irregular, Level-Width Table	0.87	7.39	-	0.013
12	BLACKWATER	2875	0735_01297D	41.18	Cross Section DB	3.86	24.1	-	0.035
13	BLACKWATER	4019.09	0735_01170E	17	Cross Section DB	5.55	14.3	-	0.035
14	BLACKWATER	4213	0735_01151Dmove	11.28	Cross Section DB	6.27	8.2	4.32	0.035
15	BLACKWATER	4723.12	0735_01099D	2.9	Cross Section DB	4.44	7.28	-	0.035
16	BLACKWATER	5126	0735_01058E	14.43	Cross Section DB	5.49	19.9	-	0.035
17	BLACKWATER	8745.5	0735_00696D	3.23	Cross Section DB	4.31	8.9	-	0.035
18	BLACKWATER	9041.8	0735_00668D	7.67	Cross Section DB	5.2	11.26	2.56	0.035
19	BLACKWATER	11074.41	0735_00464D	8.67	Cross Section DB	5.23	10.8	2.65	0.035
20	BLACKWATER	14068.5	0735_00164E	8	Cross Section DB	4.88	9.47	2.92	0.013
21	BLACKWATER	15536.8	0735_00018D	7.32	Cross Section DB	4.36	11.24	2.2	0.035
22	BLACKWATER	15536.8	0735_00018D_subarch1	7.32	Irregular, Level-Width Table	0.95	1.57	0.56	0.035
23	BLACKWATER	15536.8	0735_00018D_subarch2	7.32	Irregular, Level-Width Table	1.45	2.53	0.98	0.035
24	BLACKWATER	15536.8	0735_00018D_subarch3	7.32	Irregular, Level-Width Table	1.55	3.19	1.26	0.035
25	BLACKWATER	15536.8	0735_00018D_subarch4	7.32	Irregular, Level-Width Table	1.33	2.48	0.96	0.035
26	BLACKWATER	14068.5	0735_00164E_subarch1	7.62	Irregular, Level-Width Table	1.33	3.11	0.84	0.035
27	BLACKWATER	14068.5	0735_00164E_subarch2	7.62	Irregular, Level-Width Table	1.47	2.82	0.92	0.035
28	BLACKWATER	14068.5	0735_00164E_subarch3	7.62	Irregular, Level-Width Table	1.56	2.86	1	0.035
29	BLACKWATER	14068.5	0735_00164E_subarch4	7.62	Irregular, Level-Width Table	1.35	2.23	0.89	0.035

River Long Section Profiles







Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_JOHN5_MF_DES_Q2_Final	HA07_JOHN5_M21_DES_Q2_Final		
HA07_JOHN5_MF_DES_Q5_Final	HA07_JOHN5_M21_DES_Q5_Final		
HA07_JOHN5_MF_DES_Q10_Final	HA07_JOHN5_M21_DES_Q10_Final		
HA07_JOHN5_MF_DES_Q20_Final	HA07_JOHN5_M21_DES_Q20_Final		
HA07_JOHN5_MF_DES_Q50_Final	HA07_JOHN5_M21_DES_Q50_Final		
HA07_JOHN5_MF_DES_Q100_Final	HA07_JOHN5_M21_DES_Q100_Final		
HA07_JOHN5_MF_DES_Q200_Final	HA07_JOHN5_M21_DES_Q200_Final		
HA07_JOHN5_MF_DES_Q1000_Final	HA07_JOHN5_M21_DES_Q1000_Final		
HA07_JOHN5_MF_DES_Q2_Final_MRFS	HA07_JOHN5_M21_DES_Q2_Final_MRFS		
HA07_JOHN5_MF_DES_Q5_Final_MRFS	HA07_JOHN5_M21_DES_Q5_Final_MRFS		
HA07_JOHN5_MF_DES_Q10_Final_MRFS	HA07_JOHN5_M21_DES_Q10_Final_MRFS		
HA07_JOHN5_MF_DES_Q20_Final_MRFS	HA07_JOHN5_M21_DES_Q20_Final_MRFS		
HA07_JOHN5_MF_DES_Q50_Final_MRFS	HA07_JOHN5_M21_DES_Q50_Final_MRFS		
HA07_JOHN5_MF_DES_Q100_Final_MRFS	HA07_JOHN5_M21_DES_Q100_Final_MRFS		
HA07_JOHN5_MF_DES_Q200_Final_MRFS	HA07_JOHN5_M21_DES_Q200_Final_MRFS		
HA07_JOHN5_MF_DES_Q1000_Final_MRFS	HA07_JOHN5_M21_DES_Q1000_Final_MRFS		
HA07_JOHN5_MF_DES_Q10_Final_HEFS	HA07_JOHN5_M21_DES_Q10_Final_HEFS		
HA07_JOHN5_MF_DES_Q100_Final_HEFS	HA07_JOHN5_M21_DES_Q100_Final_HEFS		
HA07_JOHN5_MF_DES_Q1000_Final_HEFS	HA07_JOHN5_M21_DES_Q1000_Final_HEFS		
	HA07_JOHN5_DFS_BATHY_1		
	HA07_JOHN5_DFS2_FPR		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_JOHN5_M11_DES_Q2_Final	J_Bridge_02	HA07_JOHN5_XNS_DES_3_Final_12	HA07_JOHN5_BND_DES_Q2_Final
HA07_JOHN5_M11_DES_Q5_Final	J_Bridge_02_Q1000H	HA07_JOHN5_XNS_DES_3_Final_12_Q1000H	HA07_JOHN5_BND_DES_Q5_Final
HA07_JOHN5_M11_DES_Q10_Final			HA07_JOHN5_BND_DES_Q10_Final
HA07_JOHN5_M11_DES_Q20_Final			HA07_JOHN5_BND_DES_Q20_Final
HA07_JOHN5_M11_DES_Q50_Final			HA07_JOHN5_BND_DES_Q50_Final
HA07_JOHN5_M11_DES_Q100_Final			HA07_JOHN5_BND_DES_Q100_Final
HA07_JOHN5_M11_DES_Q200_Final			HA07_JOHN5_BND_DES_Q200_Final
HA07_JOHN5_M11_DES_Q1000_Final			HA07_JOHN5_BND_DES_Q1000_Final
HA07_JOHN5_M11_DES_Q2_Final_MRFS			HA07_JOHN5_BND_DES_Q2_Final_MRFS
HA07_JOHN5_M11_DES_Q5_Final_MRFS			HA07_JOHN5_BND_DES_Q5_Final_MRFS
HA07_JOHN5_M11_DES_Q10_Final_MRFS			HA07_JOHN5_BND_DES_Q10_Final_MRFS
HA07_JOHN5_M11_DES_Q20_Final_MRFS			HA07_JOHN5_BND_DES_Q20_Final_MRFS
HA07_JOHN5_M11_DES_Q50_Final_MRFS			HA07_JOHN5_BND_DES_Q50_Final_MRFS
HA07_JOHN5_M11_DES_Q100_Final_MRFS			HA07_JOHN5_BND_DES_Q100_Final_MRFS
HA07_JOHN5_M11_DES_Q200_Final_MRFS			HA07_JOHN5_BND_DES_Q200_Final_MRFS
HA07_JOHN5_M11_DES_Q1000_Final_MRFS			HA07_JOHN5_BND_DES_Q1000_Final_MRFS
HA07_JOHN5_M11_DES_Q10_Final_HEFS			HA07_JOHN5_BND_DES_Q10_Final_HEFS
HA07_JOHN5_M11_DES_Q100_Final_HEFS			HA07_JOHN5_BND_DES_Q100_Final_HEFS
HA07_JOHN5_M11_DES_Q1000_Final_HEFS			HA07_JOHN5_BND_DES_Q1000_Final_HEFS

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_JOHN5_DFS0_Q2_Final	HA07_JOHN5_HD_DES_Q2_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q2	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q2
HA07_JOHN5_DFS0_Q5_Final	HA07_JOHN5_HD_DES_Q5_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q5	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q5
HA07_JOHN5_DFS0_Q10_Final	HA07_JOHN5_HD_DES_Q10_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q10	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q10
HA07_JOHN5_DFS0_Q20_Final	HA07_JOHN5_HD_DES_Q20_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q20	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q20
HA07_JOHN5_DFS0_Q50_Final	HA07_JOHN5_HD_DES_Q50_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q50	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q50
HA07_JOHN5_DFS0_Q100_Final	HA07_JOHN5_HD_DES_Q100_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q100	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q100
HA07_JOHN5_DFS0_Q200_Final	HA07_JOHN5_HD_DES_Q200_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q200	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q200
HA07_JOHN5_DFS0_Q1000_Final	HA07_JOHN5_HD_DES_Q1000_Final	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q1000	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q1000
HA07_JOHN5_DFS0_Q2_Final_MRFS	HA07_JOHN5_HD_DES_Q2_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q2_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q2_MRFS
HA07_JOHN5_DFS0_Q5_Final_MRFS	HA07_JOHN5_HD_DES_Q5_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q5_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q5_MRFS
HA07_JOHN5_DFS0_Q10_Final_MRFS	HA07_JOHN5_HD_DES_Q10_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q10_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q10_MRFS
HA07_JOHN5_DFS0_Q20_Final_MRFS	HA07_JOHN5_HD_DES_Q20_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q20_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q20_MRFS
HA07_JOHN5_DFS0_Q50_Final_MRFS	HA07_JOHN5_HD_DES_Q50_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q50_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q50_MRFS
HA07_JOHN5_DFS0_Q100_Final_MRFS	HA07_JOHN5_HD_DES_Q100_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q100_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q100_MRFS
HA07_JOHN5_DFS0_Q200_Final_MRFS	HA07_JOHN5_HD_DES_Q200_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q200_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q200_MRFS
HA07_JOHN5_DFS0_Q1000_Final_MRFS	HA07_JOHN5_HD_DES_Q1000_Final_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q1000_MRFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q1000_MRFS
HA07_JOHN5_DFS0_Q10_Final_HEFS	HA07_JOHN5_HD_DES_Q10_Final_HEFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q10_HEFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q10_HEFS
HA07_JOHN5_DFS0_Q100_Final_HEFS	HA07_JOHN5_HD_DES_Q100_Final_HEFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q100_HEFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q100_HEFS
HA07_JOHN5_DFS0_Q1000_Final_HEFS	HA07_JOHN5_HD_DES_Q1000_Final_HEFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q1000_HEFS	HA07_JOHNSTOWN_BRIDGE_5_M11_DES_Q1000_HEFS
Q2 JBridge DS Boundary_2	lwd_mrg_clip		
Q5 JBridge DS Boundary_2	lwd_mrg_clip_addareas		
Q10 JBridge DS Boundary_2			
Q20 JBridge DS Boundary_2			
Q50 JBridge DS Boundary_2			
Q100 JBridge DS Boundary_2			
Q200 JBridge DS Boundary_2			
Q1000 JBridge DS Boundary_2			
Q2 JBridge DS Boundary_2_MRFS			

MIKE 11 - DFS0 FILE Continued	MIKE 11 – HD FILE Continued	MIKE 11 - RESULTS FILE Continued	HD RESULTS FILE Continued
Q5 JBridge DS Boundary_2_MRFS Q10 JBridge DS Boundary_2_MRFS Q20 JBridge DS Boundary_2_MRFS Q50 JBridge DS Boundary_2_MRFS Q100 JBridge DS Boundary_2_MRFS Q200 JBridge DS Boundary_2_MRFS Q1000 JBridge DS Boundary_2_MRFS Q10 JBridge DS Boundary_2_HEFS Q100 JBridge DS Boundary_2_HEFS Q1000 JBridge DS Boundary_2_HEFS			

Model Files - Sensitivity

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_JOHN5_MF_SEN_1_Q100_flow HA07_JOHN5_MF_SEN_1_Q100_fpr HA07_JOHN5_MF_SEN_1_Q100_hl_1 HA07_JOHN5_MF_SEN_1_Q100_hl_2 HA07_JOHN5_MF_SEN_1_Q100_rough	HA07_JOHN5_M21_SEN_1_Q100_flow HA07_JOHN5_M21_SEN_1_Q100_fpr HA07_JOHN5_M21_SEN_1_Q100_hl_1 HA07_JOHN5_M21_SEN_1_Q100_hl_2 HA07_JOHN5_M21_SEN_1_Q100_rough HA07_JOHN5_DFS_BATHY_1 HA07_JOHN5_DFS2_FPR HA07_JOHN5_DFS2_FPR_rough		HA07_JOHN5_M21_SEN_1_Q100_flow HA07_JOHN5_M21_SEN_Q100_fpr HA07_JOHN5_M21_SEN_Q100_hl_1 HA07_JOHN5_M21_SEN_Q100_hl_2 HA07_JOHN5_M21_SEN_Q100_rough

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_JOHN5_M11_SEN_1_Q100_flow HA07_JOHN5_M11_SEN_1_Q100_fpr HA07_JOHN5_M11_SEN_1_Q100_hl_1 HA07_JOHN5_M11_SEN_1_Q100_hl_2 HA07_JOHN5_M11_SEN_1_Q100_rough	J_Bridge_02 J_Bridge_02_SEN_1_Q100_flow J_Bridge_02_SEN_1_Q100_hl_1 J_Bridge_02_SEN_1_Q100_hl_2	HA07_JOHN5_XNS_DES_3_Final_12 HA07_JOHN5_XNS_SEN_1_Q100_flow HA07_JOHN5_XNS_SEN_1_Q100_hl_1 HA07_JOHN5_XNS_SEN_1_Q100_hl_2 HA07_JOHN5_XNS_SEN_Final_12_rough	HA07_JOHN5_BND_DES_Q100_Final HA07_JOHN5_BND_SEN_1_Q100_flow

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_JOHN5_DFS0_Q100_Final HA07_JOHN5_DFS0_SEN_1_Q100_flow Q100 JBridge DS Boundary_2	HA07_JOHN5_HD_SEN_1_Q100_flow HA07_JOHN5_HD_SEN_1_Q100_fpr HA07_JOHN5_HD_SEN_1_Q100_hl_1 HA07_JOHN5_HD_SEN_1_Q100_hl_2 HA07_JOHN5_HD_SEN_1_Q100_rough	HA07_JOHNSTOWN_BRIDGE_5_M11_SEN_1_Q100_flow HA07_JOHNSTOWN_BRIDGE_5_M11_SEN_1_Q100_fpr HA07_JOHNSTOWN_BRIDGE_5_M11_SEN_1_Q100_rough HA07_JOHNSTOWN_BRIDGE_5_M11_SEN_Q100_hl_1 HA07_JOHNSTOWN_BRIDGE_5_M11_SEN_Q100_hl_2	N/A

GIS Deliverables - Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u> E20EXFCD500F0 E20EXFCD200F0 E20EXFCD100F0 E20EXFCD050F0 E20EXFCD020F0 E20EXFCD010F0 E20EXFCD005F0 E20EXFCD001F0	Water Level and Flows <u>Fluvial</u> E20NFCDF0 E20NFMDF0 E20NFHDF0 Flood Defences (Shapefiles) N/A Defended Areas (Shapefiles) N/A Wave Overtopping (Shapefiles) N/A	<u>Fluvial</u> E20DPFCD500F0 E20DPFCD200F0 E20DPFCD100F0 E20DPFCD050F0 E20DPFCD020F0 E20DPFCD010F0 E20DPFCD005F0 E20DPFCD001F0 E20DPFMD500F0 E20DPFMD200F0 E20DPFMD100F0 E20DPFMD050F0 E20DPFMD020F0 E20DPFMD010F0 E20DPFMD005F0 E20DPFMD001F0 E20DPFHD100F0 E20DPFHD010F0 E20DPFHD001F0	<u>Fluvial</u> E20VLFCDF0500F0 E20VLFCDF0200F0 E20VLFCDF0100F0 E20VLFCDF0050F0 E20VLFCDF0020F0 E20VLFCDF0010F0 E20VLFCDF0005F0 E20VLFCDF0001F0	<u>Fluvial</u> E20RLFCD100F0 E20RLFCD010F0 E20RLFCD001F0 Flood Zones (Shapefiles) E20ZNA_FCDF0 E20ZNB_FCDF0 E20ZNA_FMDF0 E20ZNB_FMDF0
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables – Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E20RIFCD100F0 E20RIFCD010F0 E20RIFCD001F0 E20RIFMD100F0 E20RIFMD010F0 E20RIFMD001F0	<u>Fluvial</u> E07_RTFC001_F0 E07_RTFCMD001_F0	<u>Fluvial</u> E20RDFCD001F0 E20RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix F

Longwood AFA Additional Information

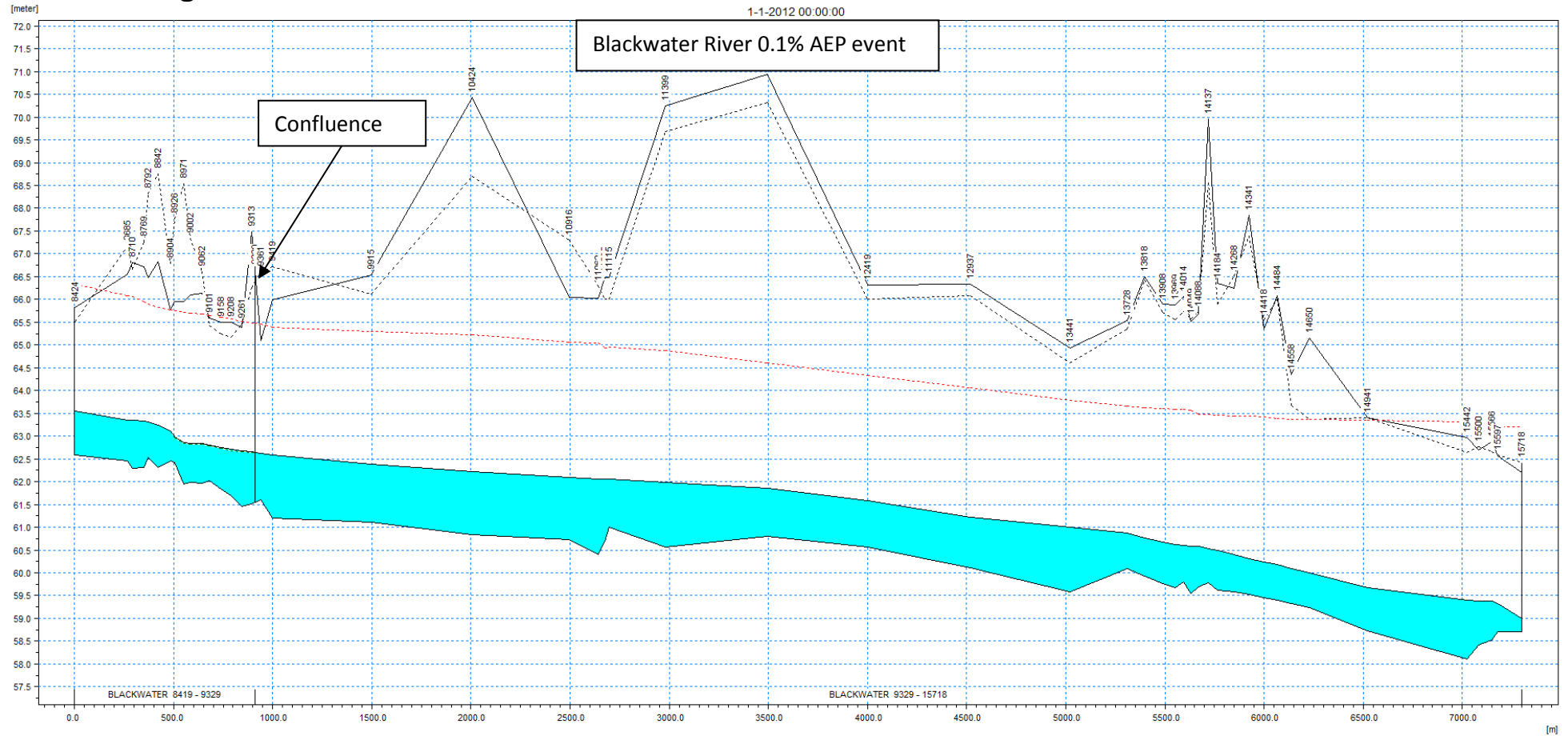
List of background information included:

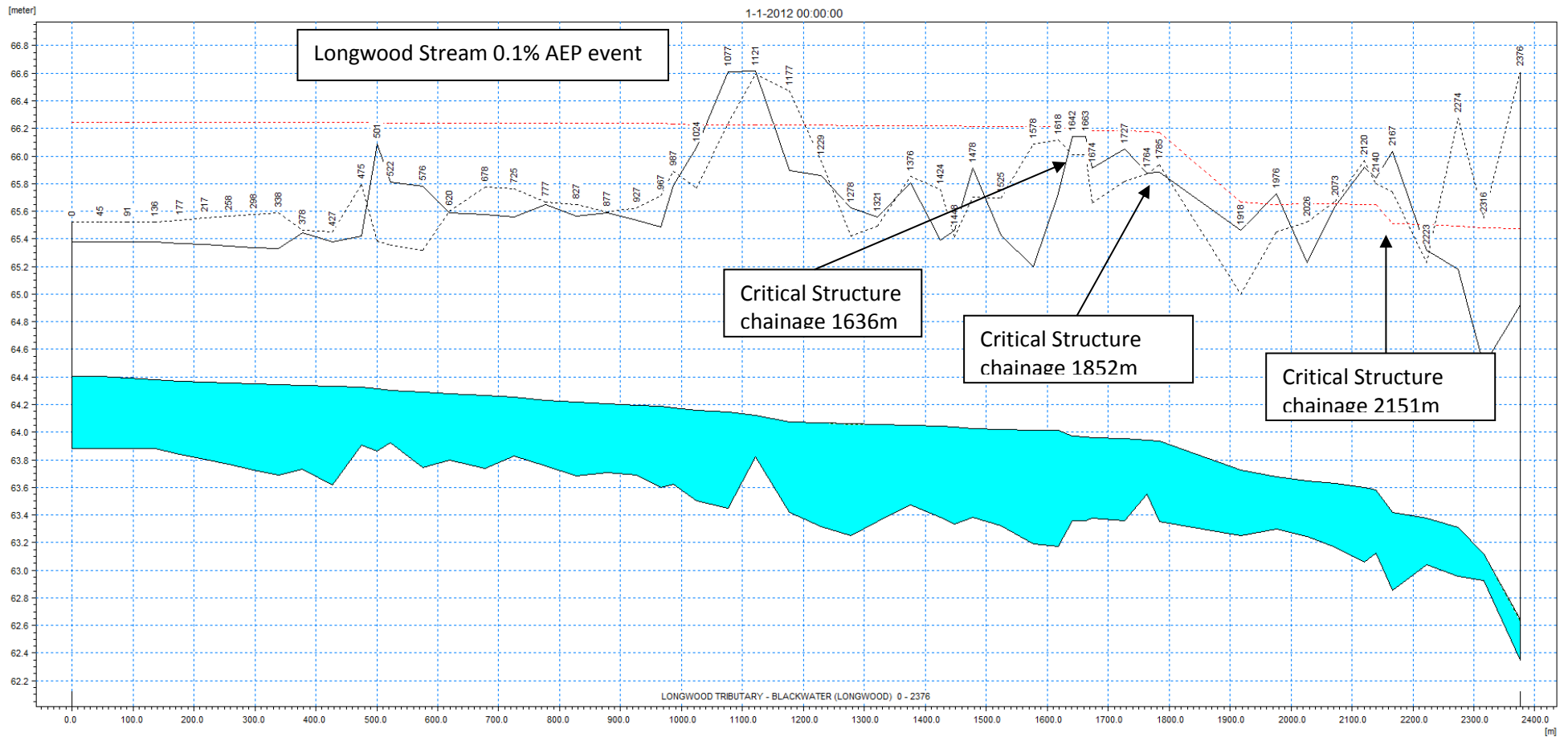
- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

1D Structures

	Branch	Chainage	ID	Length	Opening Shape	Height (m)	Width (m)	Spring Height from invert (m)	Mannings n
1	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	353.2	0736_00203I	3.84	Circular	1	-	-	0.03
2	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	486.8	0736_00190I	4.27	Cross Section DB	2.17	1.13	-	0.03
3	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	977.4	0736_00141i	4.81	Circular	1.05	-	-	0.03
4	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	997	0736_00139I	5.54	Circular	1.05	-	-	0.03
5	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	1238.5	0736_00115I	4.6	Circular	1.05	-	-	0.03
6	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	1464	0736_00093I	4.18	Circular	0.7	-	-	0.03
7	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	1635.9	0736_00075I	5.76	Circular	0.85	-	-	0.03
8	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	1651.6	0736_00074D	3.75	Cross Section DB	2.235	10.06	-	0.07
9	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	1851.8	0736_00049J	91.71	Cross Section DB	2.1	1.25	-	0.013
10	LONGWOOD TRIBUTARY - BLACKWATER (LONGWOOD)	2150.7	0736_00023I	3.95	Circular	1	-	-	0.03
11	BLACKWATER	8745.5	0735_00696D	3.23	Cross Section DB	4.31	8.9	-	0.035
12	BLACKWATER	9041.8	0735_00668D	7.67	Cross Section DB	5.2	11.26	2.56	0.035
13	BLACKWATER	11074.41	0735_00464D	8.67	Cross Section DB	5.23	10.8	2.65	0.035
14	BLACKWATER	14068.5	0735_00164E	8	Cross Section DB	4.88	9.47	2.92	0.013
15	BLACKWATER	15536.8	0735_00018D	7.32	Cross Section DB	4.36	11.24	2.2	0.035
16	BLACKWATER	15536.8	0735_00018D_subarch1	7.32	Irregular, Level-Width Table	0.95	1.57	0.56	0.035
17	BLACKWATER	15536.8	0735_00018D_subarch2	7.32	Irregular, Level-Width Table	1.45	2.53	0.98	0.035
18	BLACKWATER	15536.8	0735_00018D_subarch3	7.32	Irregular, Level-Width Table	1.55	3.19	1.26	0.035
19	BLACKWATER	15536.8	0735_00018D_subarch4	7.32	Irregular, Level-Width Table	1.33	2.48	0.96	0.035
20	BLACKWATER	14068.5	0735_00164E_subarch1	7.62	Irregular, Level-Width Table	1.33	3.11	0.84	0.035
21	BLACKWATER	14068.5	0735_00164E_subarch2	7.62	Irregular, Level-Width Table	1.47	2.82	0.92	0.035
22	BLACKWATER	14068.5	0735_00164E_subarch3	7.62	Irregular, Level-Width Table	1.56	2.86	1	0.035
23	BLACKWATER	14068.5	0735_00164E_subarch4	7.62	Irregular, Level-Width Table	1.35	2.23	0.89	0.035

River Long Section Profiles





Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_LONG8_MF_DES_Q2	HA07_LONG8_M21_DES_Q2		N/A
HA07_LONG8_MF_DES_Q5	HA07_LONG8_M21_DES_Q5		
HA07_LONG8_MF_DES_Q10	HA07_LONG8_M21_DES_Q10		
HA07_LONG8_MF_DES_Q20	HA07_LONG8_M21_DES_Q20		
HA07_LONG8_MF_DES_Q50	HA07_LONG8_M21_DES_Q50		
HA07_LONG8_MF_DES_Q100	HA07_LONG8_M21_DES_Q100		
HA07_LONG8_MF_DES_Q200	HA07_LONG8_M21_DES_Q200		
HA07_LONG8_MF_DES_Q1000	HA07_LONG8_M21_DES_Q1000		
HA07_LONG8_MF_DES_Q2_MRFS	HA07_LONG8_M21_DES_Q2_MRFS		
HA07_LONG8_MF_DES_Q5_MRFS	HA07_LONG8_M21_DES_Q5_MRFS		
HA07_LONG8_MF_DES_Q10_MRFS	HA07_LONG8_M21_DES_Q10_MRFS		
HA07_LONG8_MF_DES_Q20_MRFS	HA07_LONG8_M21_DES_Q20_MRFS		
HA07_LONG8_MF_DES_Q50_MRFS	HA07_LONG8_M21_DES_Q50_MRFS		
HA07_LONG8_MF_DES_Q100_MRFS	HA07_LONG8_M21_DES_Q100_MRFS		
HA07_LONG8_MF_DES_Q200_MRFS	HA07_LONG8_M21_DES_Q200_MRFS		
HA07_LONG8_MF_DES_Q1000_MRFS_1	HA07_LONG8_M21_DES_Q1000_MRFS_1		
HA07_LONG8_MF_DES_Q10_HEFS	HA07_LONG8_M21_DES_Q10_HEFS		
HA07_LONG8_MF_DES_2_Q100_HEFS_1	HA07_LONG8_M21_DES_2_Q100_HEFS		
HA07_LONG8_MF_DES_Q1000_HEFS	HA07_LONG8_M21_DES_Q1000_HEFS		
	HA07_LONG8_DFS2_BATHY_7		
	HA07_LONG8_DFS2_FPR_1		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_LONG8_M11_DES_Q2	HA07_LONG8_NWK_DES_5	HA07_LONG8_XNS_DES_6	HA07_LONG8_BND_DES_Q2
HA07_LONG8_M11_DES_Q5	HA07_LONG8_NWK_DES_5_Q100H	HA07_LONG8_XNS_DES_6_Q100H	HA07_LONG8_BND_DES_Q5
HA07_LONG8_M11_DES_Q10			HA07_LONG8_BND_DES_Q10
HA07_LONG8_M11_DES_Q20			HA07_LONG8_BND_DES_Q20
HA07_LONG8_M11_DES_Q50			HA07_LONG8_BND_DES_Q50
HA07_LONG8_M11_DES_Q100			HA07_LONG8_BND_DES_Q100
HA07_LONG8_M11_DES_Q200			HA07_LONG8_BND_DES_Q200
HA07_LONG8_M11_DES_Q1000			HA07_LONG8_BND_DES_Q1000
HA07_LONG8_M11_DES_Q2_MRFS			HA07_LONG8_BND_DES_Q2_MRFS
HA07_LONG8_M11_DES_Q5_MRFS			HA07_LONG8_BND_DES_Q5_MRFS
HA07_LONG8_M11_DES_Q10_MRFS			HA07_LONG8_BND_DES_Q10_MRFS
HA07_LONG8_M11_DES_Q20_MRFS			HA07_LONG8_BND_DES_Q20_MRFS
HA07_LONG8_M11_DES_Q50_MRFS			HA07_LONG8_BND_DES_Q50_MRFS
HA07_LONG8_M11_DES_Q100_MRFS			HA07_LONG8_BND_DES_Q100_MRFS
HA07_LONG8_M11_DES_Q200_MRFS			HA07_LONG8_BND_DES_Q200_MRFS
HA07_LONG8_M11_DES_Q1000_MRFS_1			HA07_LONG8_BND_DES_Q1000_MRFS
HA07_LONG8_M11_DES_Q10_HEFS			HA07_LONG8_BND_DES_Q10_HEFS
HA07_LONG8_M11_DES_2_Q100_HEFS			HA07_LONG8_BND_DES_Q100_HEFS
HA07_LONG8_M11_DES_Q1000_HEFS			HA07_LONG8_BND_DES_Q1000_HEFS

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_JOHN5_DFS0_Q2	HA07_LONG8_HD_DES_Q2	HA07_LONGWOOD_8_M11_DES_Q2	HA07_LONG8_M11_DES_Q2
HA07_JOHN5_DFS0_Q5	HA07_LONG8_HD_DES_Q5	HA07_LONGWOOD_8_M11_DES_Q5	HA07_LONG8_M11_DES_Q5
HA07_JOHN5_DFS0_Q10	HA07_LONG8_HD_DES_Q10	HA07_LONGWOOD_8_M11_DES_Q10	HA07_LONG8_M11_DES_Q10
HA07_JOHN5_DFS0_Q20	HA07_LONG8_HD_DES_Q20	HA07_LONGWOOD_8_M11_DES_Q20	HA07_LONG8_M11_DES_Q20
HA07_JOHN5_DFS0_Q50	HA07_LONG8_HD_DES_Q50	HA07_LONGWOOD_8_M11_DES_Q50	HA07_LONG8_M11_DES_Q50
HA07_JOHN5_DFS0_Q100	HA07_LONG8_HD_DES_Q100	HA07_LONGWOOD_8_M11_DES_Q100	HA07_LONG8_M11_DES_Q100
HA07_JOHN5_DFS0_Q200	HA07_LONG8_HD_DES_Q200	HA07_LONGWOOD_8_M11_DES_Q200	HA07_LONG8_M11_DES_Q200
HA07_JOHN5_DFS0_Q1000	HA07_LONG8_HD_DES_Q1000	HA07_LONGWOOD_8_M11_DES_Q1000	HA07_LONG8_M11_DES_Q1000
HA07_JOHN5_DFS0_Q2_MRFS	HA07_LONG8_HD_DES_Q2_MRFS	HA07_LONGWOOD_8_M11_DES_Q2_MRFS	HA07_LONG8_M11_DES_Q2_MRFS
HA07_JOHN5_DFS0_Q5_MRFS	HA07_LONG8_HD_DES_Q5_MRFS	HA07_LONGWOOD_8_M11_DES_Q5_MRFS	HA07_LONG8_M11_DES_Q5_MRFS
HA07_JOHN5_DFS0_Q10_MRFS	HA07_LONG8_HD_DES_Q10_MRFS	HA07_LONGWOOD_8_M11_DES_Q10_MRFS	HA07_LONG8_M11_DES_Q10_MRFS
HA07_JOHN5_DFS0_Q20_MRFS	HA07_LONG8_HD_DES_Q20_MRFS	HA07_LONGWOOD_8_M11_DES_Q20_MRFS	HA07_LONG8_M11_DES_Q20_MRFS
HA07_JOHN5_DFS0_Q50_MRFS	HA07_LONG8_HD_DES_Q50_MRFS	HA07_LONGWOOD_8_M11_DES_Q50_MRFS	HA07_LONG8_M11_DES_Q50_MRFS
HA07_JOHN5_DFS0_Q100_MRFS	HA07_LONG8_HD_DES_Q100_MRFS	HA07_LONGWOOD_8_M11_DES_Q100_MRFS	HA07_LONG8_M11_DES_Q100_MRFS
HA07_JOHN5_DFS0_Q200_MRFS	HA07_LONG8_HD_DES_Q200_MRFS	HA07_LONGWOOD_8_M11_DES_Q200_MRFS	HA07_LONG8_M11_DES_Q200_MRFS
HA07_JOHN5_DFS0_Q1000_MRFS	HA07_LONG8_HD_DES_Q1000_MRFS_1	HA07_LONGWOOD_8_M11_DES_Q1000_MRFS_1	HA07_LONG8_M11_DES_Q1000_MRFS_1
HA07_JOHN5_DFS0_Q10_HEFS	HA07_LONG8_HD_DES_Q10_HEFS	HA07_LONGWOOD_8_M11_DES_Q10_HEFS	HA07_LONG8_M11_DES_Q10_HEFS
HA07_JOHN5_DFS0_Q100_HEFS	HA07_LONG8_HD_DES_2_Q100_HEFS	HA07_LONGWOOD_8_M11_DES_2_Q100_HEFS	HA07_LONG8_M11_DES_2_Q100_HEFS
HA07_JOHN5_DFS0_Q1000_HEFS_1	HA07_LONG8_HD_DES_Q1000_HEFS	HA07_LONGWOOD_8_M11_DES_Q1000_HEFS	HA07_LONG8_M11_DES_Q1000_HEFS
HA07_JOHN5_CH8685_DFS0_Q2	lwd_mrg_clip		
HA07_JOHN5_CH8685_DFS0_Q5	lwd_mrg_clip_addareas		
HA07_JOHN5_CH8685_DFS0_Q10			
HA07_JOHN5_CH8685_DFS0_Q20			
HA07_JOHN5_CH8685_DFS0_Q50			
HA07_JOHN5_CH8685_DFS0_Q100			
HA07_JOHN5_CH8685_DFS0_Q200			
HA07_JOHN5_CH8685_DFS0_Q1000			
HA07_JOHN5_CH8685_DFS0_Q2_MRFS			

MIKE 11 - DFS0 FILE Continued	MIKE 11 – HD FILE Continued	MIKE 11 - RESULTS FILE Continued	HD RESULTS FILE Continued
HA07_JOHN5_CH8685_DFS0_Q5_MRFS HA07_JOHN5_CH8685_DFS0_Q10_MRFS HA07_JOHN5_CH8685_DFS0_Q20_MRFS HA07_JOHN5_CH8685_DFS0_Q50_MRFS HA07_JOHN5_CH8685_DFS0_Q100_MRFS HA07_JOHN5_CH8685_DFS0_Q200_MRFS HA07_JOHN5_CH8685_DFS0_Q1000_MRFS HA07_JOHN5_CH8685_DFS0_Q10_HEFS HA07_JOHN5_CH8685_DFS0_Q100_HEFS HA07_JOHN5_CH8685_DFS0_Q1000_HEFS Q2 JBridge DS Boundary_2 Q5 JBridge DS Boundary_2 Q10 JBridge DS Boundary_2 Q20 JBridge DS Boundary_2 Q50 JBridge DS Boundary_2 Q100 JBridge DS Boundary_2 Q200 JBridge DS Boundary_2 Q1000 JBridge DS Boundary_2 Q2 JBridge DS Boundary_2_MRFS Q5 JBridge DS Boundary_2_MRFS Q10 JBridge DS Boundary_2_MRFS Q20 JBridge DS Boundary_2_MRFS Q50 JBridge DS Boundary_2_MRFS Q100 JBridge DS Boundary_2_MRFS Q200 JBridge DS Boundary_2_MRFS Q1000 JBridge DS Boundary_2_MRFS_rev1 Q10 JBridge DS Boundary_2_HEFS Q100 JBridge DS Boundary_2_HEFS Q1000 JBridge DS Boundary_2_HEFS			

Model Files - Sensitivity

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_LONG8_MF_SEN_1_Q100_flow HA07_LONG8_MF_SEN_Q100_fpr HA07_LONG8_MF_SEN_Q100_fv HA07_LONG8_MF_SEN_Q100_hl_1 HA07_LONG8_MF_SEN_Q100_rough	HA07_LONG8_M21_SEN_1_Q100_flow HA07_LONG8_M21_SEN_Q100_fpr HA07_LONG8_M21_SEN_Q100_fv HA07_LONG8_M21_SEN_Q100_hl_1 HA07_LONG8_M21_SEN_Q100_rough HA07_LONG8_DFS2_BATHY_7 HA07_LONG8_DFS2_FPR_1 HA07_LONG8_DFS2_FPR_1_rough		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_LONG8_M11_SEN_1_Q100_flow HA07_LONG8_M11_SEN_Q100_fpr HA07_LONG8_M11_SEN_Q100_fv HA07_LONG8_M11_SEN_Q100_hl_1 HA07_LONG8_M11_SEN_Q100_rough	HA07_LONG8_NWK_DES_5 HA07_LONG8_NWK_SEN_1_Q100_flow HA07_LONG8_NWK_SEN_hl_1	HA07_LONG8_XNS_DES_6 HA07_LONG8_XNS_SEN_1_Q100_flow HA07_LONG8_XNS_SEN_hl_1 HA07_LONG8_XNS_SEN_rough	HA07_LONG8_BND_DES_Q100 HA07_LONG8_BND_SEN_1_Q100_flow HA07_LONG8_BND_SEN_Q100_fv

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_JOHN5_DFS0_Q100 HA07_JOHN5_DFS0_SEN_1_Q100_flow HA07_JOHN5_DFS0_SEN_Q100_fv HA07_JOHN5_CH8685_DFS0_Q100 HA07_JOHN5_CH8685_DFS0_SEN_Q100_flow HA07_JOHN5_CH8685_DFS0_SEN_Q100_fv Q100 JBridge DS Boundary_2	HA07_LONG8_HD_SEN_1_Q100_flow HA07_LONG8_HD_SEN_Q100_fpr HA07_LONG8_HD_SEN_Q100_fv HA07_LONG8_HD_SEN_Q100_hl_1 HA07_LONG8_HD_SEN_Q100_rough	HA07_LONGWOOD_8_M11_SEN_1_Q100_flow HA07_LONGWOOD_8_M11_SEN_Q100_fpr HA07_LONGWOOD_8_M11_SEN_Q100_fv HA07_LONGWOOD_8_M11_SEN_Q100_hl_1 HA07_LONGWOOD_8_M11_SEN_Q100_rough	HA07_LONG8_M21_SEN_Q100_flow HA07_LONG8_M21_SEN_Q100_fpr HA07_LONG8_M21_SEN_Q100_fv HA07_LONG8_M21_SEN_Q100_hl_1 HA07_LONG8_M21_SEN_Q100_rough

GIS Deliverables - Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u> E25EXFCD500F0 E25EXFCD200F0 E25EXFCD100F0 E25EXFCD050F0 E25EXFCD020F0 E25EXFCD010F0 E25EXFCD005F0 E25EXFCD001F0	Water Level and Flows <u>Fluvial</u> E25NFCDF0 E25NFMDF0 E25NFHDF0 Flood Defences (Shapefiles) N/A Defended Areas (Shapefiles) N/A Wave Overtopping (Shapefiles) N/A	<u>Fluvial</u> E25DPFCD500F0 E25DPFCD200F0 E25DPFCD100F0 E25DPFCD050F0 E25DPFCD020F0 E25DPFCD010F0 E25DPFCD005F0 E25DPFCD001F0 E25DPFMD500F0 E25DPFMD200F0 E25DPFMD100F0 E25DPFMD050F0 E25DPFMD020F0 E25DPFMD010F0 E25DPFMD005F0 E25DPFMD001F0 E25DPFHD100F0 E25DPFHD010F0 E25DPFHD001F0	<u>Fluvial</u> E25VLFCDF0500F0 E25VLFCDF0200F0 E25VLFCDF0100F0 E25VLFCDF0050F0 E25VLFCDF0020F0 E25VLFCDF0010F0 E25VLFCDF0005F0 E25VLFCDF0001F0	<u>Fluvial</u> E25RLFCD100F0 E25RLFCD010F0 E25RLFCD001F0 Flood Zones (Shapefiles) E25ZNA_FCDF0 E25ZNB_FCDF0 E25ZNA_FMDF0 E25ZNB_FMDF0
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables - Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E25RIFCD100F0 E25RIFCD010F0 E25RIFCD001F0 E25RIFMD100F0 E25RIFMD010F0 E25RIFMD001F0	<u>Fluvial</u> E07_RTFC001_F0 E07_RTMD001_F0	<u>Fluvial</u> E25RDFCD001F0 E25RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix G

Mornington AFA Additional Information

List of background information included:

- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

1D Structures

US node ID	DS node ID	Length (m)	Shape ID	Width (mm)	Height (mm)	Springing height (m)	Roughness Colebrook-White (mm)
c2_3594U_Inlet1	c2_3594D_Outlet1	9.4	CIRC	900	900	-	0.6
c2_3594U_Inlet2	c2_3594D_Outlet2	9.2	CIRC	900	900	-	0.6
c2_3594U_Inlet3	c2_3594D_Outlet3	9.2	CIRC	900	900	-	0.6
c2_2992U_Inlet1	c2_2992D_Outlet1	1.9	CIRC	1450	1450	-	0.6
c2_2992U_Inlet2	c2_2992D_Outlet2	1.8	CIRC	1450	1450	-	0.6
c2_3194U_MH	c2_3194D_MH	8.7	ARCHSPRUNG	1200	1650	1.2	0.6
North_MH_U	North_MH_D	14.4	CIRC	1150	1150	-	0.6
01BETT00220J.In	01BETT00220J.Out1	11.0	RECTANGULAR	1770	1000	-	1.5
01BETT00210I.In	01BETT00210J.Out	10.8	RECTANGULAR	1726	974	-	1.5

Mornington River Bridges

US node ID	DS node ID	Length (m)	Discharge coefficient	Contraction loss	Expansion loss	US invert level (m AD)	DS invert level (m AD)	Skew angle (degree)
Bridge_U	Bridge_D	21.6	1.7	0.3	0.5	1.325	1.325	-7.31
c2_2595U	c2_2595D	31	1.7	0.3	0.5	0.689	0.689	28.92
c2_2763U	c2_2763D	22.7	1.7	0.3	0.5	1.215	1.215	-1.09

Mornington River Bridge Openings

ID	Bridge US node ID	Length (m)	Shape ID	Width (m)	Height (m)	Springing height (m)	Roughness Manning's n	US invert level (m AD)	DS invert level (m AD)
c2_2595_Opening	c2_2595U	9.2	RECT	6.6	2.569	-	0.045	0.689	0.689
c2/2763_Opening	c2_2763U	3.2	RECT	6	2.335	-	0.045	1.215	1.215
Opening	Bridge_U	2.2	RECT	6	1.6	-	0.055	1.325	1.325

Lady's Finger Bridge / Flap Valve Elements

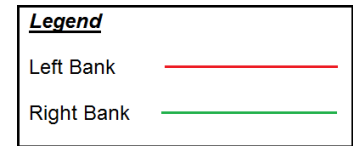
Flap Valves

ID	Bridge US node ID	Bridge DS node ID	Diameter (mm)	Invert Level (m AD)	Discharge Co-efficient
c2_3594U.2	c2_3594U	c2_3594D	900	0.887	1
c2_3594U.3	c2_3594U	c2_3594D	900	0.887	1
c2_3594U.5	c2_3594U	c2_3594D	900	0.887	1

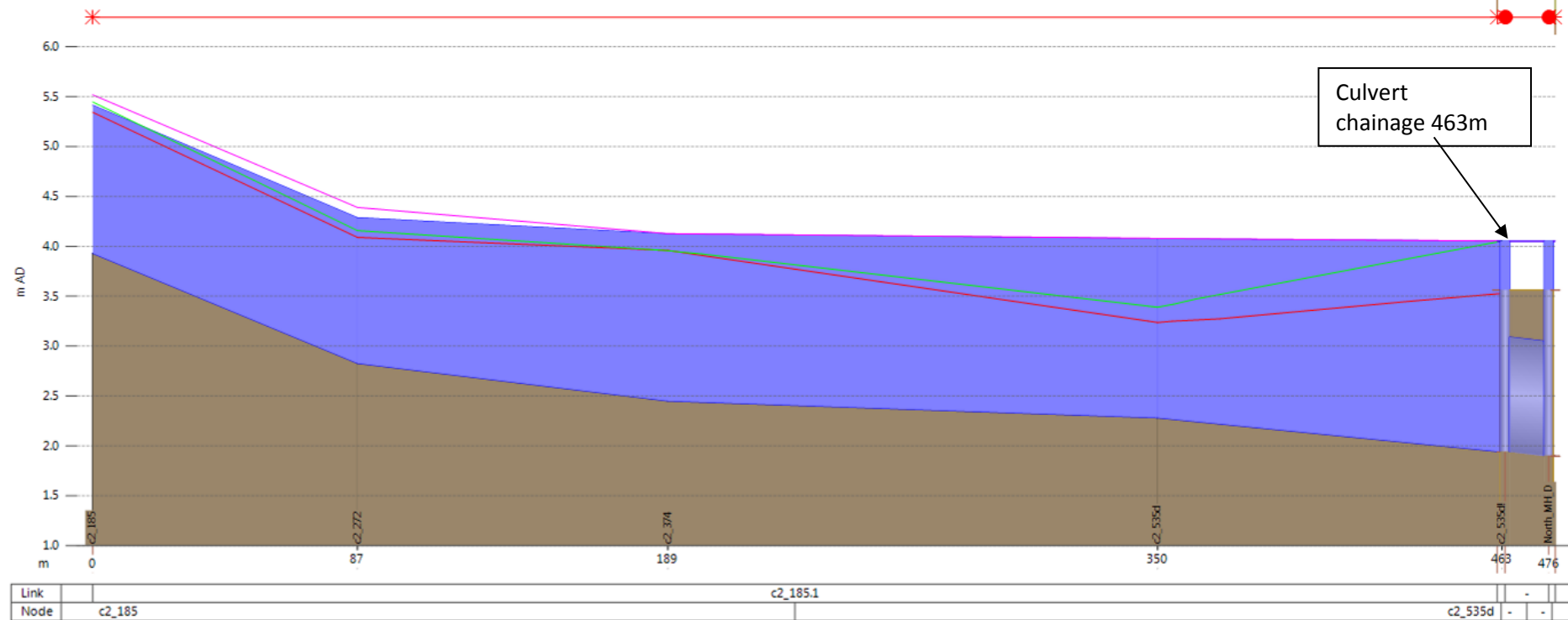
Irregular Weir (Bridge Parapet)

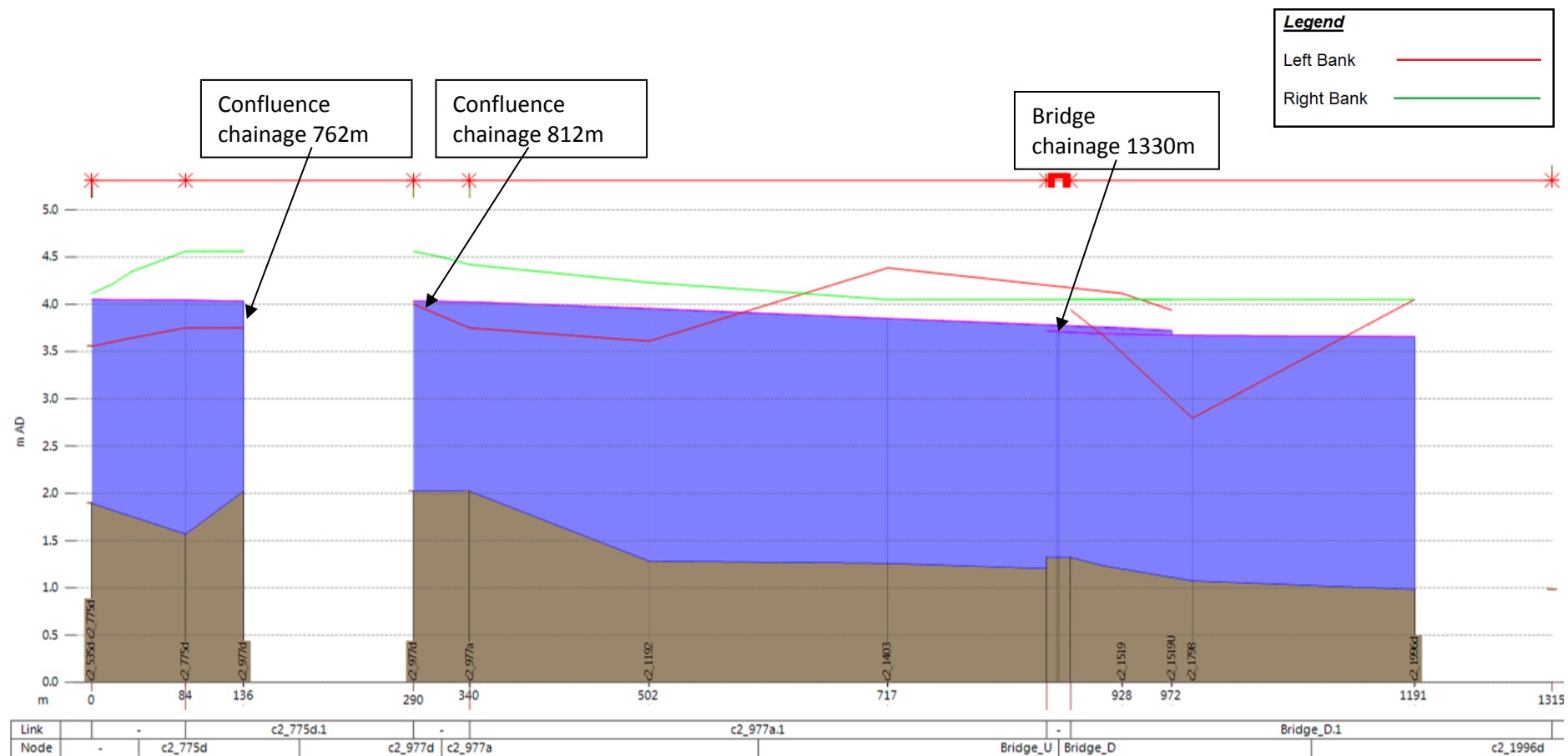
ID	Bridge US node ID	Bridge DS node ID	Crest (m AD)	Chainage / Elevation	Discharge Co-efficient	Modular Limit
c2_3594U.2	c2_3594U	c2_3594D	4.338	Varies	1.7	0.9

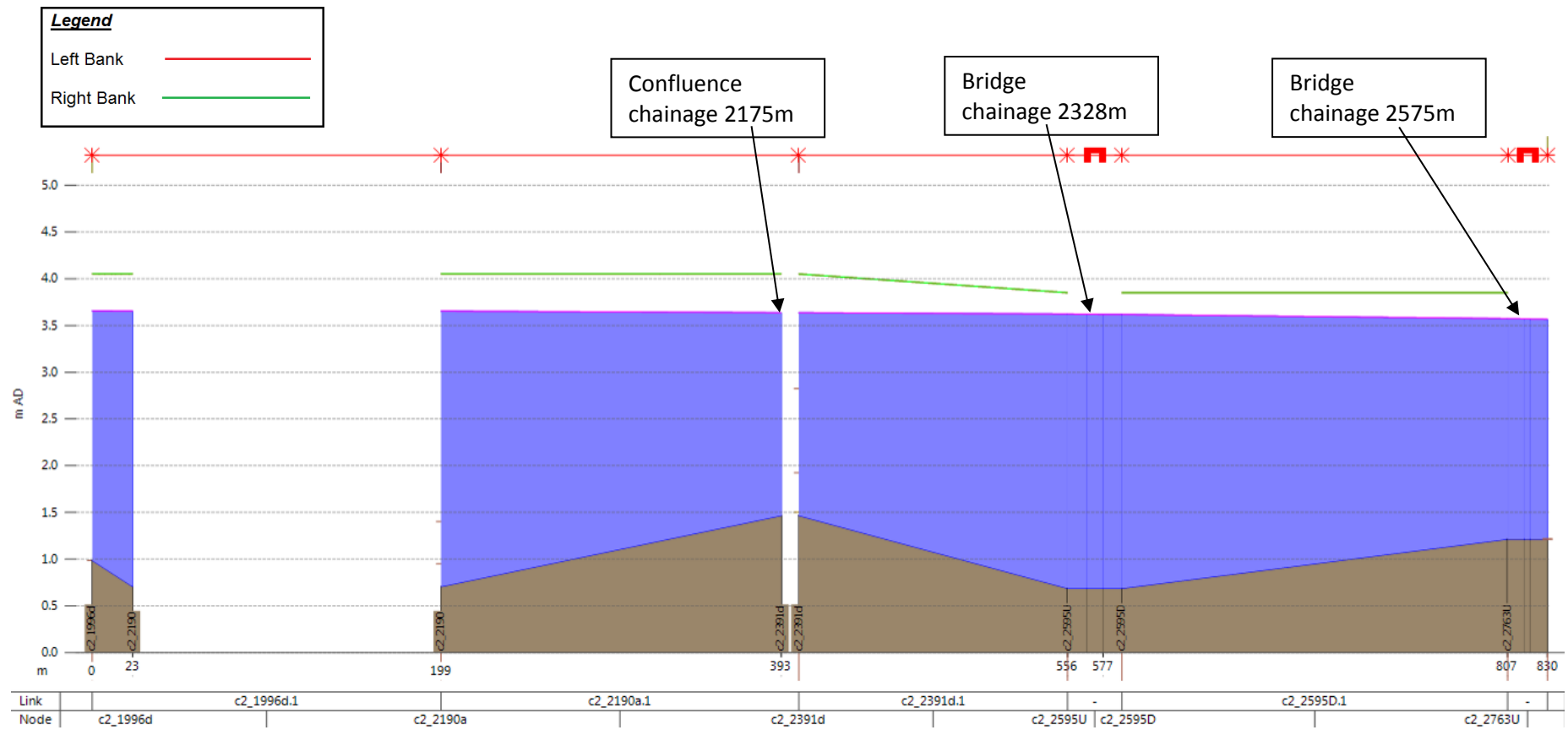
River Long Section Profiles

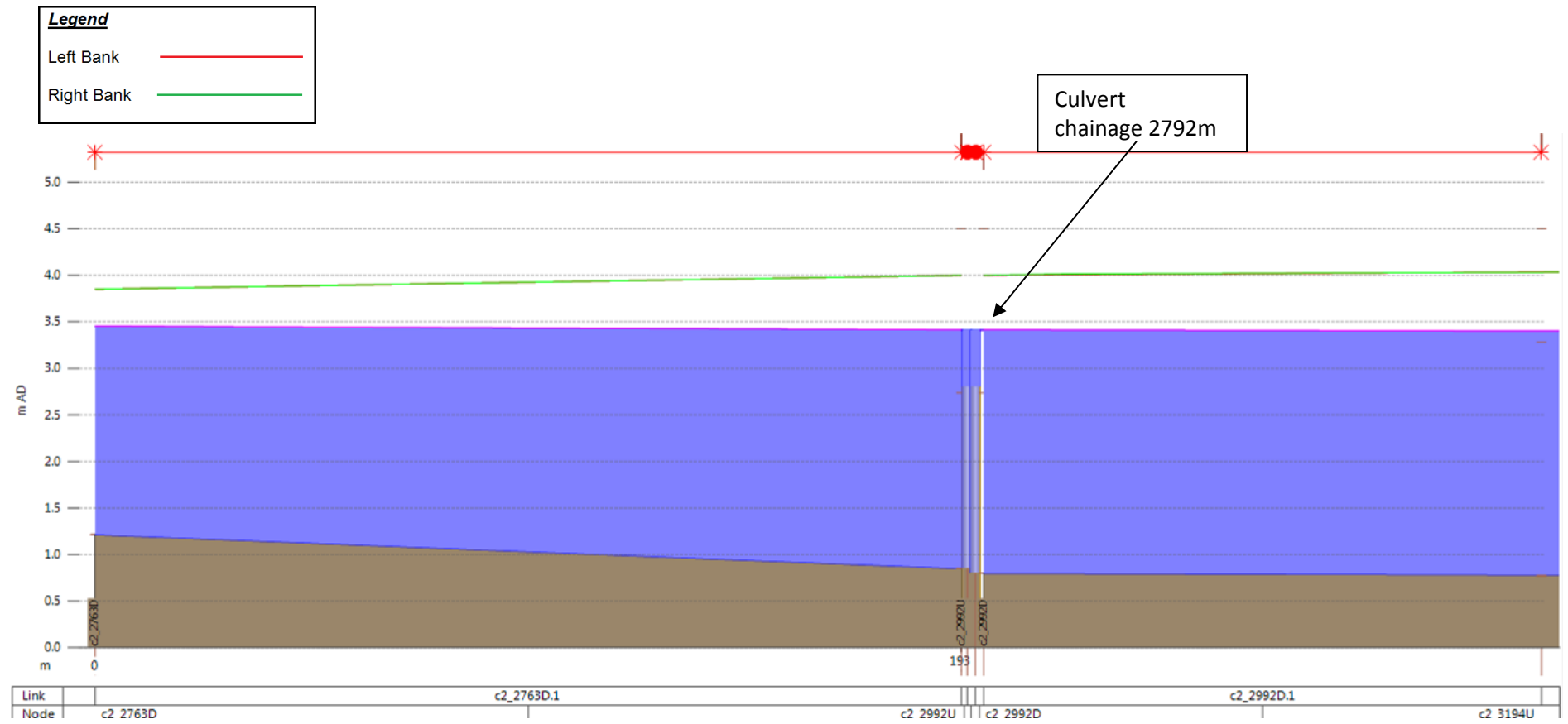


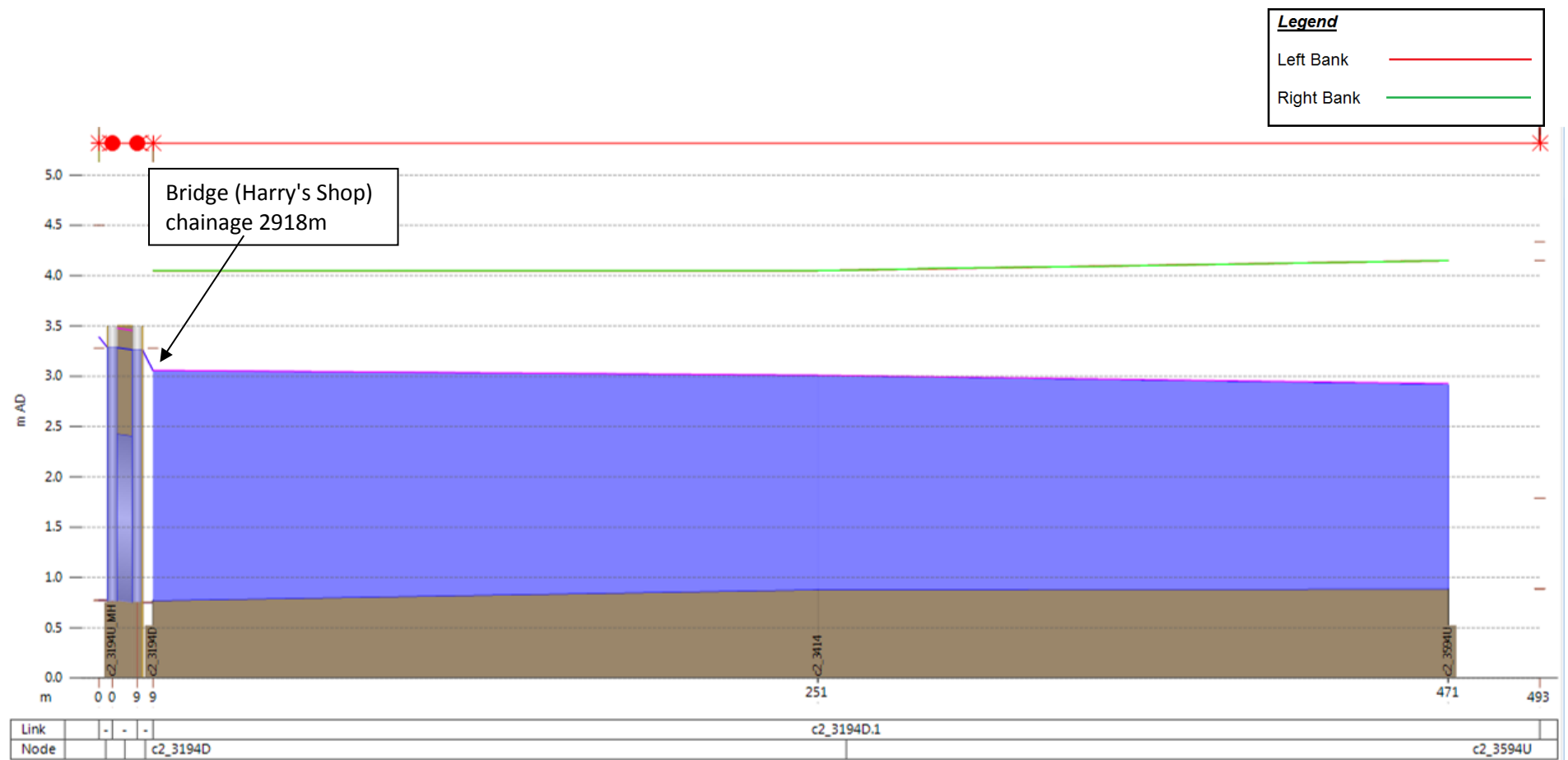
Mornington River 1% - Existing (Defended) AEP Long Sections

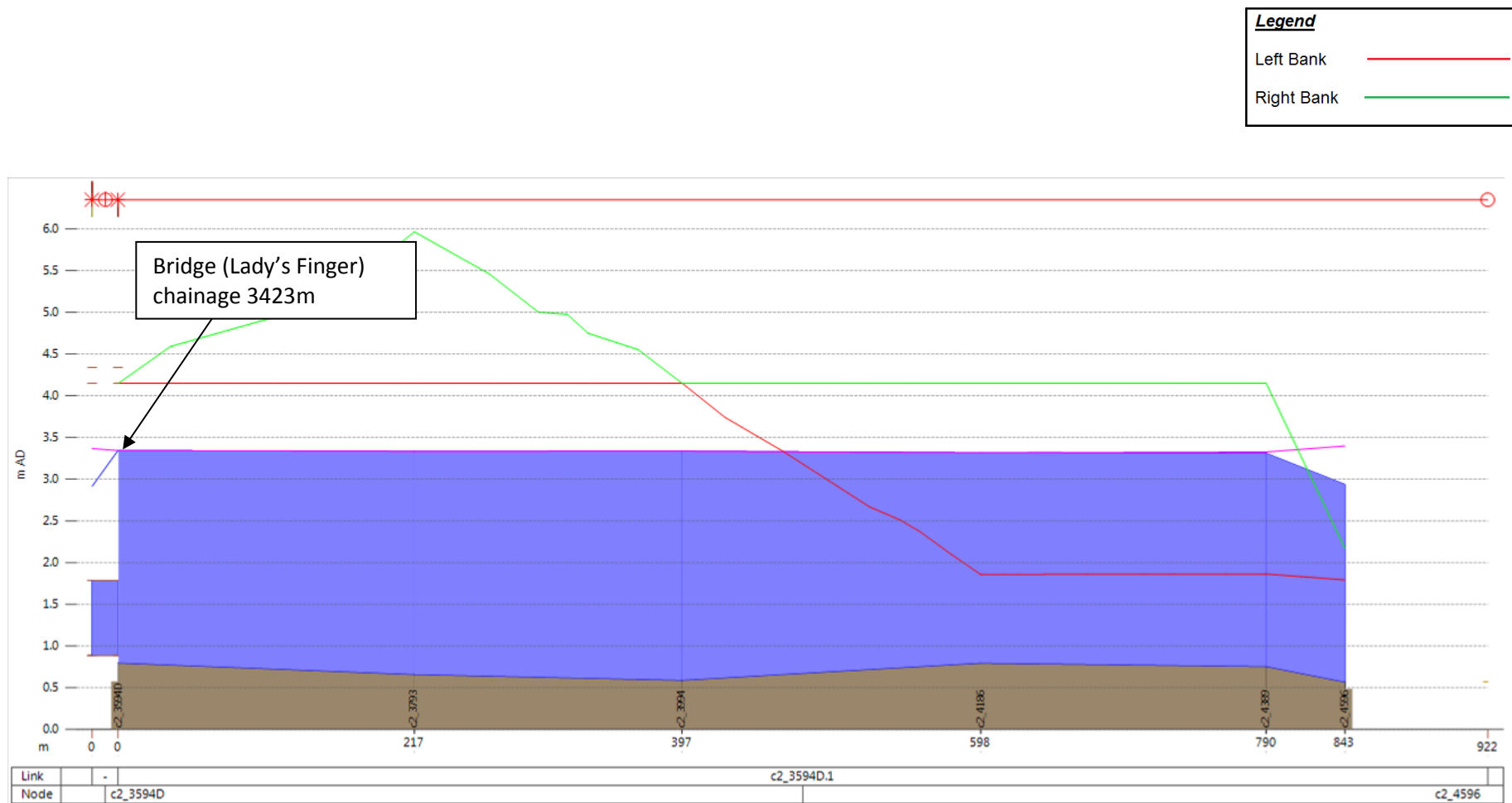












Fluvial and Mechanism 1 Tidal Model Files

ICM Transportable Database containing all relevant model files linked by simulation file for each required run – HA07_DROG07_F02.ICMT

Appendix H

Navan AFA

Additional Information

List of background information included:

- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

1D Structures

Structure Details - Bridges and Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH	OPENING SHAPE	HEIGHT	WIDTH	SPRING HEIGHT FROM INVERT (m)	MANNINGS N
			(m)		(m)	(m)		
TRIM ROAD TRIBUTARY 1 - BOYNE	1201	0714_00211D	3.76	Rectangular	0.4	0.9	N/A	0.014
TRIM ROAD TRIBUTARY 1 - BOYNE	2503	0714_00081D	15.67	Irregular	0.6	1.6	N/A	0.014
TRIM ROAD TRIBUTARY 1 - BOYNE	2579	0714_00075D	1.11	Irregular	0.8	2.9	N/A	0.013
TRIM ROAD TRIBUTARY 1 - BOYNE	3237	0714_00008D	16.59	Irregular	1	1.6	N/A	0.013
TRIM ROAD TRIBUTARY 1 - BOYNE	3272	0714_00003D	4.54	Irregular	2	4	N/A	0.013
TRIM ROAD TRIBUTARY 1 - BOYNE	1169	0714_00215I	5.61	Circular	0.6	N/A	N/A	0.015
TRIM ROAD TRIBUTARY 1 - BOYNE	1965	0714_00135I	5.02	Circular	1.2	N/A	N/A	0.015
ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	838	0716_00088I	20.54	Rectangular	0.55	1.6	N/A	0.013
ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	1364	0716_00034I	11.2	Circular	1	N/A	N/A	0.015
ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE	1603	0716_00011I	25.72	Irregular	0.5	1.5	N/A	0.014
ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE	153.5	0719_00118I	6.93	Circular	1	N/A	N/A	0.015
ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE	1209	0719_00012I	12.88	Circular	1.05	N/A	N/A	0.013
ABBEYLANDS TRIBUTARY - BLACKWATER (KELLS)	1556	0713_00084I	8.67	Circular	0.3	N/A	N/A	0.015
BAILIS TRIBUTARY - BOYNE	657	0710_00244D	10.55	Irregular	1	1.7	0.6	0.014
BAILIS TRIBUTARY - BOYNE	991	0710_00210D	2.23	Irregular	1.3	3.2	N/A	0.013
BAILIS TRIBUTARY - BOYNE	1218	0710_00188D	14.1	Irregular	1.7	2.5	N/A	0.014
BAILIS TRIBUTARY - BOYNE	1708	0710_00139D	13.04	Arch	1	1.8	0.6	0.014
BAILIS TRIBUTARY - BOYNE	1939	0710_00116D	10.52	Rectangular	1.3	1.3	N/A	0.014
BAILIS TRIBUTARY - BOYNE	1976	0710_00112D	6.75	Arch	1	1.9	0.6	0.013
BAILIS TRIBUTARY - BOYNE	1988	0710_00111D	2.75	Rectangular	1.2	1.9	N/A	0.013
BAILIS TRIBUTARY - BOYNE	2022	0710_00107D	5.36	Irregular	0.8	1.6	N/A	0.013
BAILIS TRIBUTARY - BOYNE	2075	0710_00102D	6.7	Irregular	1	1.5	0.8	0.013
BAILIS TRIBUTARY - BOYNE	2235	0710_00086D	13.51	Irregular	1	5	N/A	0.013
BAILIS TRIBUTARY - BOYNE	2519	0710_00057D	4.64	Irregular	1.4	1.6	0.7	0.014
BAILIS TRIBUTARY - BOYNE	2593	0710_00050I	4.62	Irregular	1.6	1.8	N/A	0.013
BAILIS TRIBUTARY - BOYNE	2721	0710_00039D	20.66	Arch	2	1.8	1	0.014
BAILIS TRIBUTARY - BOYNE	2871	0710_00022D	12.1	Arch	1.3	1.5	0.8	0.014
RIVER BOYNE	56452	0701_04086D	9.3	6 Openings	4.2	10	N/A	0.014
RIVER BOYNE	58903	0701_04561D	10.7	6 Openings	5	8	4	0.014
RIVER BOYNE	59060	0701_04548D	29.98	Triple Arch	8	48	6	0.014
RIVER BOYNE	62370	0701_04215D	15.27	5 Openings	6	17	N/A	0.014
RIVER BOYNE	62525	0701_04199D	7.1	11 Openings	4.5	6.5	2	0.014
RIVER BOYNE	63812	0701_04070D	18.14	Triple Opening	6.5	33	N/A	0.014
RIVER BOYNE	64829	0701_03969D	9.07	4 Arches	13	15	8	0.014
RIVER BOYNE	71934	0701_03258D	6.87	6 Arches	7	9	3.5	0.014
RIVER BLACKWATER (KELLS)	581	0712_00397D	1.56	3 Openings	4	10	N/A	0.013
RIVER BLACKWATER (KELLS)	2894	0712_00166D	5.1	Rectangular	9	24	N/A	0.013
RIVER BLACKWATER (KELLS)	4065	0712_00050D	16.9	Triple Arch	9	33	8	0.013
RIVER BLACKWATER (KELLS)	4485.028	0712_00007D	10.79	6 Arches	3.5	3.8	2	0.013
TRIM ROAD TRIBUTARY 2 - BOYNE	768.51	0742_00008I	82.19	Single Arch	1.2	1	0.9	0.013
TRIM ROAD TRIBUTARY 1 - BOYNE*	1361	0714_00198	410	Circular	1.2	N/A	N/A	0.013
ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE*	1135	0716_00057I	100	Circular	1.1	N/A	N/A	0.013
ATHLUMNEY HOUSE TRIBUTARY 1 - BOYNE*	1455	0716_00024	50	Circular	1.5	N/A	N/A	0.013
ATHLUMNEY HOUSE TRIBUTARY 4 - BOYNE*	382	0719_00096	548	Circular	1.2	N/A	N/A	0.013
OLD BALREASK TRIBUTARY - BOYNE*	759	0715_00014	50	Circular	1.6	N/A	N/A	0.013

* Denotes structures incorporated as closed cross-sections only (and are therefore not included in the Network file). All dimensions are approximate.

Structure ID Key:

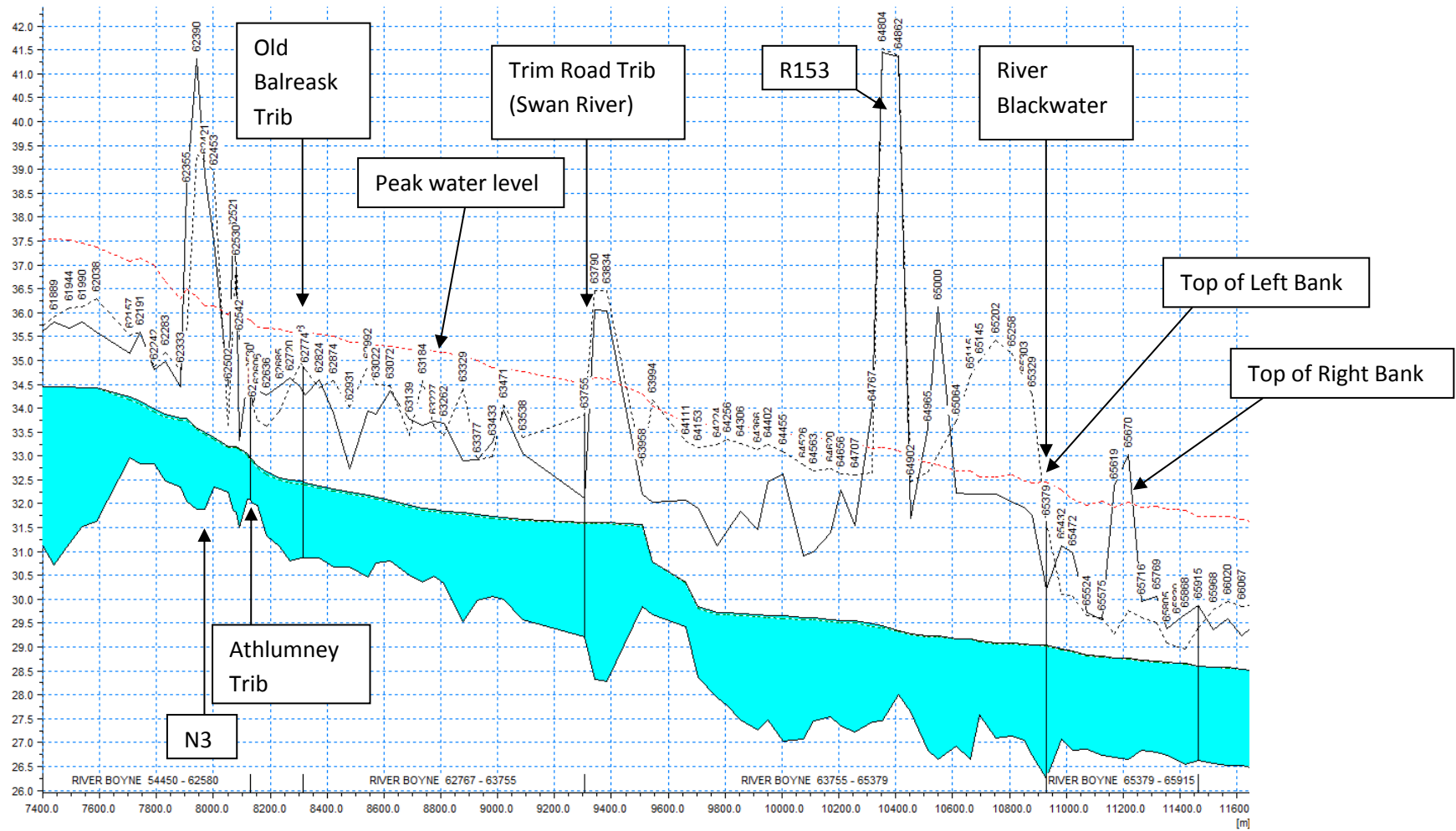
D – Bridge Upstream Face E – Bridge Downstream Facel – Culvert Upstream Face

J – Culvert Downstream Face

Structure Details - Weirs:			
RIVER BRANCH	CHAINAGE	ID	Type
RIVER BLACKWATER (KELLS)	4107.949	0712_00044W	Broad Crested Weir
RIVER BOYNE	70761.66	0701_03374W	Broad Crested Weir
RIVER BOYNE	56780.65	0701_4777W	Broad Crested Weir
RIVER BOYNE	56875.7	0701_4764W	Broad Crested Weir
RIVER BOYNE	56967.1	0701_4756W	Broad Crested Weir
RIVER BOYNE	57165.48	0701_4736W	Broad Crested Weir
RIVER BOYNE	57267.5	0701_4725W	Broad Crested Weir
RIVER BOYNE	57451.31	0701_4707W	Broad Crested Weir
RIVER BOYNE	58418.99	0701_4608W	Broad Crested Weir
RIVER BOYNE	59165.8	0701_4533W	Broad Crested Weir
RIVER BOYNE	59476.92	0701_4503W	Broad Crested Weir
RIVER BOYNE	59520.4	0701_4499W	Broad Crested Weir
RIVER BOYNE	60377.25	0701_4414W	Broad Crested Weir
RIVER BOYNE	63972.33	0701_4050W	Broad Crested Weir

* Note that all of the other weirs in the Network file are overtopping weirs which form part of a composite structure with the culvert / bridge at the corresponding chainage.

River Long Section Profiles



Long section of the River Boyne for the 0.1% AEP event

Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_NAVN6_MF_DES_14_Q2	HA07_NAVN6_M21_DES_14_Q2		
HA07_NAVN6_MF_DES_14_Q5	HA07_NAVN6_M21_DES_14_Q5		
HA07_NAVN6_MF_DES_14_Q10	HA07_NAVN6_M21_DES_14_Q10		
HA07_NAVN6_MF_DES_14_Q20	HA07_NAVN6_M21_DES_14_Q20		
HA07_NAVN6_MF_DES_14_Q50	HA07_NAVN6_M21_DES_14_Q50		
HA07_NAVN6_MF_DES_14_Q100	HA07_NAVN6_M21_DES_14_Q100		
HA07_NAVN6_MF_DES_14_Q200	HA07_NAVN6_M21_DES_14_Q200		
HA07_NAVN6_MF_DES_14_Q1000	HA07_NAVN6_M21_DES_14_Q1000		
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HA07_NAVN6_MF_DES_14_Q5_MRFS	HA07_NAVN6_M21_DES_14_Q5_MRFS		
HA07_NAVN6_MF_DES_14_Q10_MRFS	HA07_NAVN6_M21_DES_14_Q10_MRFS		
HA07_NAVN6_MF_DES_14_Q20_MRFS	HA07_NAVN6_M21_DES_14_Q20_MRFS		
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MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
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HA07_NAVN6_M11_DES_14_Q5	HA07_NAVN6_NWK_DES_14_FS2	HA07_NAVN6_XNS_DES_14_FS2	HA07_NAVN6_BND_DES_4_Q5
HA07_NAVN6_M11_DES_14_Q10	HA07_NAVN6_NWK_DES_14_FS2_4	HA07_NAVN6_XNS_DES_14_FS2_4	HA07_NAVN6_BND_DES_4_Q10
HA07_NAVN6_M11_DES_14_Q20	HA07_NAVN6_NWK_DES_14_FS3	HA07_NAVN6_XNS_DES_14_FS3	HA07_NAVN6_BND_DES_4_Q20
HA07_NAVN6_M11_DES_14_Q50	HA07_NAVN6_NWK_DES_14_FS1_undef	HA07_NAVN6_XNS_DES_14_FS1_undef	HA07_NAVN6_BND_DES_4_Q50
HA07_NAVN6_M11_DES_14_Q100	HA07_NAVN6_NWK_DES_14_FS2_4_undef	HA07_NAVN6_XNS_DES_14_FS2_4_undef	HA07_NAVN6_BND_DES_4_Q100
HA07_NAVN6_M11_DES_14_Q200	HA07_NAVN6_NWK_DES_14_FS2_undef	HA07_NAVN6_XNS_DES_14_FS2_undef	HA07_NAVN6_BND_DES_4_Q200
HA07_NAVN6_M11_DES_14_Q1000			HA07_NAVN6_BND_DES_4_Q1000
HA07_NAVN6_M11_DES_14_Q2_MRFS			HA07_NAVN6_BND_DES_4_Q2_MRFS
HA07_NAVN6_M11_DES_14_Q5_MRFS			HA07_NAVN6_BND_DES_4_Q5_MRFS
HA07_NAVN6_M11_DES_14_Q10_MRFS			HA07_NAVN6_BND_DES_4_Q10_MRFS
HA07_NAVN6_M11_DES_14_Q20_MRFS			HA07_NAVN6_BND_DES_4_Q20_MRFS
HA07_NAVN6_M11_DES_14_Q50_MRFS			HA07_NAVN6_BND_DES_4_Q50_MRFS
HA07_NAVN6_M11_DES_14_Q100_MRFS			HA07_NAVN6_BND_DES_4_Q100_MRFS
HA07_NAVN6_M11_DES_14_Q200_MRFS			HA07_NAVN6_BND_DES_4_Q200_MRFS
HA07_NAVN6_M11_DES_14_Q1000_MRFS			HA07_NAVN6_BND_DES_4_Q1000_MRFS
HA07_NAVN6_M11_DES_14_Q10_HEFS			HA07_NAVN6_BND_DES_4_Q10_HEFS
HA07_NAVN6_M11_DES_14_Q100_HEFS			HA07_NAVN6_BND_DES_4_Q100_HEFS
HA07_NAVN6_M11_DES_14_Q1000_HEFS			HA07_NAVN6_BND_DES_4_Q1000_HEFS
HA07_NAVN6_M11_DES_14_Q10_undef			
HA07_NAVN6_M11_DES_14_Q100_undef			
HA07_NAVN6_M11_DES_14_Q1000_undef			
HA07_NAVN6_M11_DES_14_Q100_MRFS_undef			
HA07_NAVN6_M11_DES_14_Q1000_MRFS_undef			

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_NAVN6_DFS0_Q2	HA07_NAVN6_HD_DES_14_Q2	HA07_NAVN6_M11_DES_14_Q2	HA07_NAVN6_M11_DES_13_Q2_MAPS
HA07_NAVN6_DFS0_Q5	HA07_NAVN6_HD_DES_14_Q5	HA07_NAVN6_M11_DES_14_Q5	HA07_NAVN6_M11_DES_13_Q5_MAPS
HA07_NAVN6_DFS0_Q10	HA07_NAVN6_HD_DES_14_Q10	HA07_NAVN6_M11_DES_14_Q10	HA07_NAVN6_M11_DES_13_Q10_MAPS
HA07_NAVN6_DFS0_Q20	HA07_NAVN6_HD_DES_14_Q20	HA07_NAVN6_M11_DES_14_Q20	HA07_NAVN6_M11_DES_13_Q20_MAPS
HA07_NAVN6_DFS0_Q50	HA07_NAVN6_HD_DES_14_Q50	HA07_NAVN6_M11_DES_14_Q50	HA07_NAVN6_M11_DES_13_Q50_MAPS
HA07_NAVN6_DFS0_Q100	HA07_NAVN6_HD_DES_14_Q100	HA07_NAVN6_M11_DES_14_Q100	HA07_NAVN6_M11_DES_13_Q100_MAPS
HA07_NAVN6_DFS0_Q200	HA07_NAVN6_HD_DES_14_Q200	HA07_NAVN6_M11_DES_14_Q200	HA07_NAVN6_M11_DES_13_Q200_MAPS
HA07_NAVN6_DFS0_Q1000	HA07_NAVN6_HD_DES_14_Q1000	HA07_NAVN6_M11_DES_14_Q1000	HA07_NAVN6_M11_DES_13_Q1000_MAPS
HA07_NAVN6_DFS0_Q2_MRFS	HA07_NAVN6_HD_DES_14_Q2_MRFS	HA07_NAVN6_M11_DES_14_Q2_MRFS	HA07_NAVN6_M11_DES_13_Q2_MRFS_MAPS
HA07_NAVN6_DFS0_Q5_MRFS	HA07_NAVN6_HD_DES_14_Q5_MRFS	HA07_NAVN6_M11_DES_14_Q5_MRFS	HA07_NAVN6_M11_DES_13_Q5_MRFS_MAPS
HA07_NAVN6_DFS0_Q10_MRFS	HA07_NAVN6_HD_DES_14_Q10_MRFS	HA07_NAVN6_M11_DES_14_Q10_MRFS	HA07_NAVN6_M11_DES_13_Q10_MRFS_MAPS
HA07_NAVN6_DFS0_Q20_MRFS	HA07_NAVN6_HD_DES_14_Q20_MRFS	HA07_NAVN6_M11_DES_14_Q20_MRFS	HA07_NAVN6_M11_DES_13_Q20_MRFS_MAPS
HA07_NAVN6_DFS0_Q50_MRFS	HA07_NAVN6_HD_DES_14_Q50_MRFS	HA07_NAVN6_M11_DES_14_Q50_MRFS	HA07_NAVN6_M11_DES_13_Q50_MRFS_MAPS
HA07_NAVN6_DFS0_Q100_MRFS	HA07_NAVN6_HD_DES_14_Q100_MRFS	HA07_NAVN6_M11_DES_14_Q100_MRFS	HA07_NAVN6_M11_DES_13_Q100_MRFS_MAPS
HA07_NAVN6_DFS0_Q200_MRFS	HA07_NAVN6_HD_DES_14_Q200_MRFS	HA07_NAVN6_M11_DES_14_Q200_MRFS	HA07_NAVN6_M11_DES_13_Q200_MRFS_MAPS
HA07_NAVN6_DFS0_Q1000_MRFS	HA07_NAVN6_HD_DES_14_Q1000_MRFS	HA07_NAVN6_M11_DES_14_Q1000_MRFS	HA07_NAVN6_M11_DES_13_Q1000_MRFS_MAPS
HA07_NAVN6_DFS0_Q10_HEFS	HA07_NAVN6_HD_DES_14_Q10_HEFS	HA07_NAVN6_M11_DES_14_Q10_HEFS	HA07_NAVN6_M11_DES_13_Q10_HEFS_MAPS
HA07_NAVN6_DFS0_Q100_HEFS	HA07_NAVN6_HD_DES_14_Q100_HEFS	HA07_NAVN6_M11_DES_14_Q100_HEFS	HA07_NAVN6_M11_DES_13_Q100_HEFS_MAPS
HA07_NAVN6_DFS0_Q1000_HEFS	HA07_NAVN6_HD_DES_14_Q1000_HEFS	HA07_NAVN6_M11_DES_14_Q1000_HEFS	HA07_NAVN6_M11_DES_13_Q1000_HEFS_MAPS
	HA07_NAVN6_HD_DES_14_Q10_undef	HA07_NAVN6_M11_DES_14_Q10_undef	HA07_NAVN6_M21_DES_14_Q10_undef_maxse
	HA07_NAVN6_HD_DES_14_Q100_undef	HA07_NAVN6_M11_DES_14_Q100_undef	HA07_NAVN6_M21_DES_14_Q100_undef_maxse
	HA07_NAVN6_HD_DES_14_Q1000_undef	HA07_NAVN6_M11_DES_14_Q1000_undef	HA07_NAVN6_M21_DES_14_Q1000_undef_maxse
	HA07_NAVN6_HD_DES_14_Q100_MRFS_undef	HA07_NAVN6_M11_DES_14_Q100_MRFS_undef	HA07_NAVN6_M21_DES_14_Q100_MRFS_undef_maxse
	HA07_NAVN6_HD_DES_14_Q1000_MRFS_undef	HA07_NAVN6_M11_DES_14_Q1000_MRFS_undef	HA07_NAVN6_M21_DES_14_Q1000_MRFS_undef_maxse

Model Files - Sensitivity

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_NAVN6_MF_SEN_1_Q100_bld HA07_NAVN6_MF_SEN_1_Q100_HEFS_flow HA07_NAVN6_MF_SEN_Q100_hl_1 HA07_NAVN6_MF_SEN_Q100_hl_2 HA07_NAVN6_MF_SEN_Q100_hl_3 HA07_NAVN6_MF_SEN_Q100_rough	HA07_NAVN6_M21_SEN_1_Q100_bld HA07_NAVN6_M21_SEN_1_Q100_HEFS_flow HA07_NAVN6_M21_SEN_Q100_hl_1 HA07_NAVN6_M21_SEN_Q100_hl_2 HA07_NAVN6_M21_SEN_Q100_hl_3 HA07_NAVN6_M21_SEN_Q100_rough HA07_NAVN6_DFS2_MESH6 HA07_NAVN6_DFS2_MESH6_3 HA07_NAVN6_DFS2_MESH6_SEN_bld HA07_NAVN6_Mannings_M_1 HA07_NAVN6_Mannings_M_1_rough HA07_NAVN6_Mannings_M_1_SEN_bld		HA07_NAVN6_M21_SEN_1_Q100_HEFS_flow HA07_NAVN6_M21_SEN_Q100_bld HA07_NAVN6_M21_SEN_Q100_hl_1 HA07_NAVN6_M21_SEN_Q100_hl_2 HA07_NAVN6_M21_SEN_Q100_hl_3 HA07_NAVN6_M21_SEN_Q100_rough

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_NAVN6_M11_SEN_1_Q100_bld HA07_NAVN6_M11_SEN_1_Q100_HEFS_flow HA07_NAVN6_M11_SEN_Q100_hl_1 HA07_NAVN6_M11_SEN_Q100_hl_2 HA07_NAVN6_M11_SEN_Q100_hl_3 HA07_NAVN6_M11_SEN_Q100_rough	HA07_NAVN6_NWK_DES_14_FS1 HA07_NAVN6_NWK_DES_14_FS2_4 HA07_NAVN6_NWK_SEN_hl_2 HA07_NAVN6_NWK_SEN_hl_3	HA07_NAVN6_XNS_DES_14_FS1 HA07_NAVN6_XNS_DES_14_FS2_4 HA07_NAVN6_XNS_SEN_FS1_rough HA07_NAVN6_XNS_SEN_hl_1	HA07_NAVN6_BND_DES_4_Q100 HA07_NAVN6_BND_SEN_1_Q100_flow

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_NAVN6_DFS0_Q100 HA07_NAVN6_DFS0_SEN_1_Q100_flow	HA07_NAVN6_HD_DES_14_Q100 HA07_NAVN6_HD_DES_14_Q100_HEFS HA07_NAVN6_HD_SEN_hl HA07_NAVN6_HD_SEN_Q100_bld	HA07_NAVN6_M11_SEN_1_Q100_HEFS_flow HA07_NAVN6_M11_SEN_Q100_bld HA07_NAVN6_M11_SEN_Q100_hl_1 HA07_NAVN6_M11_SEN_Q100_hl_2 HA07_NAVN6_M11_SEN_Q100_hl_3 HA07_NAVN6_M11_SEN_Q100_rough	N/A

GIS Deliverables – Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u> E31EXFCD500F0 E31EXFCD200F0 E31EXFCD100F0 E31EXFCD050F0 E31EXFCD020F0 E31EXFCD010F0 E31EXFCD005F0 E31EXFCD001F0	Water Level and Flows <u>Fluvial</u> E31NFCDF0 E31NFMDF0 E31NFHDF0 Flood Defences (Shapefiles) N/A Defended Areas (Shapefiles) N/A Wave Overtopping (Shapefiles) N/A	<u>Fluvial</u> E31DPFCD500F0 E31DPFCD200F0 E31DPFCD100F0 E31DPFCD050F0 E31DPFCD020F0 E31DPFCD010F0 E31DPFCD005F0 E31DPFCD001F0	<u>Fluvial</u> E31VLFCDF0500F0 E31VLFCDF0200F0 E31VLFCDF0100F0 E31VLFCDF050F0 E31VLFCDF020F0 E31VLFCDF010F0 E31VLFCDF005F0 E31VLFCDF001F0	<u>Fluvial</u> E31RLFCD100F0 E31RLFCD010F0 E31RLFCD001F0
				Flood Zones (Shapefiles)
E31EXFMD500F0 E31EXFMD200F0 E31EXFMD100F0 E31EXFMD050F0 E31EXFMD020F0 E31EXFMD010F0 E31EXFMD005F0 E31EXFMD001F0		E31DPFMD500F0 E31DPFMD200F0 E31DPFMD100F0 E31DPFMD050F0 E31DPFMD020F0 E31DPFMD010F0 E31DPFMD005F0 E31DPFMD001F0		E31ZNA_FCDF0 E31ZNB_FCDF0
E31EXFHD100F0 E31EXFHD010F0 E31EXFHD001F0		E31DPFHD100F0 E31DPFHD010F0 E31DPFHD001F0		E31ZNA_FMDF0 E31ZNB_FMDF0
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables - Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E31RIFCD100F0 E31RIFCD010F0 E31RIFCD001F0 E31RIFMD100F0 E31RIFMD010F0 E31RIFMD001F0	<u>Fluvial</u> E07_RTFC001_F0 E07_RTMD001_F0	<u>Fluvial</u> E31RDFCD001F0 E31RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A

Appendix I

Trim AFA

Additional Information

List of background information included:

- 1D Structures Modelled in the 1D Domain
- 1D Structures Modelled in the 2D Domain
- River Long Section Profiles
- Final Model Files – Design
- Final Model Files – Sensitivity
- GIS Deliverables – Hazard
- GIS Deliverables – Risk

1D Structures

Structure Details – Bridges & Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING'S N
Bridges								
Trim Tributary 1 – Boyne	1933.121	0720_00036D_bridge	12.36	Arch x 2	2.442, 2.293	1.6, 1.7	1.911, 1.783	0.013
Trim Tributary 1 – Boyne	1969.947	0720_00032D_bridge	24.12	Irregular	2.196	2.8	0.174	0.021
Trim Tributary 1 – Boyne	2111.26	0720_00017D_bridge	3.34	Irregular	2.388	2.4	N/A	0.013
Trim Tributary 3 – Boyne	73.647	0722_00299D_bridge	3.85	Arch	3.428	3	1.648	0.013
Trim Tributary 3 – Boyne	269.715	0722_00280D_bridge	4.72	Irregular	2.761	6.191	N/A	0.013
Trim Tributary 3 – Boyne	2610.565	0722_00045D_bridge	9.25	Irregular x 2	2.797, 2.826	1.9, 2.1	2.107, 2.826	0.014
Trim Tributary 3 – Boyne	2982.128	0722_00008D_bridge	10.73	Irregular x 2	2.773, 2.657	2.36, 1.797	2.031, 2.037	0.021
Blackfriary Tributary – Boyne	404.694	0723_00239D_bridge	9.69	Arch	1.917	1.802	1.387	0.013
Blackfriary Tributary – Boyne	1563.018	0723_00124D_bridge	22.15	Arch	1.498	2	0.798	0.014
Blackfriary Tributary – Boyne	1865.117	0723_00094D_bridge	18.84	Irregular	1.359	2.6	N/A	0.013
Blackfriary Tributary – Boyne	2289.989	0723_00050D_bridge	1.22	Irregular	2.049	6.7	N/A	0.013
Blackfriary Tributary – Boyne	2309.534	0723_00048D_bridge	3.54	Irregular	1.18	2	N/A	0.013
Blackfriary Tributary – Boyne	2514.604	0723_00033D_bridge	1.87	Irregular	1.676	2.8	N/A	0.013
Blackfriary Tributary – Boyne	2751.699	0723_00009D_bridge	3.78	Irregular	0.919	1.7	0.539	0.013
Stonehall Tributary 1 – Boyne	1398.705	0724_00003D_bridge	1.01	Irregular	1.246	3.716	N/A	0.013
Stonehall Tributary 2 – Boyne	1026.17	0725_00066D_bridge	32.11	Arch	1.807	1.1	1.487	0.013
Athboy Road Tributary 1 – Boyne	1428.016	0726_00005D_bridge	1.27	Irregular	1.164	1.9	N/A	0.013
River Boyne	21084.45	0701_08343D_bridge	12.27	Arch x 3	7.387, 10.024, 7.177	12.4, 12.3, 12.2	3.107, 5.374, 2.545	0.013
River Boyne	23577.994	0701_08093D_bridge	5.6	Arch x 3	1.889, 1.769, 6.667	2.7, 2.8, 11.4	0.489, 0.389, 4.615	0.013
River Boyne	28778.35	0701_07573D_bridge	5.84	Arch x 5	Ranging from 1.183-6.775	Ranging from 2.104-15.511	Ranging from 0.993-3.629	0.013

River Boyne	33774.508	0701_07073E_bridge	6.66	Arch x 5	Ranging from 1.92-7.558	Ranging from 2.5-18.78	Ranging from 0.835-4.283	0.013
River Boyne	45303.876	0701_05921D_bridge	2.22	Irregular	4.396	35.6	N/A	0.014
River Boyne	45387.559	0701_05913D_bridge	15.3	Irregular	4.608	30.8	N/A	0.014
River Boyne	45528.548	0701_05898D_bridge	7	Arch x 4	Ranging from 4.796-5.052	Ranging from 4.683-5.7	Ranging from 2.616-2.872	0.013
River Boyne	45662.687	0701_05885D_bridge	4.73	Arch	4.175	28.3	3.475	0.014
River Boyne	46038.489	0701_05848D_bridge	13.54	Irregular x 3	4.875, 4.606, 4.722	9.8, 13.9, 9.9	N/A	0.013
River Boyne	47255.772	0701_05726D_bridge	5.07	Arch x 5	Ranging from 4.417-4.758	Ranging from 4.861-6.06	Ranging from 2.312-3.437	0.013
River Boyne	53721.656	0701_05079D_bridge	6.3	Arch x 12	Ranging from 1.014-4.704	Ranging from 1.95-4.29	Ranging from 0.334-3.079	0.013
River Boyne	56451.917	0701_04806D_bridge	9.37	Arch x 6	Ranging from 5.905-9.235	Ranging from 6.1-10.8	Ranging from 2.464-4.718	0.013
River Boyne	58902.713	0701_04561D_bridge	10.7	Arch x 6	Ranging from 3.998-6.878	Ranging from 6.7-8	Ranging from 1.428-4.418	0.013
River Boyne	38514.806	0701_06598E_bridge	6.78	Arch x 5	Ranging from 3.735-6.212	Ranging from 3.8-11.191	Ranging from 1.929-4.339	0.013
River Boyne	20198.033	0701_08431D_bridge	6.39	Arch x 6	Ranging from 2.171-5.837	Ranging from 3.174-8.7	Ranging from 0.34-3.173	0.013
River Boyne	21033.875	0701_08347D_bridge	7.8	Arch x 3	6.232, 9.912, 5.921	12.1, 11.7, 12.2	2.504, 6.532, 2.311	0.013

Culverts								
RIVER BRANCH	CHAINAGE	ID	LENGTH (m)	OPENING SHAPE	HEIGHT (m)	WIDTH (m)	SPRING HEIGHT FROM INVERT (m)	MANNING'S N
Trim Tributary 2 – Boyne	595.706	0721_00031l_culvert	2.6	Circular	0.9	N/A	N/A	0.013
Trim Tributary 2 – Boyne	609.781	0721_00030l_culvert	16.42	Circular	0.9	N/A	N/A	0.013
Trim Tributary 3 – Boyne	2576.612	0722_00050l_culvert	5.43	Irregular	2.621	3	N/A	0.013
Trim Tributary 3 – Boyne	2953.798	0722_00012l_culvert	31.1	Arch	2.881	4.8	0.672	0.021
Blackfriary Tributary – Boyne	1915.211	0723_00088l_culvert	10.2	Circular	2.4	N/A	N/A	0.013
Blackfriary Tributary – Boyne	2415.516	0723_00045l_culvert	16.99	Irregular	1.361	3	N/A	0.014
Stonehall Tributary 1 – Boyne	625.68	0724_00080l_culvert	4.56	Circular	0.75	N/A	N/A	0.013
Stonehall Tributary 1 – Boyne	1337.759	0724_00010l_culvert	25.18	Circular	1.1	N/A	N/A	0.013
Stonehall Tributary 2 – Boyne	558.296	0725_00116l_culvert	74.29	Circular	1.5	N/A	N/A	0.013
Stonehall Tributary 2 – Boyne	665.702	0725_00102l_culvert	3.7	Circular	1.5	N/A	N/A	0.013
Stonehall Tributary 2 – Boyne	720.709	0725_00098l_culvert	37.37	Circular	1.5	N/A	N/A	0.013
Stonehall Tributary 2 – Boyne	790.659	0725_00090l_culvert	16.49	Circular	1.4	N/A	N/A	0.013
Stonehall Tributary 2 – Boyne	874.513	0725_00082l_culvert	31.22	Circular	1.4	N/A	N/A	0.014
Stonehall Tributary 2 – Boyne	1607.353	0725_00009l_culvert	33.41	Circular	0.85	N/A	N/A	0.014

Athboy Road Tributary 1 – Boyne	718.835	0726_000761_culvert	3.89	Circular	0.8	N/A	N/A	0.014
Athboy Road Tributary 1 – Boyne	746.908	0726_000731_culvert	4.89	Circular	1.6	N/A	N/A	0.014
Athboy Road Tributary 1 – Boyne	959.624	0726_000521_culvert	31.64	Circular	1.5	N/A	N/A	0.014
Athboy Road Tributary 1 – Boyne	1060.754	0726_000421_culvert	17.41	Circular	1.5	N/A	N/A	0.014
Athboy Road Tributary 1 – Boyne	1096.479	0726_000381_culvert	11.17	Circular	1.5	N/A	N/A	0.013
Athboy Road Tributary 2 – Boyne	149.148	0727_000581_culvert	4.68	Circular	1	N/A	N/A	0.014
Athboy Road Tributary 2 – Boyne	223.694	0727_000511_culvert	4.05	Circular	0.9	N/A	N/A	0.014
Athboy Road Tributary 2 – Boyne	404.817	0727_000321_culvert	4.47	Circular	0.9	N/A	N/A	0.013
Athboy Road Tributary 2 – Boyne*	641.04	0727_000171	224.6 (approx.)	Circular x 2, ending circular x 1	1, 1.6	N/A	N/A	0.014

*Denotes structures incorporated as closed cross-sections only (and are therefore not included in the Network file which is the reason for a discrepancy in the amount of culverts from the hydraulic model).

**Structure created (further information in Section 4.9.6.1).

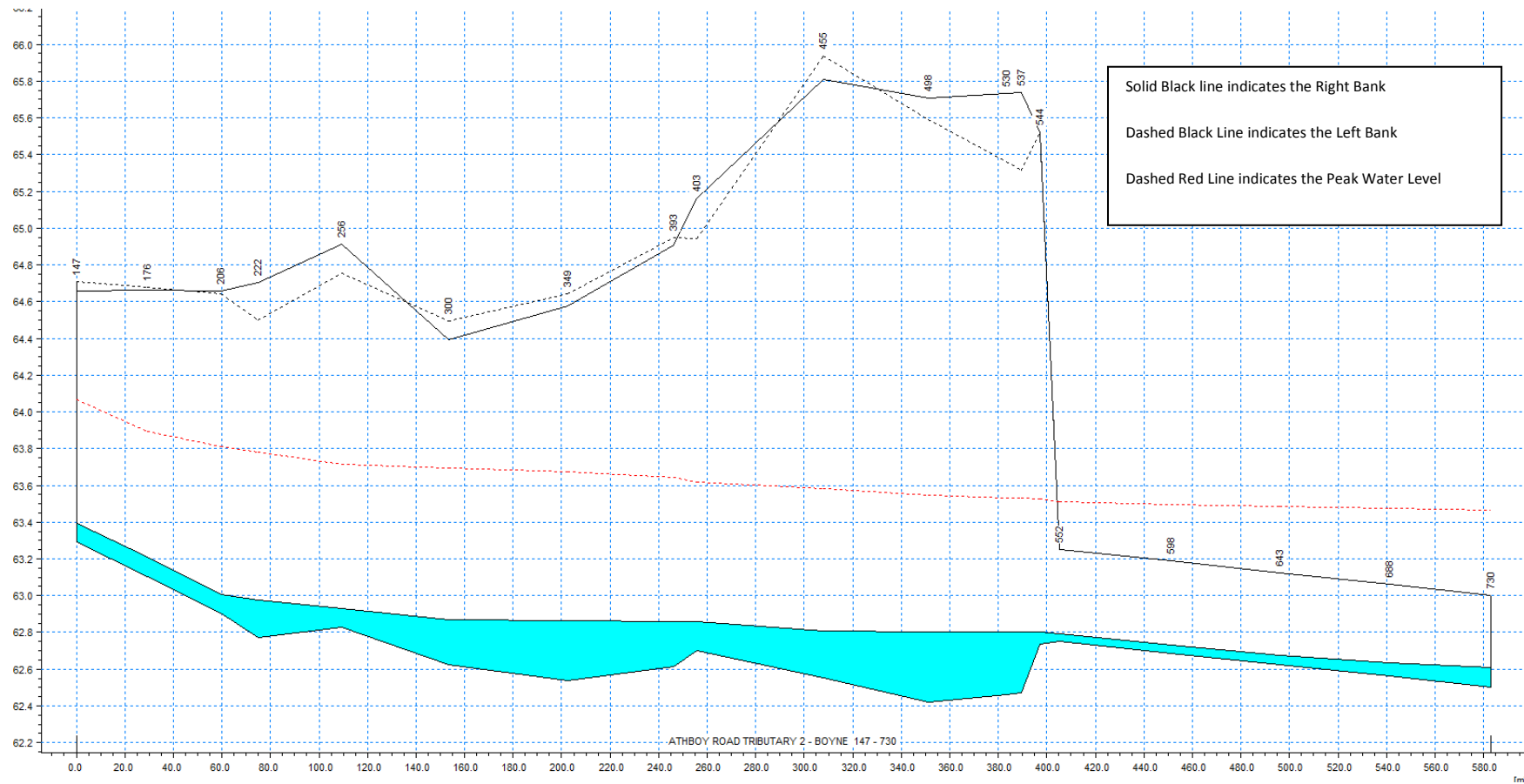
Structure Details - Weirs			
RIVER BRANCH	CHAINAGE	ID	Type
Trim Tributary 1 – Boyne	1306.243	0720_00099W_weir	Broad Crested
Trim Tributary 1 – Boyne	1867.047	0720_00042W_weir	Broad Crested
Trim Tributary 3 – Boyne	1475.313	0722_00160W_weir	Broad Crested
Blackfriary Tributary - Boyne	2764.166	0723_00007W_weir	Broad Crested
River Boyne	50710.259	0701_05380W_weir	Broad Crested
River Boyne	52812.385	0701_05170W_weir	Broad Crested
River Boyne	53088.181	0701_05142W_weir	Broad Crested
River Boyne	53120.113	0701_05139W_weir	Broad Crested
River Boyne	53603.826	0701_05090W_weir	Broad Crested
River Boyne	54855.174	0701_04965W_weir	Broad Crested
River Boyne	54949.811	0701_04955W_weir	Broad Crested
River Boyne	55089.471	0701_04942W_weir	Broad Crested
River Boyne	55578.301	0701_04892W_weir	Broad Crested
River Boyne	56576.191	0701_04796W_weir	Broad Crested
River Boyne	56649.032	0701_04789W_weir	Broad Crested
River Boyne	56705.362	0701_04784W_weir	Broad Crested
River Boyne	56780.65	0701_04777W_weir	Broad Crested
River Boyne	56875.701	0701_04764W_weir	Broad Crested
River Boyne	56967.099	0701_04756W_weir	Broad Crested
River Boyne	57165.483	0701_04736W_weir	Broad Crested
River Boyne	57267.497	0701_04725W_weir	Broad Crested
River Boyne	57451.312	0701_04707W_weir	Broad Crested
River Boyne	58418.991	0701_04608W_weir	Broad Crested

River Boyne	58440.779	0701_04606W_weir	Broad Crested
River Boyne	58482.71	0701_04602W_weir	Broad Crested
River Boyne**	49172.673	0701_05534_acting as a weir	Broad Crested
River Boyne**	48815.958	0701_05570_acting as a weir	Broad Crested
River Boyne**	43940.568	0701_06057_acting as a weir	Broad Crested

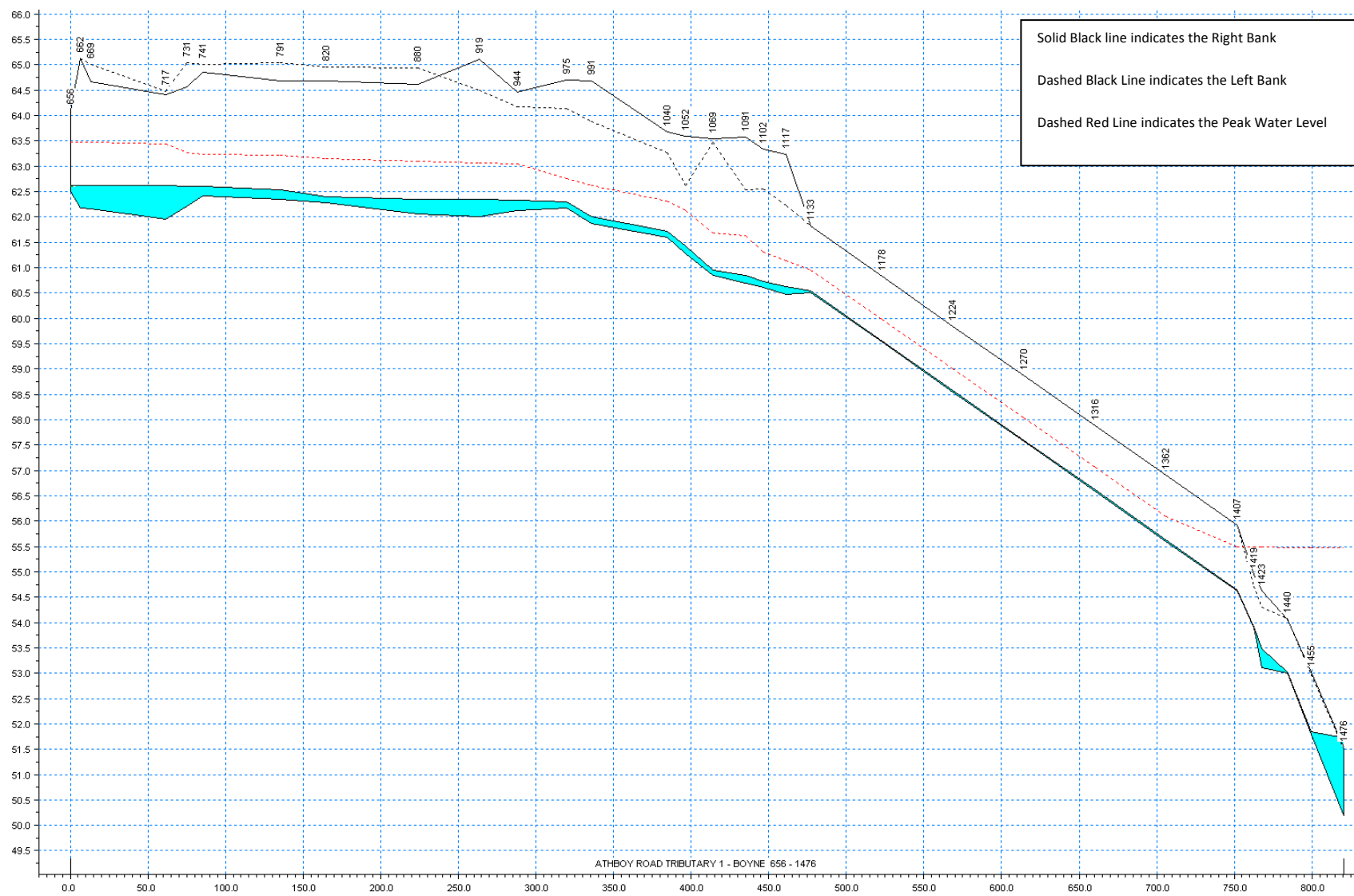
**Structure created (further information in Section 4.9.6.1).

NB: Number of weirs varies from number stated in hydraulic model as this table includes weirs only, therefore excluding all overtopping weirs.

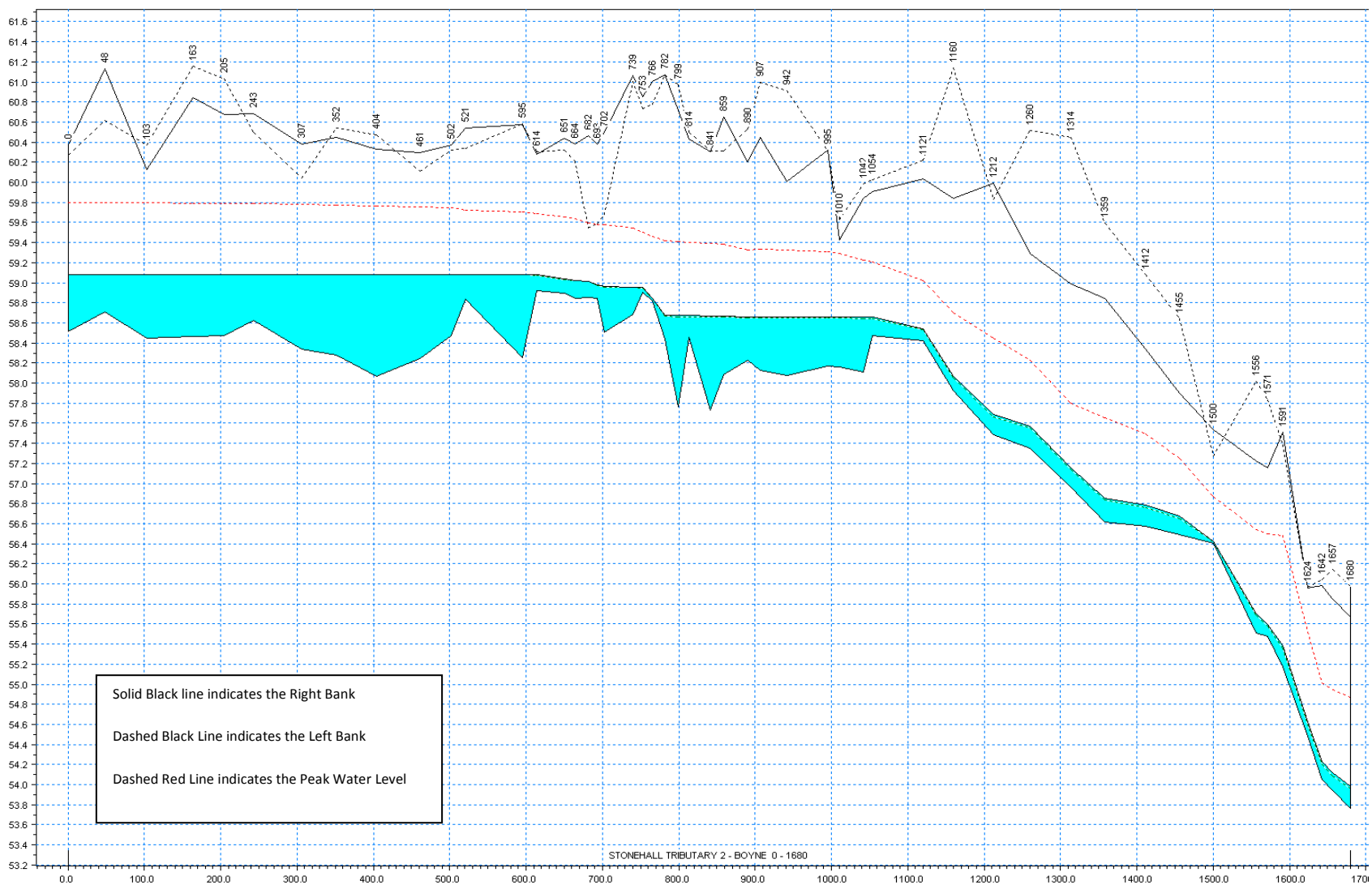
River Long Section Profiles



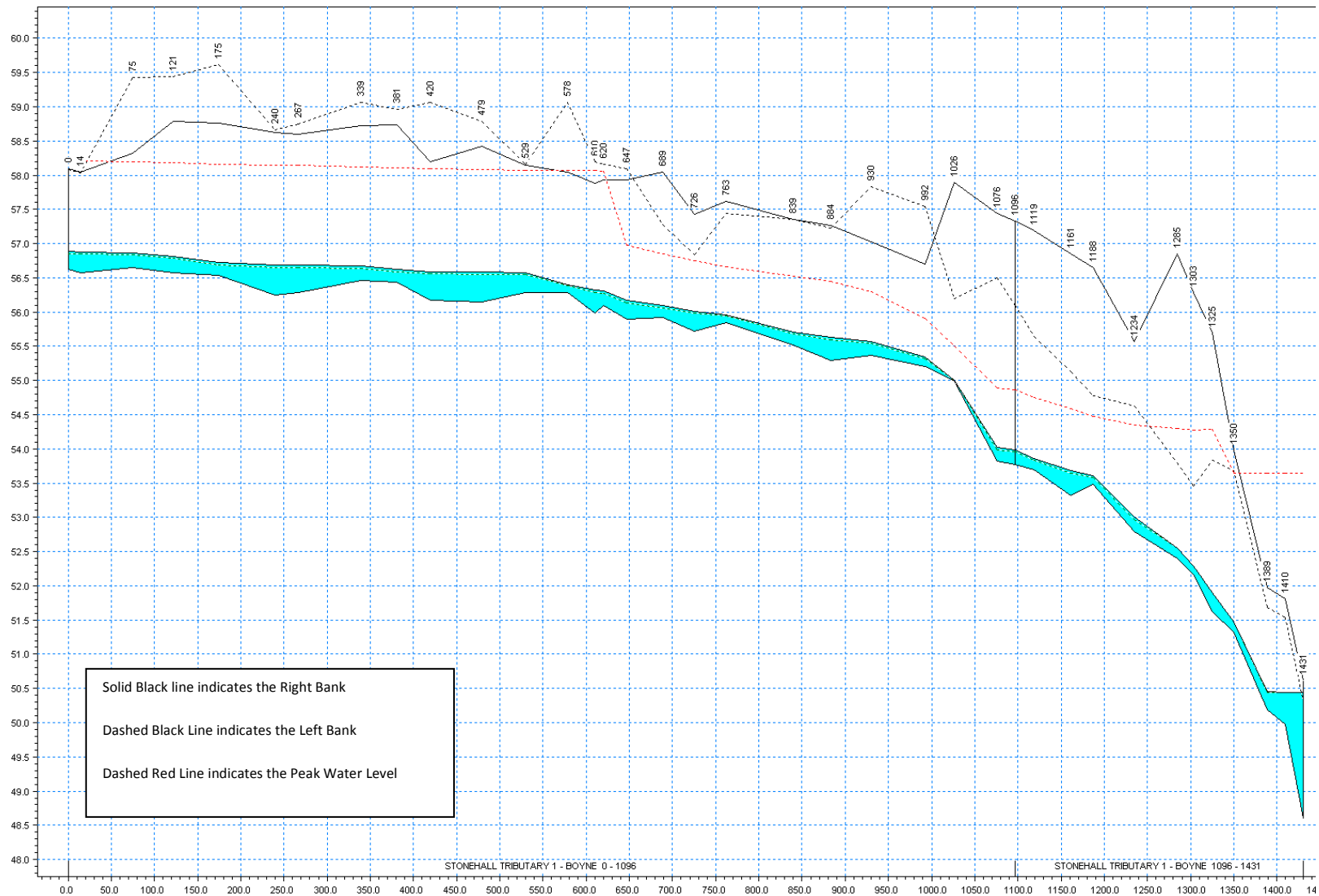
Athboy Road Tributary 2 - Boyne (0727) Watercourse 0.1% AEP fluvial flow



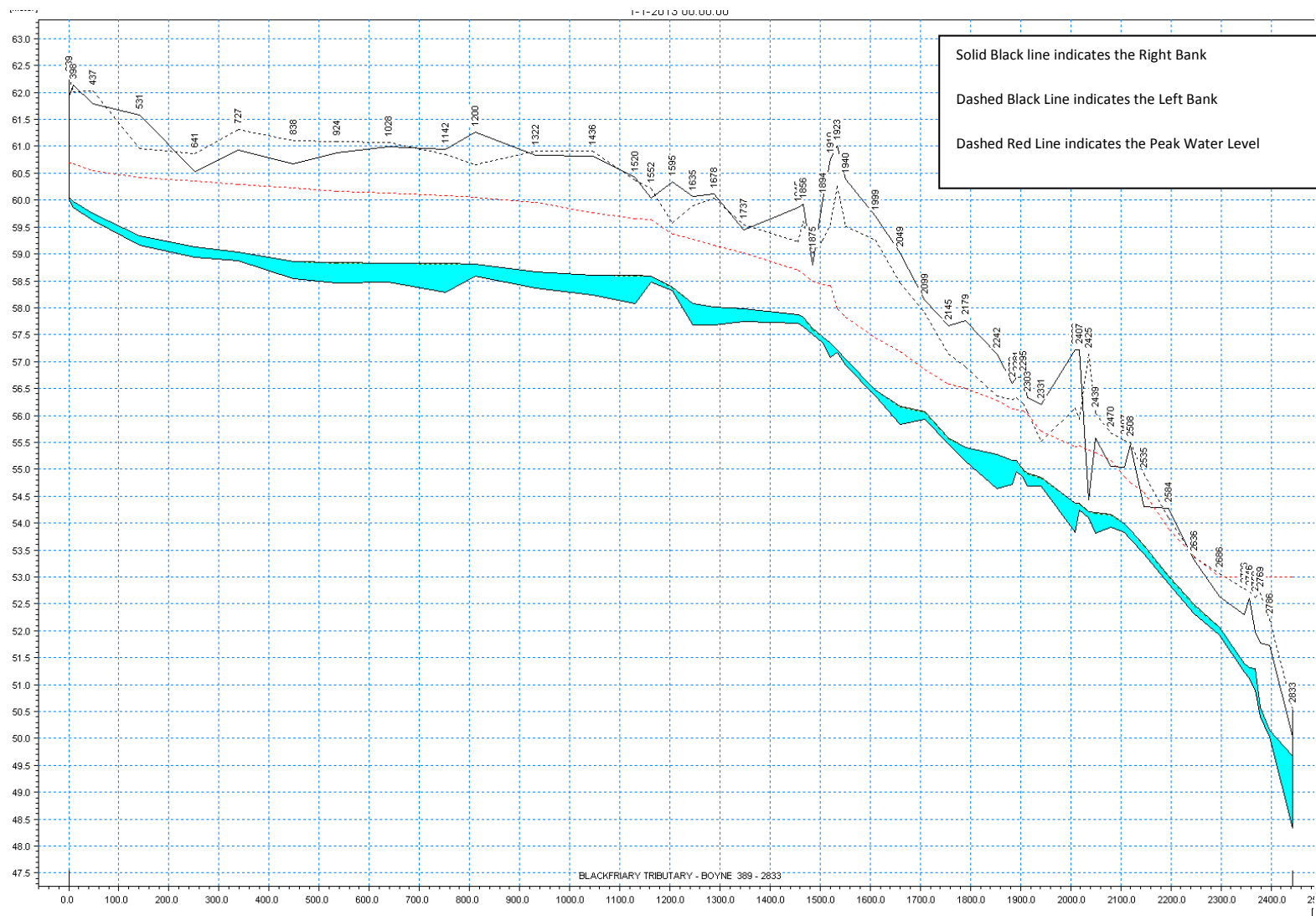
Athboy Road Tributary 1 - Boyne (0726) Watercourse 0.1% AEP fluvial flow



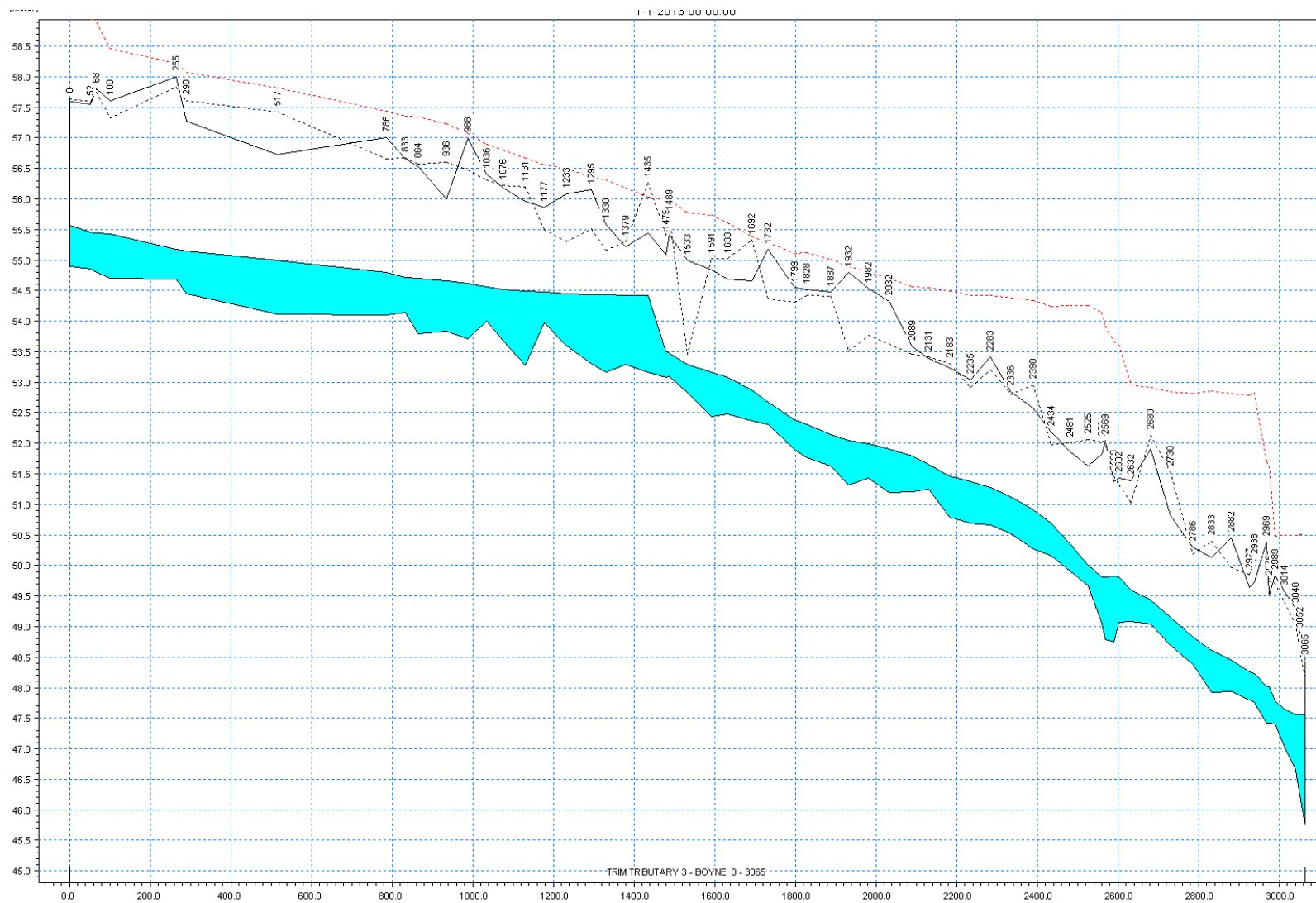
Stonehall Tributary 2 - Boyne (0725) Watercourse 0.1% AEP Fluvial Flow



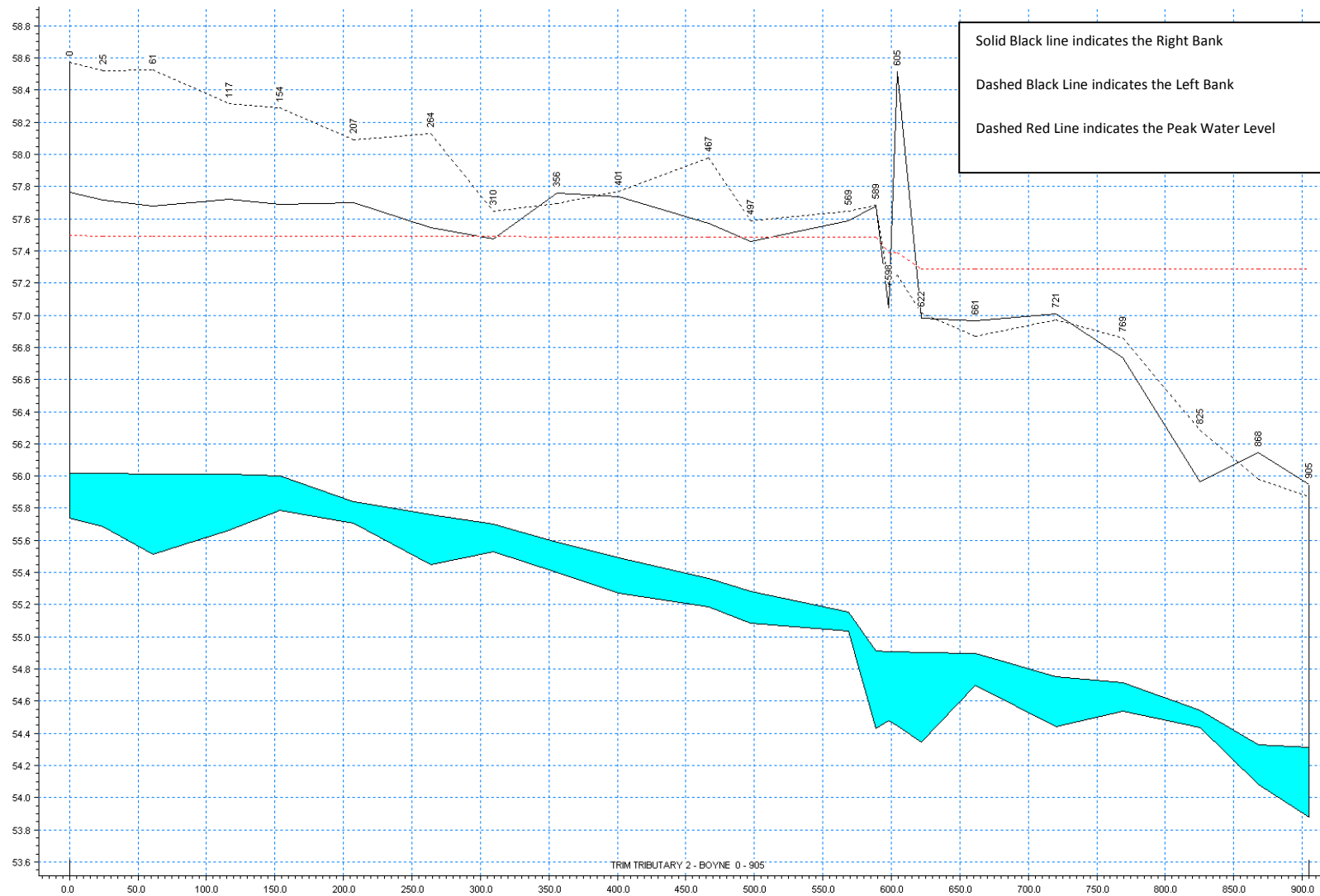
Stonehall Tributary 1 - Boyne (0724) Watercourse 0.1% AEP Fluvial Flow



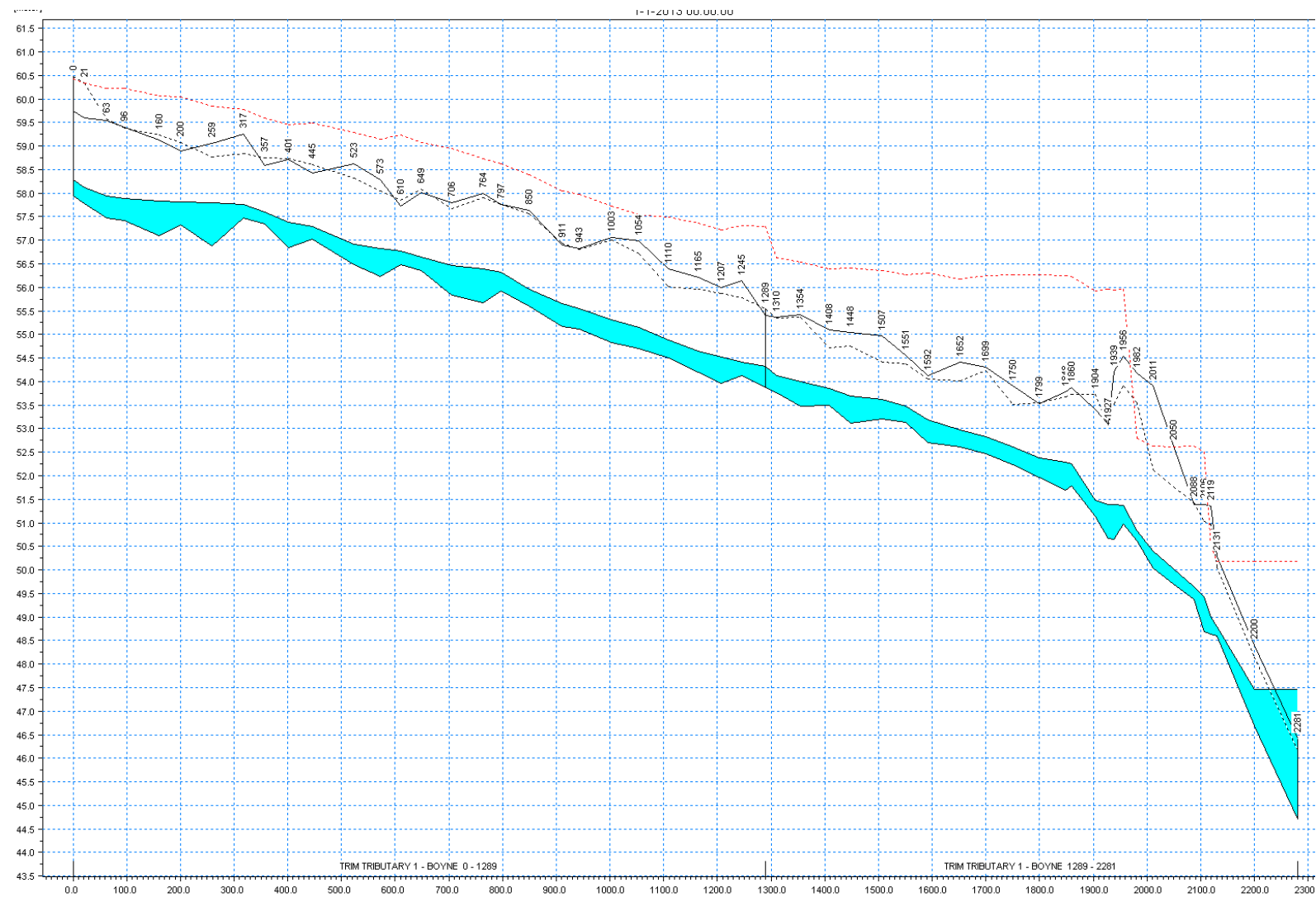
Blackfriary Tributary - Boyne (0723) Watercourse 0.1% AEP Fluvial Flow



Trim Tributary 3 - Boyne (0722) Watercourse 0.1% AEP Fluvial Flow



Trim Tributary 2 - Boyne (0721) Watercourse 0.1% AEP Fluvial Flow



Trim Tributary 1 - Boyne (0720) Watercourse 0.1% AEP Fluvial Flow

Model Files - Design

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_TRIM4_MF_DES_3_Q2	HA07_TRIM4_M21_DES_3_Q2		HA07_TRIM4_MF_DES_3_Q2
HA07_TRIM4_MF_DES_3_Q5	HA07_TRIM4_M21_DES_3_Q5		HA07_TRIM4_MF_DES_3_Q5
HA07_TRIM4_MF_DES_3_Q10	HA07_TRIM4_M21_DES_3_Q10		HA07_TRIM4_MF_DES_3_Q10_1
HA07_TRIM4_MF_DES_3_Q20	HA07_TRIM4_M21_DES_3_Q20		HA07_TRIM4_MF_DES_3_Q20
HA07_TRIM4_MF_DES_3_Q50	HA07_TRIM4_M21_DES_3_Q50		HA07_TRIM4_MF_DES_3_Q50
HA07_TRIM4_MF_DES_3_Q100	HA07_TRIM4_M21_DES_3_Q100		HA07_TRIM4_MF_DES_3_Q100_1
HA07_TRIM4_MF_DES_3_Q200	HA07_TRIM4_M21_DES_3_Q200		HA07_TRIM4_MF_DES_3_Q200
HA07_TRIM4_MF_DES_3_Q1000	HA07_TRIM4_M21_DES_3_Q1000		HA07_TRIM4_MF_DES_3_Q1000_1
HA07_TRIM4_MF_DES_3_Q2_MRFS	HA07_TRIM4_M21_DES_3_Q2_MRFS		HA07_TRIM4_MF_DES_3_Q2_MRFS
HA07_TRIM4_MF_DES_3_Q5_MRFS	HA07_TRIM4_M21_DES_3_Q5_MRFS		HA07_TRIM4_MF_DES_3_Q5_MRFS
HA07_TRIM4_MF_DES_3_Q10_MRFS	HA07_TRIM4_M21_DES_3_Q10_MRFS		HA07_TRIM4_MF_DES_3_Q10_MRFS
HA07_TRIM4_MF_DES_3_Q20_MRFS	HA07_TRIM4_M21_DES_3_Q20_MRFS		HA07_TRIM4_MF_DES_3_Q20_MRFS
HA07_TRIM4_MF_DES_3_Q50_MRFS	HA07_TRIM4_M21_DES_3_Q50_MRFS		HA07_TRIM4_MF_DES_3_Q50_MRFS
HA07_TRIM4_MF_DES_3_Q100_MRFS	HA07_TRIM4_M21_DES_3_Q100_MRFS		HA07_TRIM4_MF_DES_3_Q100_MRFS
HA07_TRIM4_MF_DES_3_Q200_MRFS	HA07_TRIM4_M21_DES_3_Q200_MRFS		HA07_TRIM4_MF_DES_3_Q200_MRFS
HA07_TRIM4_MF_DES_3_Q1000_MRFS	HA07_TRIM4_M21_DES_3_Q1000_MRFS		HA07_TRIM4_MF_DES_3_Q1000_MRFS
HA07_TRIM4_MF_DES_3_Q10_HEFS	HA07_TRIM4_M21_DES_3_Q10_HEFS		HA07_TRIM4_MF_DES_3_Q10_HEFS
HA07_TRIM4_MF_DES_3_Q100_HEFS	HA07_TRIM4_M21_DES_3_Q100_HEFS		HA07_TRIM4_MF_DES_3_Q100_HEFS
HA07_TRIM4_MF_DES_3_Q1000_HEFS	HA07_TRIM4_M21_DES_3_Q1000_HEFS		HA07_TRIM4_MF_DES_3_Q1000_HEFS
	HA07_TRIM4_dtsf2_Rec_MESH_bldgs_blockeddrivers_3 rastert_ha07_trimr		

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_TRIM4_M11_DES_3_Q2 HA07_TRIM4_M11_DES_3_Q5 HA07_TRIM4_M11_DES_3_Q10 HA07_TRIM4_M11_DES_3_Q20 HA07_TRIM4_M11_DES_3_Q50 HA07_TRIM4_M11_DES_3_Q100 HA07_TRIM4_M11_DES_3_Q200 HA07_TRIM4_M11_DES_3_Q1000 HA07_TRIM4_M11_DES_3_Q2_MRFS HA07_TRIM4_M11_DES_3_Q5_MRFS HA07_TRIM4_M11_DES_3_Q10_MRFS HA07_TRIM4_M11_DES_3_Q20_MRFS HA07_TRIM4_M11_DES_3_Q50_MRFS HA07_TRIM4_M11_DES_3_Q100_MRFS HA07_TRIM4_M11_DES_3_Q200_MRFS HA07_TRIM4_M11_DES_3_Q1000_MRFS HA07_TRIM4_M11_DES_3_Q10_HEFS HA07_TRIM4_M11_DES_3_Q100_HEFS HA07_TRIM4_M11_DES_3_Q1000_HEFS	HA07_TRIM4_NWK_DES_1_Irish	HA07_TRIM4_XNS_DES_1	HA07_TRIM4_BND_DES_3_Q2 HA07_TRIM4_BND_DES_3_Q5 HA07_TRIM4_BND_DES_3_Q10 HA07_TRIM4_BND_DES_3_Q20 HA07_TRIM4_BND_DES_3_Q50 HA07_TRIM4_BND_DES_3_Q100 HA07_TRIM4_BND_DES_3_Q200 HA07_TRIM4_BND_DES_3_Q1000 HA07_TRIM4_BND_DES_3_Q2_MRFS HA07_TRIM4_BND_DES_3_Q5_MRFS HA07_TRIM4_BND_DES_3_Q10_MRFS HA07_TRIM4_BND_DES_3_Q20_MRFS HA07_TRIM4_BND_DES_3_Q50_MRFS HA07_TRIM4_BND_DES_3_Q100_MRFS HA07_TRIM4_BND_DES_3_Q200_MRFS HA07_TRIM4_BND_DES_3_Q1000_MRFS HA07_TRIM4_BND_DES_3_Q10_HEFS HA07_TRIM4_BND_DES_3_Q100_HEFS HA07_TRIM4_BND_DES_3_Q1000_HEFS

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_TRIM4_DFS0_50%AEP	HA07_TRIM4_HD_DES_3_Q2	HA07_TRIM_4_MF_DES_3_Q2	HA07_TRIM4_HDmaps_DES_3_Q2_ds
HA07_TRIM4_DFS0_20%AEP	HA07_TRIM4_HD_DES_3_Q5	HA07_TRIM_4_MF_DES_3_Q5	HA07_TRIM4_HDmaps_DES_3_Q2_us
HA07_TRIM4_DFS0_10%AEP	HA07_TRIM4_HD_DES_3_Q10	HA07_TRIM_4_MF_DES_3_Q10	HA07_TRIM4_HDmaps_DES_3_Q5_ds
HA07_TRIM4_DFS0_5%AEP	HA07_TRIM4_HD_DES_3_Q20	HA07_TRIM_4_MF_DES_3_Q20	HA07_TRIM4_HDmaps_DES_3_Q5_us
HA07_TRIM4_DFS0_2%AEP	HA07_TRIM4_HD_DES_3_Q50	HA07_TRIM_4_MF_DES_3_Q50	HA07_TRIM4_HDmaps_DES_3_Q10
HA07_TRIM4_DFS0_1%AEP	HA07_TRIM4_HD_DES_3_Q100	HA07_TRIM_4_MF_DES_3_Q100	HA07_TRIM4_HDmaps_DES_3_Q20_ds
HA07_TRIM4_DFS0_0.5%AEP	HA07_TRIM4_HD_DES_3_Q200	HA07_TRIM_4_MF_DES_3_Q200	HA07_TRIM4_HDmaps_DES_3_Q20_us
HA07_TRIM4_DFS0_0.1%AEP	HA07_TRIM4_HD_DES_3_Q1000	HA07_TRIM_4_MF_DES_3_Q1000	HA07_TRIM4_HDmaps_DES_3_Q50_ds
HA07_TRIM4_BND_MRFS_Q2	HA07_TRIM4_HD_DES_3_Q2_MRFS	HA07_TRIM_4_MF_DES_3_Q2_MRFS	HA07_TRIM4_HDmaps_DES_3_Q50_us
HA07_TRIM4_BND_MRFS_Q5	HA07_TRIM4_HD_DES_3_Q5_MRFS	HA07_TRIM_4_MF_DES_3_Q5_MRFS	HA07_TRIM4_HDmaps_DES_3_Q100
HA07_TRIM4_BND_MRFS_Q10	HA07_TRIM4_HD_DES_3_Q10_MRFS	HA07_TRIM_4_MF_DES_3_Q10_MRFS	HA07_TRIM4_HDmaps_DES_3_Q200_ds
HA07_TRIM4_BND_MRFS_Q20	HA07_TRIM4_HD_DES_3_Q20_MRFS	HA07_TRIM_4_MF_DES_3_Q20_MRFS	HA07_TRIM4_HDmaps_DES_3_Q200_us
HA07_TRIM4_BND_MRFS_Q50	HA07_TRIM4_HD_DES_3_Q50_MRFS	HA07_TRIM_4_MF_DES_3_Q50_MRFS	HA07_TRIM4_HDmaps_DES_3_Q1000
HA07_TRIM4_BND_MRFS_Q100	HA07_TRIM4_HD_DES_3_Q100_MRFS	HA07_TRIM_4_MF_DES_3_Q100_MRFS	HA07_TRIM4_HDmaps_DES_3_Q2_MRFS_ds
HA07_TRIM4_BND_MRFS_Q200	HA07_TRIM4_HD_DES_3_Q200_MRFS	HA07_TRIM_4_MF_DES_3_Q200_MRFS	HA07_TRIM4_HDmaps_DES_3_Q2_MRFS_us
HA07_TRIM4_BND_MRFS_Q1000	HA07_TRIM4_HD_DES_3_Q1000_MRFS	HA07_TRIM_4_MF_DES_3_Q1000_MRFS	HA07_TRIM4_HDmaps_DES_3_Q5_MRFS_ds
HA07_TRIM4_BND_HEFS_Q10	HA07_TRIM4_HD_DES_3_Q10_HEFS	HA07_TRIM_4_MF_DES_3_Q10_HEFS	HA07_TRIM4_HDmaps_DES_3_Q5_MRFS_us
HA07_TRIM4_BND_HEFS_Q100	HA07_TRIM4_HD_DES_3_Q100_HEFS	HA07_TRIM_4_MF_DES_3_Q100_HEFS	HA07_TRIM4_HDmaps_DES_3_Q10_MRFS_ds
HA07_TRIM4_BND_HEFS_Q1000	HA07_TRIM4_HD_DES_3_Q1000_HEFS	HA07_TRIM_4_MF_DES_3_Q1000_HEFS	HA07_TRIM4_HDmaps_DES_3_Q10_MRFS_us
	HA07_TRIM4_HD_Background	HA07_TRIM_4_M11_DES_1_HOTSTART	HA07_TRIM4_HDmaps_DES_3_Q20_MRFS_ds
		HA07_TRIM_4_M11_DES_1_Q2_HOTSTART	HA07_TRIM4_HDmaps_DES_3_Q20_MRFS_us
		HA07_TRIM_4_M11_DES_1_Q5_HOTSTART	HA07_TRIM4_HDmaps_DES_3_Q50_MRFS_ds
		HA07_TRIM_4_M11_DES_1_Q20_HOTSTART	HA07_TRIM4_HDmaps_DES_3_Q50_MRFS_us
		HA07_TRIM_4_M11_DES_1_Q50_HOTSTART	HA07_TRIM4_HDmaps_DES_3_Q100_MRFS_ds
		HA07_TRIM_4_M11_DES_1_Q200_HOTSTART	HA07_TRIM4_HDmaps_DES_3_Q100_MRFS_us
		HA07_TRIM_4_M11_DES_1_HOTSTART_MRFS	HA07_TRIM4_HDmaps_DES_3_Q200_MRFS_ds
		HA07_TRIM_4_M11_DES_1_HOTSTART_HEFS	HA07_TRIM4_HDmaps_DES_3_Q200_MRFS_us
			HA07_TRIM4_HDmaps_DES_3_Q1000_MRFS_ds

MIKE 11 - DFS0 FILE Continued	MIKE 11 – HD FILE Continued	MIKE 11 - RESULTS FILE Continued	HD RESULTS FILE Continued
			HA07_TRIM4_HDmaps_DES_3_Q1000_MRFS_us HA07_TRIM4_HDmaps_DES_3_Q10_HEFS_ds HA07_TRIM4_HDmaps_DES_3_Q10_HEFS_us HA07_TRIM4_HDmaps_DES_3_Q100_HEFS_ds HA07_TRIM4_HDmaps_DES_3_Q100_HEFS_us HA07_TRIM4_HDmaps_DES_3_Q1000_HEFS_ds HA07_TRIM4_HDmaps_DES_3_Q1000_HEFS_us

Model Files - Sensitivity

MIKE FLOOD	MIKE 21	Boundary DFS0	MIKE 21 RESULTS
HA07_TRIM4_MF_SEN_1_Q100_bld HA07_TRIM4_MF_SEN_1_Q100_flow HA07_TRIM4_MF_SEN_1_Q100_hl_1 HA07_TRIM4_MF_SEN_1_Q100_hl_2 HA07_TRIM4_MF_SEN_1_Q100_rough	HA07_TRIM4_M21_SEN_1_Q100_bld HA07_TRIM4_M21_SEN_1_Q100_flow HA07_TRIM4_M21_SEN_1_Q100_hl_1 HA07_TRIM4_M21_SEN_1_Q100_hl_2 HA07_TRIM4_M21_SEN_1_Q100_rough HA07_TRIM4_dtsf2_Rec_MESH_bldgs_blockeddrivers_3 HA07_TRIM4_dtsf2_Rec_MESH_bldgs_blockeddrivers_3_SEN_bld rastert_ha07_trimr rastert_ha07_trimr_rough rastert_ha07_trimr_SEN_bld		HA07_TRIM4_MF_SEN_1_Q100_1_flow HA07_TRIM4_MF_SEN_1_Q100_bld HA07_TRIM4_MF_SEN_1_Q100_hl_1 HA07_TRIM4_MF_SEN_1_Q100_hl_2 HA07_TRIM4_MF_SEN_1_Q100_rough

MIKE 11 - SIM FILE	MIKE 11 - NETWORK FILE	MIKE 11 - CROSS-SECTION FILE	MIKE 11 - BOUNDARY FILE
HA07_TRIM4_M11_SEN_1_Q100_bld HA07_TRIM4_M11_SEN_1_Q100_flow HA07_TRIM4_M11_SEN_1_Q100_hl_1 HA07_TRIM4_M11_SEN_1_Q100_hl_2 HA07_TRIM4_M11_SEN_1_Q100_rough	HA07_TRIM4_NWK_DES_1_Irish HA07_TRIM4_NWK_SEN_hl_1 HA07_TRIM4_NWK_SEN_hl_2	HA07_TRIM4_XNS_DES_1 HA07_TRIM4_XNS_SEN_hl_1 HA07_TRIM4_XNS_SEN_hl_2 HA07_TRIM4_XNS_SEN_rough	HA07_TRIM4_BND_DES_1_Q100 HA07_TRIM4_BND_SEN_1_Q100_flow HA07_TRIM4_BND_DES_1_HOTSTART

MIKE 11 - DFS0 FILE	MIKE 11 – HD FILE	MIKE 11 - RESULTS FILE	HD RESULTS FILE
HA07_TRIM4_DFS0_1%AEP HA07_TRIM4_DFS0_1%AEP_SEN_flow	HA07_TRIM4_HD_SEN_1_Q100_bld HA07_TRIM4_HD_SEN_1_Q100_flow HA07_TRIM4_HD_SEN_1_Q100_hl_1 HA07_TRIM4_HD_SEN_1_Q100_hl_2 HA07_TRIM4_HD_SEN_1_Q100_rough	HA07_TRIM_4_MF_SEN_1_Q100_bld HA07_TRIM_4_MF_SEN_1_Q100_flow HA07_TRIM_4_MF_SEN_1_Q100_hl_1 HA07_TRIM_4_MF_SEN_1_Q100_hl_2 HA07_TRIM_4_MF_SEN_1_Q100_rough HA07_TRIM_4_M11_DES_1_HOTSTART HA07_TRIM_4_M11_SEN_1_HOTSTART_rough	N/A

GIS Deliverables - Hazard

Flood Extent Maps (Shapefiles) (Shapefiles)		Flood Depth Files (Raster)	Flood Velocity Files (Raster)	Risk to Life Function (Raster)
<u>Fluvial</u> E42EXFCD500F0 E42EXFCD200F0 E42EXFCD100F0 E42EXFCD050F0 E42EXFCD020F0 E42EXFCD010F0 E42EXFCD005F0 E42EXFCD001F0	Water Level and Flows <u>Fluvial</u> E42NFCDF0 E42NFMDF0 E42NFHDF0 Flood Defences (Shapefiles) N/A Defended Areas (Shapefiles) N/A Wave Overtopping (Shapefiles) N/A	<u>Fluvial</u> E42DPFCD500F0 E42DPFCD200F0 E42DPFCD100F0 E42DPFCD050F0 E42DPFCD020F0 E42DPFCD010F0 E42DPFCD005F0 E42DPFCD001F0 E42DPFMD500F0 E42DPFMD200F0 E42DPFMD100F0 E42DPFMD050F0 E42DPFMD020F0 E42DPFMD010F0 E42DPFMD005F0 E42DPFMD001F0 E42DPFHD100F0 E42DPFHD010F0 E42DPFHD001F0	<u>Fluvial</u> E42VLFCDF0500F0 E42VLFCDF0200F0 E42VLFCDF0100F0 E42VLFCDF0050F0 E42VLFCDF0020F0 E42VLFCDF0010F0 E42VLFCDF0005F0 E42VLFCDF0001F0	<u>Fluvial</u> E42RLFCD100F0 E42RLFCD010F0 E42RLFCD001F0 Flood Zones (Shapefiles) E42ZNA_FCDF0 E42ZNB_FCDF0 E42ZNA_FMDF0 E42ZNB_FMDF0
Defence Failure Scenario				
Extent	Depth	Velocity		
N/A	N/A	N/A		

GIS Deliverables - Risk

Specific Risk - No. Inhabitants (Raster)	Specific Risk –Type of Activity (UoM Scale)	Specific Risk – Risk Density (Raster)
<u>Fluvial</u> E42RIFCD100F0 E42RIFCD010F0 E42RIFCD001F0 E42RIFMD100F0 E42RIFMD010F0 E42RIFMD001F0	<u>Fluvial</u> E07_RTFC001_F0 E07_RTMD001_F0	<u>Fluvial</u> E42RDFCD001F0 E42RDFMD001F0
General Risk - Environment	General Risk – Cultural Heritage	General Risk – Economy
N/A	N/A	N/A