South Western CFRAM Study

Hydraulics and Flood Mapping Report,
Unit of Management 20

June 2016

The Office of Public Works

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Issue and revision record

Revision	Date	Originator	Checker	Approver	Description	Standard
A	July 2014	M Piggott	R Gamble	R Gamble	Draft	
B	February 2015	M Piggott	R Gamble	R Gamble	Draft Final	
C	October 2015	M Piggott	B O'Connor	B O'Connor	Draft Final Minor amendments	
D	June 2016	M Piggott	B O'Connor	B O'Connor	Final Additional minor amendments	

Contents

Executive Summary

The Office of Public Works (OPW) is undertaking six catchment-based flood risk assessment and management (CFRAM) studies to identify and map areas with existing and potential future flood risk across Ireland. Mott MacDonald Ireland Ltd. has been appointed by the OPW to assess flood risk and develop flood risk management options in the South Western River Basin District. This hydraulics and flood mapping report is one of a series of reports being produced as part of the South West Catchment Flood Risk Assessment and Management Study (SW CFRAM Study). It details the development of the hydraulic models used to map current and future flood risk across Unit of Management 20. The model results and flood maps from this report inform the subsequent strategic environmental assessment and flood risk management plans.

Three 1D/2D hydraulic models have been developed for Dunmanway, Innishannon and Schull, which are defined as Areas for Further Assessment (AFAs), to assess fluvial and coastal flood risk for various flood probabilities. Bandon, Skibbereen and Clonakilty have been assessed separately to the main CFRAM study due to the recent flooding in these areas and are not reviewed within this report. The river channels have been modelled using 1D ISIS and 1D ESTRY software to calculate flows and head losses at hydraulic structures. The 2D TUFLOW software has been used to simulate the multi-directional flows across the complex urban floodplains. The 1D and 2D components of the models are hydrodynamically linked such that water can flow between the river and floodplain during the event to simulate the observed flood mechanisms.

Dunmanway has been calibrated for three events, namely: 1st January 1991, 12th October 1996 and 19th November 2009. Innishannon has also been validated against the 19th November 2009 event. Schull has been calibrated for two recent events namely: 6th October 2009 and 15th August 2012. Sensitivity tests were undertaken on flow, downstream level and Manning's 'n' for all models.

The calibrated and tested models were then run for eight flood probabilities under the current design scenario, eight flood probabilities under the mid-range future scenario, and three flood probabilities under the high end future scenario from both fluvial and coastal sources. The flood extents, flood zones, flood depths, flood velocities and flood hazards have all been mapped for the specified scenarios, and are provided in the Appendices to this report.

The findings from the modelling results and flood maps will be used as inputs to the flood risk review. The knowledge of the flood mechanisms, critical structures and impact of flooding established in this report will support the development of sustainable and appropriate flood risk management options in the flood risk areas.

1 Introduction

1.1 The CFRAM Process

Flooding is a natural process that occurs throughout Ireland as a result of extreme rainfall, river flows, storm surges, waves, and high groundwater. Flooding can become an issue where the flood waters interact with people, property, farmland and protected habitats.

The Office of Public Works (OPW) is the lead agency in implementing flood risk management policy in Ireland. Mott MacDonald Ireland Ltd. has been appointed by the OPW to undertake the Catchment Flood Risk Assessment and Management Study (CFRAM Study) for the South Western River Basin District, henceforth referred to as the SW CFRAM Study. Under the project, Mott MacDonald will produce Flood Risk Management Plans which will set out recommendations for the management of existing flood risk in the Study Area, and also assess the potential for significant increases in this risk due to climate change, on-going development and other pressures that may arise in the future.

The South Western River Basin District is split into five Units of Management (UoM). These Units follow watershed catchment boundaries and do not relate to political boundaries. The Units are as follows;

- The Blackwater catchment (UoM18)
- The Lee / Cork Harbour Catchment (UoM19)
- The Bandon / Skibbereen Catchment (UoM20)
- The Dunmanus / Bantry / Kenmare Bay Catchment (UoM21)
- The Laune / Maine / Dingle Bay Catchment (UoM22)

Map 1.1 displays the extent of UoM20 which is the subject of this report. The report covers three Areas for Further Assessment (AFAs), namely: Dunmanway, Innishannon and Schull. Bandon, Skibbereen and Clonakilty have been assessed separately to the main CFRAM study due to the recent flooding in these areas and are not reviewed within this report.

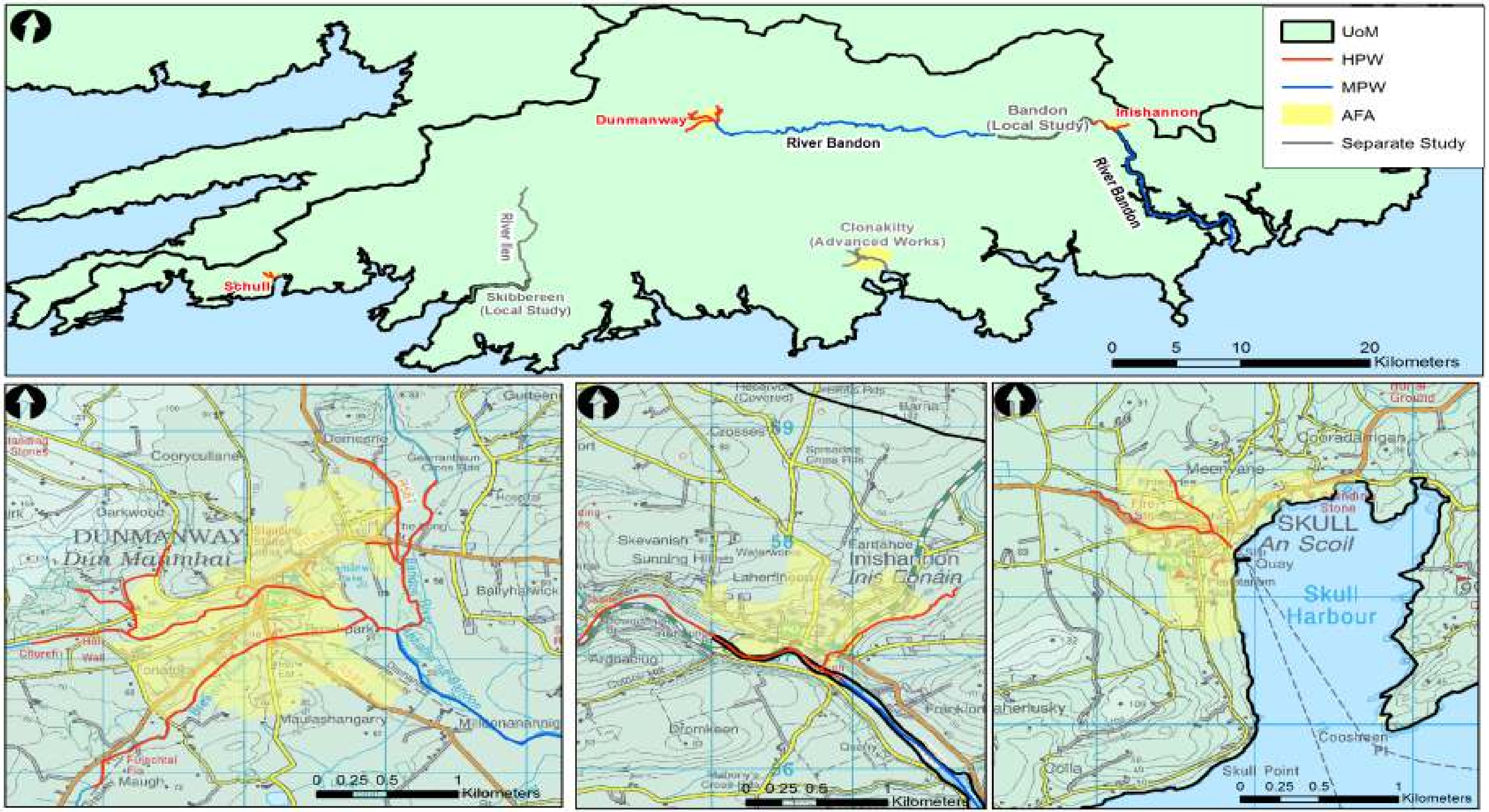
The overarching aims of the SW CFRAM Study are as follows:

- Identify and map the existing and potential future flood hazard;
- Assess and map the existing and potential future flood risk; and,
- Identify viable structural and non-structural options and measures for the effective and sustainable management of flood risk in the South Western River Basin District.

In order to achieve the overarching aims, the study is being undertaken in the following stages:

- Data collection;
- Hydrological analysis;
- Hydraulic analysis;
- Development of flood maps;
- Strategic Environmental Assessment and a Habitats Directive Appropriate Assessment;
- Flood risk assessment of people, economy and environment;
- Development and assessment of flood risk mitigation options; and,
- Development of the Flood Risk Management Plans (FRMPs).

Map 1.1: Unit of Management 20



1.2 Report Structure

This report details the hydraulic analysis and flood mapping at the following locations within Unit of Management 20:

- Dunmanway and the River Bandon between Dunmanway and Bandon
- Inishannon and the River Bandon between Inishannon and Kinsale
- Schull

This report does not review or update flood maps in Bandon Town, Clonakilty and Skibbereen which have been assessed separately under the Bandon Flood Relief Scheme (2011), Advanced Works for Clonakilty (2012) and River Ilen Flood Risk Assessment and Management Study (2013) respectively.

The objectives of this report are:

- To document the findings and conclusions of the topographic survey
- To document the analysis and assumptions taken to develop hydraulic models for the AFAs and MPWs
- To map existing and potential flood hazard for the design scenarios
- To use the hydraulic models and maps to assess existing and potential future flood risk and make recommendations for feasible flood risk management options and future modelling.

The main report outlines the generic approach to the hydraulic modelling and mapping. Detailed analysis and discussion of hydraulic modelling and mapping for each Area for Further Assessment (AFA) is provided in the Appendices. Table 1.1 outlines the report structure and scope of work with a description of the key contents.

Table 1.1: Report Structure

Chapter	Key Contents of Chapter
1. Introduction	▪ The SW CFRAM process ▪ Report structure ▪ Flood probabilities
2. Data Collection, Survey and Review	▪ Summary of data sources ▪ Review of all topographical and land cover data used
3. Hydrological Approach	▪ Summary of design inflows and downstream conditions ▪ Summary of joint probability ▪ Integration of design hydrology into the hydraulic model
4. Hydraulic Modelling Approach	▪ Discussion of general schematisation ▪ Discussion of overarching methodology for modelling river channels, key structure types and the floodplain ▪ Model parameters
5. Calibration and Sensitivity Analysis	▪ Discussion of calibration events ▪ Discussion of sensitivity tests on key parameters
6. Design Runs and Model Performance	▪ List of design runs ▪ Discussion of model convergence and performance
7. Assumptions and Limitations	▪ The key limitations and assumptions of the models and associated data
8. Flood Mapping Approach	▪ Discussion of the flood mapping process ▪ The types of flood hazard and specific flood risk maps and how these were calculated
9. Model and Mapping Results	▪ Discussion of flood mechanism, frequency of flood issues, risk to life, critical structures, sensitivity to assumptions and

Chapter	Key Contents of Chapter
	guidance on flood risk management options for each AFA
10. Summary and Recommendations	▪ Conclusions and key findings from the hydraulic analysis ▪ Summary of flood hazard in the Unit of Management ▪ Recommendations for flood mitigation option development and the FRMP ▪ Recommendations for future improvements in the hydraulic modelling

1.3 Flood Probabilities

The SW CFRAM Study refers to flood probabilities in terms of annual exceedance probability in preference to the use of "return periods" as used in previous reports. The probability or chance of a flood event occurring in any given year can be a useful tool to better understand the rarity of events of specific magnitude for flood risk management. Due to popular descriptors of floods involving terms like the "1 in 100 year flood" there can be public misunderstanding that a location will be safe from a repeat event of the same magnitude, extent and volume for the duration of the term (100 years in the above example). In reality, flood events of a similar or greater magnitude can occur again at any time.

Annual Exceedance Probability, henceforth referred to as AEP, is a term used throughout this report and the wider CFRAM studies to refer to the rarity of a flood event. The probability of a flood relates to the likelihood of an event of that size or larger occurring within any one year period. For example, a 1 in 100 year flood has a chance of one in a hundred of occurring in any given year; 1:100 odds of occurring in any given year; or a 1% likelihood of occurring. This is described as a 1% annual exceedance probability (AEP) flood event.

Table 1.2 converts the 'return periods' to %AEP for key flood events as a reference to previous studies.

Table 1.2: Flood Probabilities

% Annual Exceedance Probability (%AEP)	Odds of a Flood Event in Any Given Year	Chance of a Flood Event in Any Given Year or Previous 'Return Period'
50%	1:2	1 in 2
20%	1:5	1 in 5
10%	1:10	1 in 10
5%	1:20	1 in 20
2%	1:50	1 in 50
1%	1:100	1 in 100
0.5%	1:200	1 in 200
0.1%	1:1000	1 in 1000

The hydraulic analysis and flood mapping uses a number of other acronyms and technical terminology which are defined in the glossary of this report.

2 Data Collection, Survey and Review

2.1 Data Collection and Review

A range of different data sources have been used to undertake the hydraulic analysis for the SW CFRAM Study. Table 2.1 lists the data used in Unit of Management 20 and the confidence in each dataset based on the review in the following sections.

Table 2.1: Summary of Data Used

Type	Details	Owner	Date Captured
Geometric Survey Data	River channel and structure survey and photographs of all HPWs and MPWs in UoM20	OPW	As part of this study 2013-2014
	Bandon and Ballymakerra Channel and floodplain Survey	OPW	2010
Detailed Digital Terrain Models	Filtered LiDAR data for AFAs	OPW	2010
	Associated high resolution aerial photography	OPW	2010
National Height Model	IFSAR coarse elevation data with national coverage	OPW	2010
OSI Mapping	Building footprints and vector data of land cover	OSI	2010

The specific details of the data used for each model are included in the model Appendices.

2.2 Geometric Survey Data

As part of this study, extensive river channel survey was undertaken of all the High Priority Watercourses (HPWs) in UoM20 between February 2013 and March 2014 by Murphy Surveys Ltd (Map 2.1). In additional, river section and structure data was extracted for the lower Bandon was extracted from the Bandon and Ballymakeera Channel and Floodplain Survey (October 2010). This survey data was within 0.1m of the recent Murphy's Survey and was used to inform the upstream sections of the Innishannon model.

In both cases, the survey captured topographic information about the elevations, dimensions and hydraulic conditions of the river channel and hydraulic structures. The detailed location of each cross-section is displayed in the model geoschematics provided at the end of the model build proformas in the Appendices. The detailed South West CFRAM Contract 5 Survey is available in a separate report (August 2013).

The following quality assurance of the survey data was also undertaken as part of the hydraulic analysis:

- Sections were surveyed from left bank to right bank facing downstream;
- Sections at the structure face were surveyed parallel to the structure and the skew angle recorded;
- Identification of any gaps and anomalies in the survey drawings or hydraulic model-formatted files;
- Analysis of changes and consistency with any other recent survey data.

The river channel survey was found to be surveyed from left to right bank and in parallel with structures, in accordance with the survey specification. Therefore, bed levels and low flow channel shape were linearly interpolated from the upstream and downstream sections. This assumption ensures that:

- The bed is not artificially elevated due to missing data; and,
- These sections do not act as hydraulic weir controls when the flow through is sub-critical in reality.

All of the geometric survey data captured by the surveyor was reviewed, with checks carried out on 10% of the cross sections. Using GPS survey equipment spot levels checks were carried out on structures and cross sections captured by the surveyor. The levels were reviewed and differences compared at bank crest. The average difference between the levels of the survey and the spot checks was found to be 0.27m in UoM20. This is considered to be a good correlation when considering that the comparison points were mostly on rough ground in rural areas. The exact locations are difficult to replicate, and the bank crest could vary or settle where they comprise of natural materials.

2.3 Digital Terrain Model Data

As part of this study, an aerial LiDAR (Light Detection And Ranging) survey of each AFA was captured in April 2012 as a point cloud with an average of 2 points per square metre (Map 2.2). Subsequently, the raw LiDAR was collated to produce a digital surface model, and post-processed to produce a bare-earth or Digital Terrain Model (DTM) by removing artificial structures, including buildings walls and bridges, and vegetation such as trees and hedges. The DTMs were processed for grid resolutions of 2m, 5m and 10m based on the same raw data.

The LiDAR data and OSi mapping data has been checked against site observations and latest Cork County Council Proposal for the AFAs and findings commented on in the model build proforma. No new developments were found in the floodplain for Dunmanway and Inishannon. There was ongoing construction at Schull "Copper Point". However this was outside the fluvial floodplain so did not affect results. Any future assessment of pluvial flooding will need to consider changes to urban drainage and land cover with this development.

The LiDAR DTM was compared with the validated survey for large flat surfaces such as roads and hard-standing or flat pasture where hard-standing was limited and deemed to be appropriate for use without further adjustment. The vertical accuracy was found to be +/-0.05m on average within urban areas, such as Dunmanway. The vertical accuracy increased to +/-0.23m in more rural areas such as downstream of Dunmanway where the filtering process to remove dense vegetation reduced the accuracy of LiDAR elevations.

Where LiDAR was not available on the Bandon MPW reach, IFSAR data from OSI's national height model has been used to create the DTM for hydraulic modelling and flood mapping. IFSAR has a lower vertical accuracy than LiDAR of ± 0.5m on average. When the IFSAR data was compared with river channel survey on the floodplain discrepancies between -1m and +1m were found in some locations on the Bandon, but typically differed by 0.4m on average. Therefore, the IFSAR data was adjusted to meet the river channel survey points and then joined with the LiDAR data to create a complete DTM. Every effort

has been made to ensure a consistent transition from LiDAR to IFSAR but some uncertainty remains in the areas which use IFSAR due to the poorer data quality.

2.4 Land Cover Data

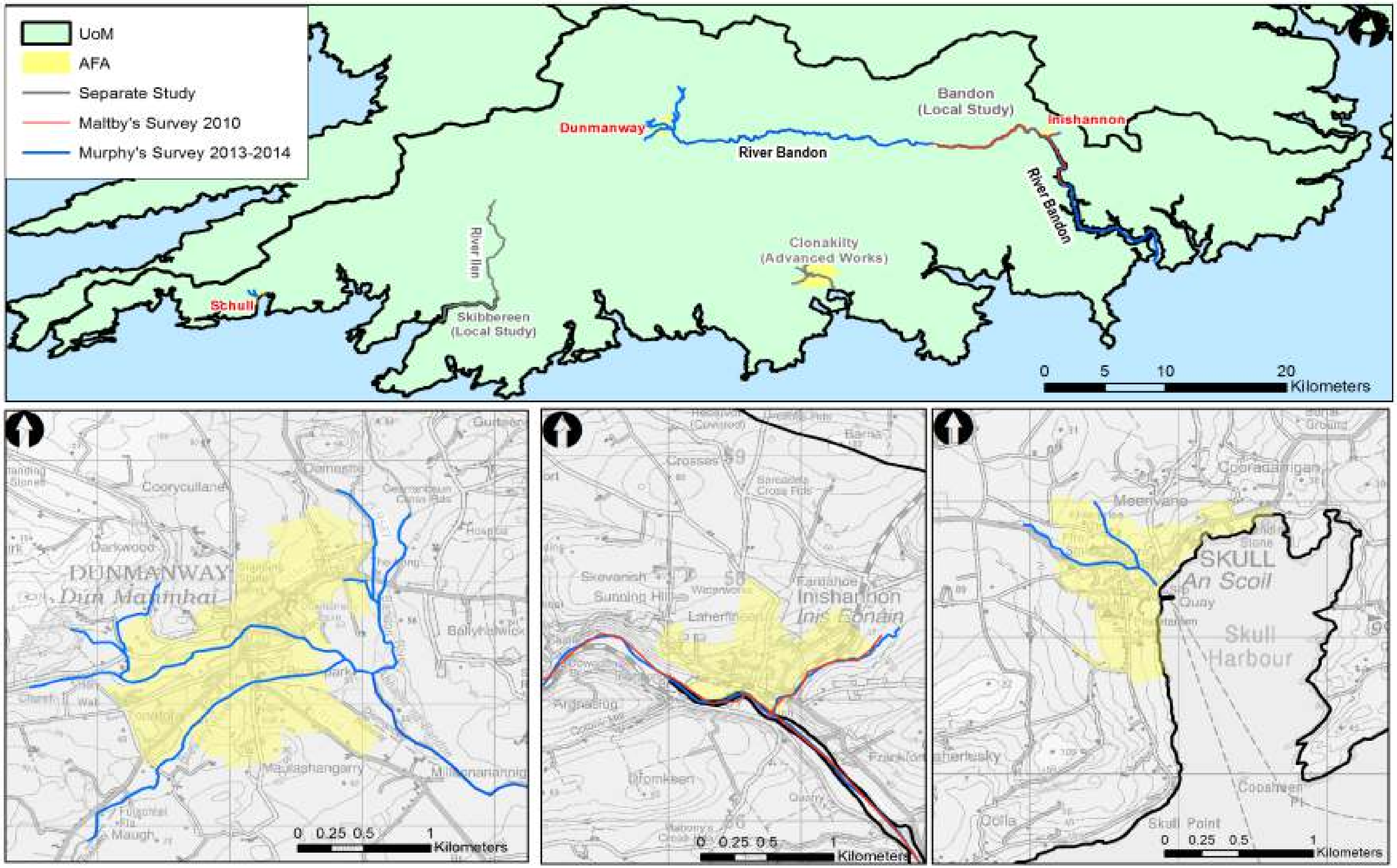
The various types of surfaces in the AFAs were assessed from the following data sources to inform the hydraulic roughness parameters for modelling:

- Building footprints derived from OSI mapping
- 1:1000,1;2500 and 1:5000 vector OSI Mapping
- Surface cover detailed in the geometric survey cross-section data, geometric survey photographs and high resolution aerial photography associated with the LiDAR survey
- Site visits

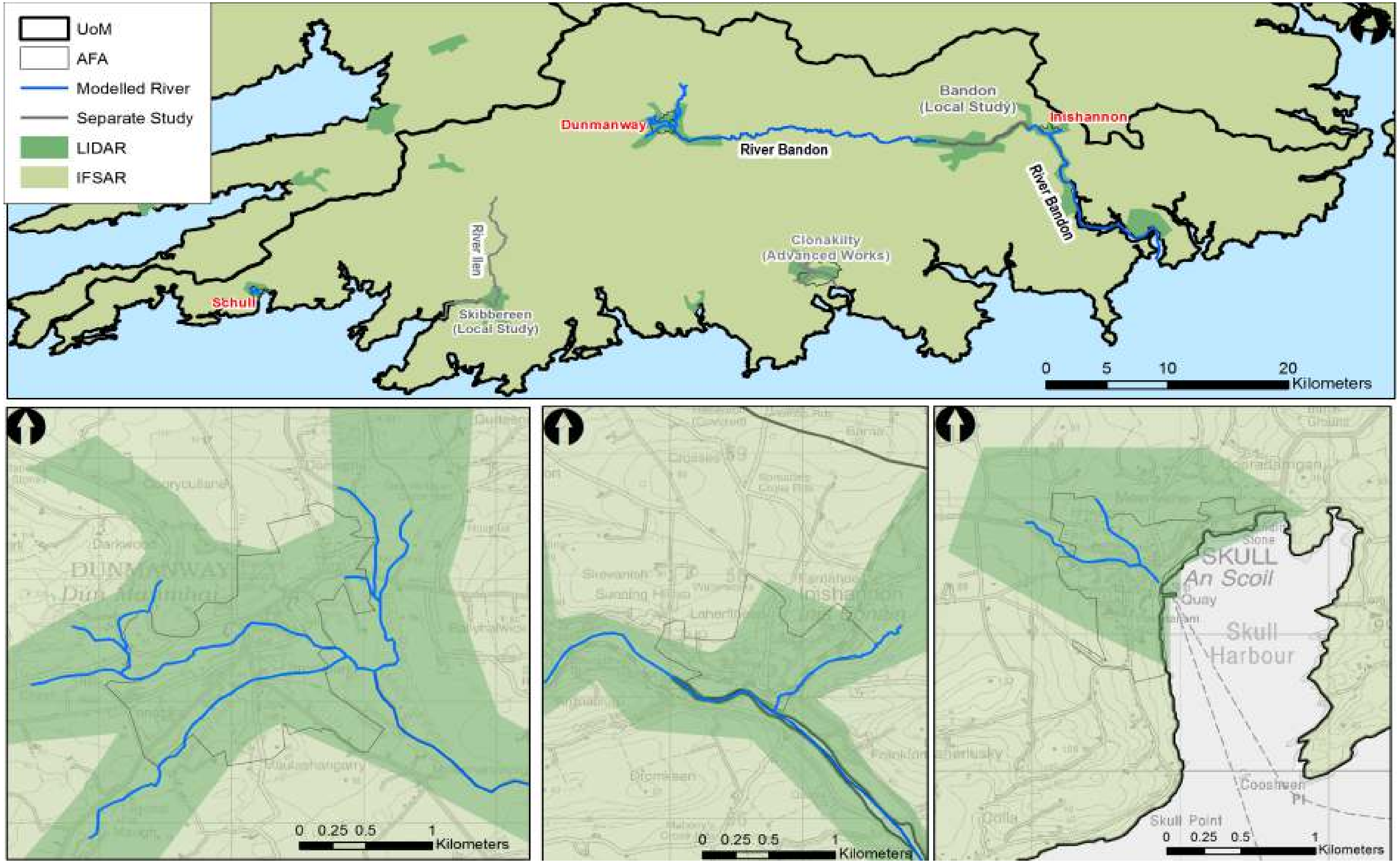
The mapping datasets were used in the first instance to classify land cover within each AFA into broad surface types of river bed and standing water; river banks; dense vegetation; pasture, parkland and arable; buildings; and, hard-standing urban areas. The land cover was subsequently refined during the model build process using the survey, aerial photography and site observations. The resultant detailed land cover for each AFA is provided in the Appendices.

The European Environment Agency CORINE land cover dataset was not used because the data is based on satellite imagery which is relatively coarse and does not differentiate buildings from surrounding roads and gardens within urban areas. Therefore, the more detailed OSI mapping was used in urban areas in conjunction with site observations.

Map 2.1: River Channel Survey in UoM20



Map 2.2: LiDAR Coverage in UoM20



3 Hydrological Approach

3.1 Summary of Design Hydrology

As part of the previous UoM20 Hydrology Report, design peak flows and hydrographs were derived at hydrological estimation points for the 50%, 20%, 10%, 5%, 2%, 1% , 0.5% and 0.1%AEP fluvial flood events.

The hydrological estimation points were located in AFAs at the inflows to the hydraulic models, upstream and downstream of confluences with significant tributaries, and at the downstream limit of the hydraulic models. Catchment descriptors were extracted from the FSU database and checked against the National Height Model, OSI contours and site observations. For smaller catchments not available in the FSU database, the catchment descriptors were derived from the difference between the upstream and downstream points and checked against the available data.

The design peak flows were derived using the recommended statistical method outlined in FSU Work Packages 2.2 and 2.3, and adjusted using the hydrological similar pivotal sites of 20001, 20002, 20006, 20008, 20015 and 20016, as well as 22006, 22022, 30020 and 36021.White Bridge gauge 22009 was used to inform the QMED for the small Dunmanway Lake catchment. However the gauge was not deemed suitable to estimate extreme flows above QMED. Table 3.1 summarises the design peak flows for each catchment in the AFAs for ease of reference.

Table 3.1: UoM20 Design Peak Flood Flows at Key Locations

HEP	Gauge	Flow (m^3/s)							
		50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.1%AEP
Dunmanway AFA									
20015	Ardcahan Bridge Gauge	59	73	82	92	107	120	135	177
20008	Long Bridge Gauge	75	90	102	116	135	151	169	222
20_2096_1	Bandon at Dunmanway Lake Confluence	76	91	103	117	136	153	171	225
20_2093_1	Bandon at Dirty Confluence	93	112	128	145	173	197	225	309
20016	Bealaboy Bridge Gauge	95	115	131	148	176	201	229	315
20_602_2	Dirty River at Bandon Confluence	21	26	30	34	40	46	51	68
20_790_7	Brewery River at Dirty Confluence	7	8	10	11	13	14	16	21
Upper Bandon MPW									
20_754_1	Bandon at Bealanscartane	101	121	138	157	186	212	242	333

HEP	Gauge	Flow (m^3/s)							
		50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.1%AEP
20_742_1	Bandon at River Blackwater	121	146	166	189	224	256	292	401
20_664_3	Bandon Reach 1 downstream	134	162	184	209	248	283	323	443
20_649_1	20001 Bandon Town Gauge	122	162	194	222	288	339	399	575
Inishannon AFA and Lower Bandon MPW									
20_2230_3	20002 Curranure Gauge	128	169	204	232	302	356	419	604
20_2230_3+	Bandon at Inishannon Stream	144	192	231	263	342	403	473	683
20_147_2+	Bandon at Rockhouse Creek	173	230	277	315	411	483	568	820
20_2236_5+	Bandon at Ballinadee Creek	181	24	290	330	430	506	595	858
20_2224_3+	Bandon at Whitecastle Creek (Kinsale)	184	245	295	336	437	515	605	872
Schull AFA									
20_1990_1	Schull Stream upstream	1.0	1.3	1.5	1.7	2.0	2.2	2.5	3.3
20_1916_1	Schull at Meenvane Stream Confluence	2.8	3.6	4.1	4.6	5.4	6.1	6.9	9.2
20_1916_2	Schull Stream downstream	3.1	3.9	4.4	5.0	5.9	6.7	7.5	10.0

The annual maximum flood hydrographs were standardised and compared to derive the width exceedance for specific percentage flows at gauges on the River Bandon. The design median flood hydrograph was derived from the width exceedance analysis. The FSU WP 3.2 UPO-ERR-gamma curve was fitted to the design median flood hydrographs and the parameters applied to derive the design hydrograph shape for the ungauged HEPs.

The design hydrographs for Schull have been derived using the FSSR16 rainfall-runoff methodology scaled to the design peak flows above. The FSSR16 approach was applied because it is the most appropriate approach for such wet rapid response catchments, as it takes slope (S1085) and catchment area into account within hydrograph shape calculations.

The tidal conditions used in combination with the fluvial flows are discussed in Section 3.3.

3.2 Summary of Design Coastal Conditions

As part of the previous UoM20 Hydrology Report, design total tide plus surge levels and tidal hydrographs were derived at each AFA for the 50%, 20%, 10%, 5%, 2%, 1% , 0.5% and 0.1%AEP coastal flood events. The total tide plus surge levels were extracted directly from the nearest ICPSS offshore point in the absence of more detailed level data at each AFA. The resultant design levels are provided in Table 3.2.

Table 3.2: UoM21 Design Total Tide Plus Surge Levels

Location	Source	Total Tide Plus Surge Level (mODM)							
		50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.1%AEP
Kinsale Lower Bandon	ICPSS Point S24	2.05	2.14	2.21	2.27	2.36	2.43	2.49	2.64
Schull	ICPSS Point S12	1.91	2.00	2.07	2.14	2.23	2.30	2.36	2.52

The design astronomic tidal curve was transferred from the primary port of Cobh based on the United Kingdom Hydrographic Office Admiralty Tide Tables. The design surge profile was derived from analysis of typical surge durations along the South West coast and scaled on top of the astronomic tide to meet the design total tide plus surge level above.

The fluvial flows used in combination with the extreme tide plus surge conditions are discussed in Section 3.3.

3.3 Joint Probability

The design flows on each river reach and total tide plus surge levels provided above have been derived independently of each other. In reality, there can be dependency between sources of flooding which can be described by the joint probability to achieve a target %AEP event. The CFRAM study considers the following joint probabilities:

- Fluvial-fluvial – Where a range of combinations of flow on a main river combines with flow on a tributary to generate a specific %AEP flood downstream.
- Fluvial-coastal – Where an approaching depression generates a storm surge which combines with a river flood to generate a specific %AEP flood at the coast.
- Tidal- Wave – Where an approaching depression generates a storm surge which combines with extreme wave to generate a specific %AEP flood at the coast.

The fluvial-fluvial dependence was guided by the methodology set out in Flood Studies Update Work Package 3.4. In UoM20, the joint probability of tributaries was found to be largely dictated by the size of the incoming catchment relative to the main watercourse. The joint probability %AEP on the smaller tributary inflows tended to be the more frequent smaller events in order to achieve the target flow on the main watercourse (Table 3.4). The exception was the Upper Bandon-Caha River confluence and Brewery River-Dirty River confluence in Dunmanway, and Bandon-Blackwater confluence on the Upper Bandon MPW.

These have similar probabilities to the main river as the tributaries contribute approximately half of the flow to the downstream reach.

The extreme fluvial flow estimates at Kinsale and ICPSS total tide plus surge levels were used to derive the joint probability combinations between fluvial and coastal events based on the DEFRA FD2308_TR1 desk-based assessment tool in accordance with GN20[1]. The dependence of river flow and storm surge in these estuaries tended to be “well” to “strongly” correlated due to the orientation of the bays and catchments. Extensive sensitivity analysis was undertaken on the 0.5% AEP event as part of the nearby Lee CFRAM pilot study, and found the two main critical scenarios to be as follows:

- Target flow and the MHWS tide; and
- 50%AEP Flow and the target total tide plus surge level.

The Irish Coastal Water Level and Wave Study (ICWWS) did not identify any location vulnerable to wave overtopping in UoM20. Therefore, wave overtopping has not been considered any further in the hydrological analysis.

In order to simplify the modelling process, the closest design AEP to the joint probability estimate was selected. The flow was interpolated where the joint probability was half way between two design AEPs. The resultant joint probabilities are provided in Table 3.3.

Table 3.3: Summary of Joint Probabilities Used

Source	Main River AEP	Tributary AEP	Coastal AEP
Fluvial	50%	71%	50% to MHWS
	20%	46%	50% to MHWS
	10%	35%	50% to MHWS
	5%	23%	50% to MHWS
	2%	10%	50% to MHWS
	1.0%	6.1%	50% to MHWS
	0.5%	3.8%	50% to MHWS
	0.1%	1.2%	50% to MHWS
Coastal	50%	71%	50%
	50%	71%	20%
	50%	71%	10%
	50%	71%	5%
	50%	71%	2%
	50%	71%	1.0%
	50%	71%	0.5%
	50%	71%	0.1%

[1] RPS(2012) CFRAM Guidance Note 20, Joint Probability Guidance.

3.4 Integration of Hydrology and Hydraulic Modelling

The design hydrological inflows summarised in Section 3.1 have been integrated with the hydraulic models as follows:

- Point inflows at the upstream model extents;
- Point inflows at key tributary inflows;
- Lateral inflows representing the inflow from the intervening areas between target HEPs.

The lateral inflows have been calculated from the difference between the design flow hydrographs from the upstream and downstream HEPs for a reach. The resultant hydrographs have been distributed evenly across those locations where the contributing area increases linearly downstream, or area-weighted where the contributing area increases disproportionally downstream.

The point inflows representing the upstream model extents and tributary inflows were applied to the uppermost cross-sections in the hydraulic model. The inflow for the entire catchment was simplified and lumped at the upstream end of the model for the Innishannon, Brewery and Schull catchments, because the intermediate catchment was relatively small.

The lateral inflows have been integrated with the relevant cross-sections at locations which fit the following criteria:

- Natural inflows from minor watercourses which are not considered explicitly within the hydrology;
- Overland flow paths identified from surveyed low points in the river bank and site walkover;
- Reconciliation adjustments of hydrological flow estimates and hydraulic models.

The model proformas provided in the Appendices detail the location of each lateral inflow.

In order to enhance the modelling outputs and ensure hydrological continuity along the larger catchments, the hydraulic models were calibrated to the design peak flows derived at the target HEPs. The hydrological inflows were iteratively scaled and phased such that the hydraulic model maintains the design peak flows along the reach as part of the hydraulic modelling process. However, it should be noted that the design fluvial flows do not consider the following hydraulic processes:

- Backwater effect at confluences;
- Exchange of flows between tributaries at confluences; and,
- Significant modification to the hydrograph shape due to floodplain attenuation and /or hydraulic structures.

Therefore, it was not appropriate to calibrate the hydraulic model to HEPs upstream of confluences where there are significant out-of-bank flows. Table 3.4 details the timing adjustments made to the inflow hydrographs to achieve the design peak flows at the target HEPS for each reach. The downstream tributaries on the Upper Bandon were iteratively phased to meet the design peak flows for the 1%AEP and greater magnitude events.

Table 3.4: Phasing of Inflows

Model	Sub-catchment	Time Shift Applied to the Tributary Inflows to Achieve the Design Peak Flows at the target HEPS (Hours)
Dunmanway AFA and Upper Bandon MPW	Tributaries to Bealboy Bridge	0.00
	Bealascartane	+2.00
	Blackwater (Bandon)	+4.00 (+8.00 for 1%AEP+)
	Enniskean	+7.00(+11.00 for 1%AEP+)
	Kilowen	+8.00(+12.00 for 1%AEP+)
Inishannon AFA and Lower Bandon MPW	Inishannon	+1.00
	Tributaries to Kinsale	+3.00
Schull AFA	Schull Stream	0.00
	Meevane Stream	0.00

The design tide plus surge hydrographs discussed in Section 3.2 were used to form the downstream boundary conditions for the hydraulic models. An iterative approach was used to phase the design tide plus surge hydrographs so that the peak tide coincides with the peak flow in the AFA. This phasing is a conservative assumption of combined flood risk in line with the joint-probability analysis in Section 3.3 above. Table 3.5 outlines the downstream conditions applied and time by which the tidal hydrograph was adjusted in order to meet the peak river flow.

Table 3.5: Downstream Boundary Conditions

Model	Downstream Condition	Time Adjustment to Coincide Peak Tide with Peak Flow (Hours)
Dunmanway AFA and Upper Bandon MPW	Fluvial downstream boundary set by Flow-Stage boundary	Not Applicable
Inishannon AFA and Lower Bandon MPW	Tidal boundary at Kinsale.	5.0
Schull AFA	Tidal boundary along the coast.	0.0

3.5 Critical Storm Duration

In UoM20, the median width hydrographs have been derived at the gauged locations to establish the design storm duration at target HEPs across each catchment. The duration of the tributary inflows were based on the gauged duration. The intermediate inflows account for the difference in duration between the target HEPs within the same hydrological catchment. In Schull, the duration has been based entirely on the FSR time peak equation (function of SAAR, S1085 and MSL) due to lack of gauges in this wet rapid response catchment. Table 3.7 outlines design storm durations for UoM20.

Table 3.6: Critical Storm Durations for Rainfall-Runoff Inflows

AFA/MPW	Method	Design Duration (Hours)
Dunmanway and Upper Bandon	Gauged Median Width Exceedance	31
Lower Bandon	Gauged Median Width Exceedance	42
Inishannon	FSR estimate used to refine FSU UPO-ERR Gamma Parameters	7
Schull	FSR estimate	10

4 Hydraulic Modelling Approach

4.1 Schematisation

Table 4.1 outlines the general approach for each AFA in UoM20. Map 4.1 presents the areas and reaches modelled.

Table 4.1: UoM20 Model Approach

Model ID	AFA/MPW	Approach	No. Models	Area Modelled (km^2)	Length Modelled (km)	Upstream Limit	Downstream Limit
I23DY	Dunmanway AFA and Upper Bandon MPW	1D/2D ISIS/TUFLOW into 1D ISIS	1	7.4	37.4	124000,055980	145196,054676
I25IN	Inishannon AFA and Lower Bandon MPW	1D/2D ISIS/ TUFLOW into 1D ISIS	1	0.9	23.6	155610,057492 152916,057173	165580,047350
I28SL	Schull AFA	1D/2D ESTRY/ TUFLOW	1	1.0	1.6	091957,031814	092852,031383

Modelling of AFAs

A hydrodynamically linked one-dimensional (1D) and two-dimensional (2D) approach has been taken for Dunmanway, Inishannon and Schull AFAs. The HPWs in Dunmanway and Inishannon have been modelled in ISIS one-dimensional modelling software (version 3.6.0) to simulate in-bank flows as ISIS is capable of accurately calculating conveyance, attenuation and head loss at structures in narrow rivers.

However, the ESTRY 1D software has been used to simulate the steep, culverted flows in Schull. This alternative software has been used to better model the supercritical shallow flow and pressurised flow through culverts on these reaches that can otherwise lead to inherent instability in the ISIS software. The reaches modelled in ESTRY have been directly linked to the 2D TUFLOW model of the floodplain to simulate flood hazard in the AFAs.

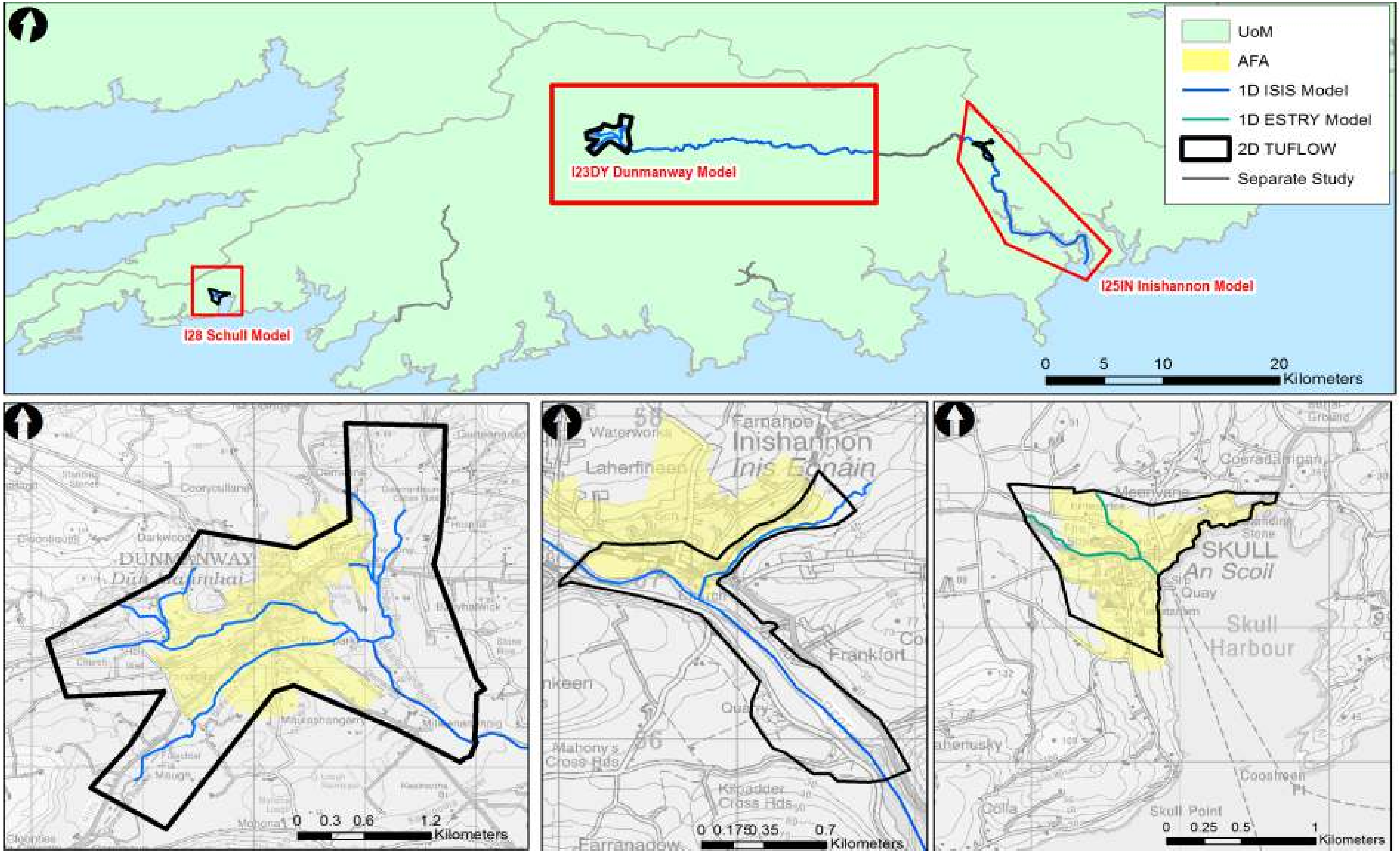
TUFLOW two-dimensional modelling software (version 2013-AC) has been used to model the floodplains in all the AFAs in order to simulate complex flow paths and variable velocities across the urban floodplains. The 2D approach is also the most appropriate to simulate complex flows in urban areas along the streets and over embankment such as found in Dunmanway.

Modelling of MPWs

The MPW reaches have been modelled using ISIS to simulate both in-bank and out-of-bank flows by extending the river sections across the floodplain. In order to improve hydrological routing and simplify modelling the Bandon MPW upstream of Bandon Town has been modelled with Dunmanway AFA(I23DY); and the Bandon MPW downstream of Bandon Town has been modelling with Innishannon AFA (I25IN).

A full geoschematic of each model is provided in the Appendices of this report, along with proformas detailing the model build assumptions, run parameters, model performance and flood maps.

Map 4.1: Model Approach



4.2 River Channels

The 1D model components were developed to simulate in-bank flows between the left and right river banks. The river channel survey data was used to inform the river cross-sections in ISIS and ESTRY. The raw survey data did not require correction for the majority of sections in UoM20. However, the following modifications were made during the modelling process for open channel Sections:

- River sections were extended to cover the complex loop channels on the Bandon. This avoids instability associated with significant flow rapidly changing between the 1D and 2D domains across the islands, and is representative of extreme water levels for high flow conditions.
- Additional river channel sections have been interpolated for the steep Innishannon Stream between the upstream and downstream bridge in the town. This improves the interpolation of flows over the steep channel bed and stability of the model overall.

The river channel gradient, width and shape can vary rapidly on the approach and exit of bridges, which is not necessarily representative of the typical open channel reach. Therefore, the surveyed sections observed 20m upstream and downstream of bridges tended to be used to inform the open channel modelled upstream and downstream of bridges because these survey sections tended to be more representative of the typical reach.

Resistance to flow from varying surface roughness across the river channel was represented by various Manning's 'n' values based on the material type and vegetation density (Table 4.2). The material types were assigned based on the survey data, photographs and site observations. The selection of the Manning's 'n' value was guided by the industry standard value ranges (Chow 1959), and subsequently adjusted during the calibration process where data was available. The selected Manning's 'n' values for each model are summarised in the model build proformas and in the model section data.

Table 4.2: Summary of Channel Manning's 'n' Values

Material Type	Selected Manning's 'n'	Applicable Reaches
Active river bed with gravel to boulders	0.045 to 0.050	River Bandon (Dunmanway)
Active river bed with silts	0.040 to 0.045	River Bandon (Lower) Dirty River Brewery Stream Inishannon Stream Schull Stream Meevane Stream
Light brush and/or grass during winter	0.060 to 0.075	River Bandon (Lower) Dirty River Brewery Stream Inishannon Stream Schull Stream Meevane Stream
Dense vegetation year round	0.075 to 0.080	River Bandon (Dunmanway)

Source: Chow 1959

Dunmanway Lake has been represented as a point inflow to the lake in the 2D domain with the downstream channel and structures represented as…

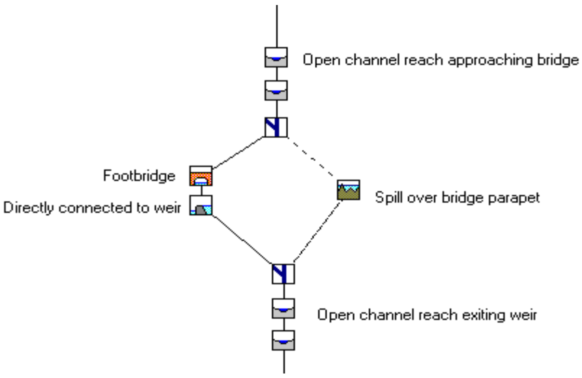
4.3 Structures

The surveyed structure dimensions were used to conceptualise bridges, culverts and weirs to simulate the hydraulic controls and flow paths that modify flood risk in the AFAs. The conceptualisation sought to reduce complex structures to the simplest schematisation that accurately represented the hydraulic mechanisms at the target flows, whilst maintaining model stability and robustness.

For example, many bridges in the South West Region have a plinth extending a short distance from the downstream face which causes a hydraulic jump similar to a weir at low flows (Figure 4.1a). The short open channel reach between the bridge and the weir is likely to cause instability at high flows, as the reach is so much shorter than the other reaches in the 1D model and connection to the 2D model may cause recirculation of water. Therefore, the model is simplified to the configuration in Figure 4.1b, which maintains the weir as the level control at low flows but avoids instabilities at high flow.

Figure 4.1: Simplification of Kanturk Footbridge and Weir

A: Kanturk Footbridge with Weir 2m downstream



B: Simplified Model Configuration

The simplification of structures in UoM20 is discussed in the following sections. There were no operable structures within the UoM20 AFAs. Full details of the hydraulic parameters and justification of structure specific assumptions can be found in Schedule 2 of the Model Build Proformas in the relevant appendices.

Bridges

Bridges were modelled in three ways in the SW CFRAM Study:

- Using the USBPR approach where the bridge was a flat soffit highways bridge and the afflux was largely controlled by the flow around the piers and a spill over the deck to consider high flow routes.
- Using the HR Wallingford arched bridge approach where the bridge was arched and the afflux was largely controlled by the flow under the arch above springing point and a spill over the deck to consider high flow routes.
- Using a Bernoulli head loss unit based on the calculated head loss with the effects of piers, skew, eccentricity and other hydraulic losses.

The first two approaches were applied in UoM20. The Schull bridges modelled in ESTRY used the USBPR approach available in the TUFLOW software which is appropriate for the flat soffit footbridges across the channel. No bridges were modelling using the Bernoulli Loss approach in UoM20.The main bridge opening at Long Bridge, Dunmanway was modelled using the HR Wallingford arched bridge approach. However, the parallel flood relief culverts were estimated from site survey and modelled as 1D ESTRY culverts embedded in the 2D domain, such that the flood relief culverts could be dry and flood at different points.

In Dunmanway, a number of bridges on the Brewery River and loop channel were modelled as orifices because the bridge opening was relatively small compared to the open channel section, and the bridge remained in orifice conditions for all flows. Orifice units were appropriate for these short small openings and improved model stability. In addition, for the leftmost arch at Killbarry Road Bridge, which is separate to the rest of the bridge. The modular limit was adjusted to match drowned and orifice flow at mode change.

Culverts

For the SW CFRM Study in general, culverts modelled in ISIS use; i) a culvert inlet to simulate losses associated with the constriction of flow at the entrance, ii) an appropriately sized and shaped conduit unit, and iii) a culvert outlet to simulate losses associated with the expansion of flow at the exit, or a weir unit to simulate the bed drop for culverts out-falling above the downstream river water level. However, there were no online culverts modelled in ISIS within the Dunmanway and Inishannon models.

Culverts modelled in ESTRY for the Schull model were based on the survey structure dimensions and contraction and expansion losses calculated using the recommended coefficients from Capacity Charts for the Hydraulic Design of Highway Culverts (Henderson, 1996). Manning's 'n' values of 0.025 were typically used to represent the rougher brick material in these culverts, and 0.015 for smooth concrete walls. It is not within the CFRAM scope to assess urban drainage. However, flood water has been observed to originate from a manhole by the Bunratty Inn and escape up through the paved over garden at Carbery House attached to Schull Stream under the High Street. There was no survey data available for the manhole dimensions so these have been estimated from site visits. A fixed energy loss through the manhole was applied based on the typical K value for the manhole type and the velocity in the culvert, and calibrated to achieve the observed flooding in Schull.

Losses associated with trash screens have been considered as part of the inlet coefficients for both ISIS and ESTRY. The trash screens have been assumed to be clear in accordance with the design scenario defined by OPW. Blockage of such structures will be considered separately as part of the option development process.

Weirs

Formal weir structures such as those found in Schull, and other informal weirs/natural bed drops such as the steep changes in bed across Woodhock House Bridge (Dirty River, Dunmanway) have been modelled using weir and online spill approaches. In both cases, the river sections have been extracted 20m upstream and downstream of the weir structure based on the surveyed weir long profile to adjust the bed

levels and better represent the upstream and downstream open channel reaches. The surveyed weir crest was then used to inform the width and elevation in the formal round-nose weir structures and the spill elevations for informal structures. This approach ensures the weir or spill crest forms the hydraulic control and the localised scour pool effects are removed. Where the defined weir crest is narrower than the river channel width, online spills have been used to represent flow over the banks, with calibrated coefficients to simulate the effects of bank vegetation.

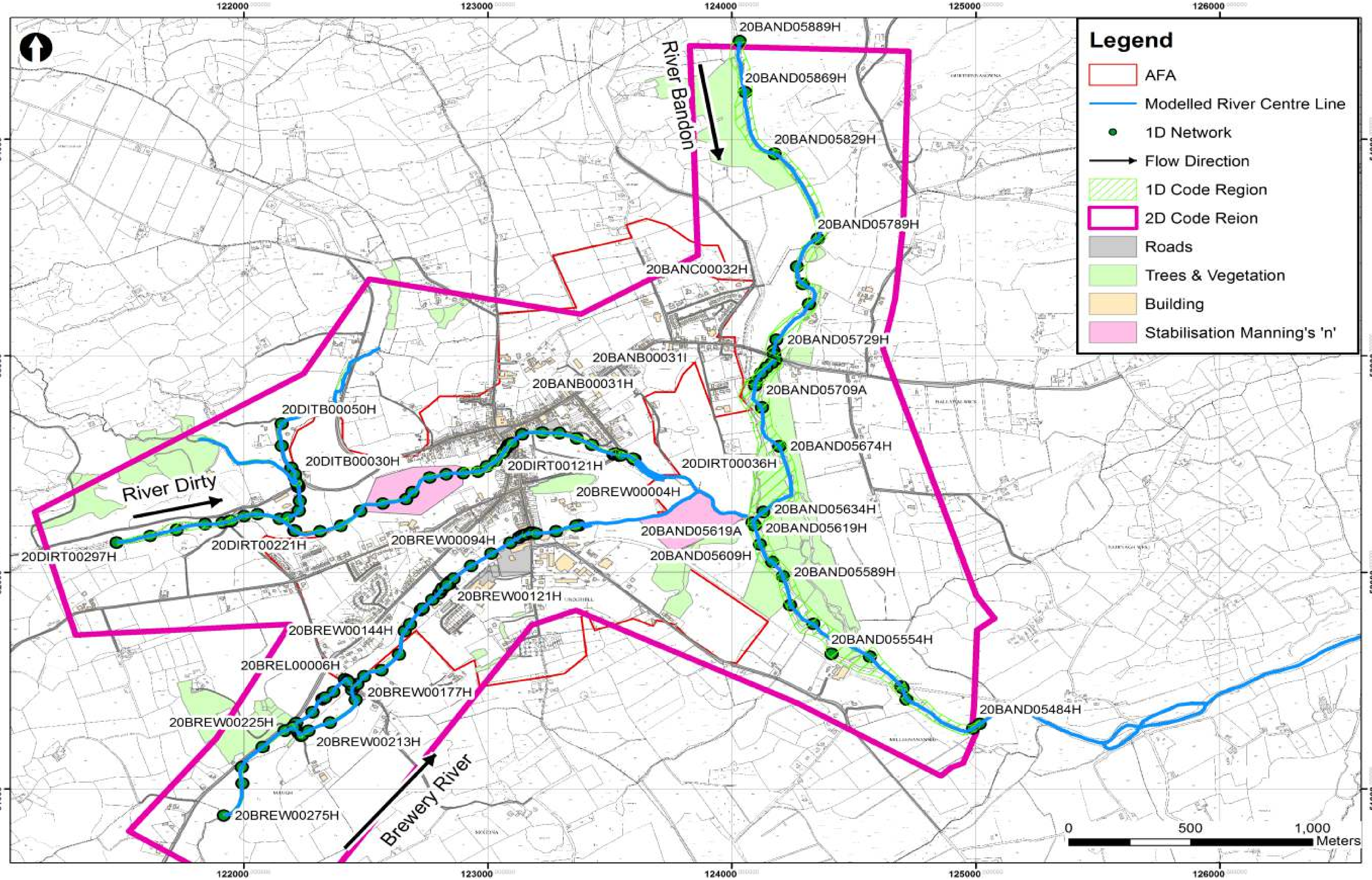
4.4 Floodplain

The floodplain in all the AFAs was represented by a regular 5m grid orientated to be perpendicular to the dominant flow path. A 5m grid cell size was selected to optimise the run time whilst adequately representing the complex urban nature of these AFAs. Map 4.2 presents an example for Dunmanway.

Floodplain Topography

The 2D topography was extracted from the LiDAR DTMs. The 5m grid resolution does limit the representation of small and thin urban features. Therefore, key floodplain features that would modify flow paths have been explicitly represented in the 2D domain. This includes raised barriers to flow, such as road and rail embankments, as well as flow routes such as drainage ditches and archways through buildings. The elevations for these features have been extracted from the LiDAR data and enforced in the 2D domain using the “Z-line” option. Thin features, such as fences and garden walls, have not been considered, as they cannot be guaranteed to retain water during a flood event where they are not designed as flood defences.

Map 4.2: Example Geoschematic of Dunmanway



Urban Features

Buildings within the floodplain were represented as footprints with a threshold level of 150mm above ground level extracted from the DTM. The threshold of 150mm was selected as typical from threshold surveys and survey photographs. Once out-of-bank, flood extents are largely determined by the narrow valley topography, and the raised building footprint does not significantly alter the floodplain capacity. Therefore, the threshold value selected does not significantly affect the flood risk and extent in the AFA. The buildings were assigned a Manning's 'n' value of 0.2 to simulate the reduction in flow and velocity through the buildings once water was above the threshold value of 0.15m.

This combined approach using threshold and raised Manning's 'n' is the most appropriate approach to enable the extraction of realistic flood depths at each property which is required during the Flood Impact Damage Assessment to be undertaken at a later stage. Other approaches which block out the buildings do not provide the required flood depths at the property points, and therefore are not appropriate.

This approach assumes water is able to flow through the buildings which might otherwise be diverted if the building was made watertight such as from the use of sandbags or individual property protection measures. The use of individual protection property measures has been considered when assessing historic flood extents.

The roads in UoM20 are typically 8m wide, and are neither significantly raised above nor sunken below the floodplain. Therefore, the model grid topography was deemed to represent the flow paths of the roads without further modification to the model topography. Instead, a lower Manning's 'n' of 0.03 was used to represent the relatively lower resistance to flow of the tarmac. This approach enforces the roads as flow paths across the floodplain to better model flood progression.

Land Cover

The floodplain was classified into broad land use types from the survey information, photographs of the river banks, site observations and OSI mapping. The European Environment Agency CORINE land cover dataset was not used because the data is based on satellite imagery which is relatively coarse and does not differentiate buildings from surrounding roads and gardens within urban areas.

Each land classification from the OSI mapping was then assigned an appropriate Manning's 'n' roughness value based on the type and density of the vegetation, guided by industry standard value ranges (Chow 1959). Small urban features, such as fences and walls, have not been considered explicitly as they are not designed to retain water during a flood event. However, the overall impact of these features has been incorporated into the selection of the upper range of recommended floodplain Manning's 'n'. Table 4.3 summarises the design values selected. Sensitivity tests on Manning's 'n' values are discussed in Section 5.2.3.

Table 4.3: Floodplain Roughness Values

Surface	Manning's 'n' Roughness Value
Standing water	0.040 to 0.050
River Banks - Dense Vegetation	0.075 to 0.085
Buildings	0.200
Roads and Hard Standing	0.030
Pasture, Parklands and Gardens	0.060

4.5 Model Run Parameters

The design models were run for the full inflow hydrograph duration to consider attenuation and the recession of any flooding in each AFA.

Initial river flow and level conditions were derived at every river section along the entire modelled reach for the 1D model components to match the start of the hydrograph for the current scenario, as well as the mid-range and high-end future scenarios. The minimum flows used to derive the initial conditions and lower limit of model stability are stated for each model reach in the model proformas included in the Appendices.

The initial coastal conditions were set to start at low water and below the floodplain level to ensure the river channel and floodplain represented pre-flood conditions and the 2D model was stable.

A 1D timestep interval of one second was applied to all the UoM20 models to ensure stability along the steep tributaries and to be divisible into the 2D timestep. A 2D timestep of two seconds was applied to all models to be divisible by the 1D timestep and within the recommended a half to a quarter of the 2D cell size.

All other run parameters were set to default both in ISIS and TUFLOW. The river sections were extended in the 1D only reaches to avoid "glass-walling" of water above the limit of the cross-section. Hence the height added to the maximum section elevation (Dflood) was set to the default value of 3m.

5 Calibration and Sensitivity Analysis

5.1 Calibration

Table 5.1 outlines the historic flood events selected for the calibration of the hydraulic models during the hydrological analysis. The selection of historic events was based on scoring the flow estimates, observed data and reliable flood history as set out in Guidance Note 23[2].

Table 5.1: Selection of Calibration Events

Event	Model	Likely Accuracy of Flow Estimate[1]	Likely Accuracy of Gauged Level Estimate	Known Hydraulic Conditions[2]	Likely Accuracy of Spot Levels[3]	Reliable Flood History[4]	Indicative Calibration Score	Calibration Approach
01/01/1991	Dunmanway	3	3	1	0	1	8	Significant catchment changes in urban area channel bed levels as evidenced by the changes in gauge datum since the event makes calibration less certain. Otherwise calibrate main channels.
12/10/1996	Dunmanway	3	3	1	0	1	8	Significant catchment changes since event makes calibration less certain. Otherwise calibrate main channels.
06/10/2009	Schull	1	0	3	0	2	6	Validate flood extent and mechanisms due to lack of spot levels.
19/11/2009	Dunmanway Inishannon	3	3	2	1	3	12	Entire catchment calibration available. Calibrate main channel to gauge data and flood extent. Validation only at Innishannon with flood extent.
15/08/2012	Schull	1	0	3	3	3	9	Calibrate flood extent and mechanisms. Recorded depths are only accurate to within +/- 0.1m.

Note 1: 3 = gauged flows are available in the catchment, 2 = gauged flows used from pivotal gauges nearby, 1 = rainfall data used to estimate flows using rainfall-runoff methodology and 0= no flow estimate available
Note 2: Hydraulic conditions relate to controls on water levels during a flood e.g. level of blockage, wall collapse etc.
Note 3 Levels during a known flood event NOT at a gauged location that represents a true flood level rather than a localised issue.
Note 4: Any information that includes date/time, precise location and mechanism of flooding

[2] Jacobs, (January 2013) Guidance Note 23 Model Calibration. Version 1.

The following events have been considered for the calibration of the Bandon catchment (excluding Bandon town) based on the indicative calibration score:

- 1st January 1991 – Less than the index flood at Dunmanway;
- 12th October 1996 – Extreme fluvial event at Dunmanway and along the River Bandon to Bandon Town;
- 19th November 2009 – Extreme fluvial event at Dunmanway, Bandon MPW and Inishannon.

The following events have been considered for the hydraulic calibration of the Schull catchment based on the indicative calibration score:

- 6th October 2009 - Extreme flash flood event in Schull;
- 15th August 2012 – Extreme flash flood event in Schull.

Reports of recurring flooding and information from local engineers were also used to verify the modelled outlines such that there is “reasonable” representation of the historical flood frequency. Comments on representation of the historical flood frequency are also provided in the model performance proformas of each Appendix.

5.1.1 1st January 1991

Water levels were observed during a high flow event on 1st January 1991. Flooding began after overtopping on the River Bandon at Long Bridge, and flood waters overflowed onto Macroom Road towards Dunmanway, being diverted from town into Dunmanway Lake, therefore missing the properties on Main Street.[3]

The quality of the historic flood data has been reviewed based on the Flood Alleviation Scheme Report:

- Photographs and Extent
  - No photographs were available from the original report.
  - The report indicates that there was no property flooding and surface water runoff was diverted into Dunmanway Lake watercourse.
  - This was before the implementation of the current Flood Alleviation Scheme
- Levels and flow
  - Water levels were taken at Long Bridge and Bealaboy gauges during the event and combined with with gauged levels.
  - The rating developed by this CFRAM Study was then used to convert these levels to a flow hydrograph.

The design hydraulic model was modified as follows to represent the hydrological conditions of this event:

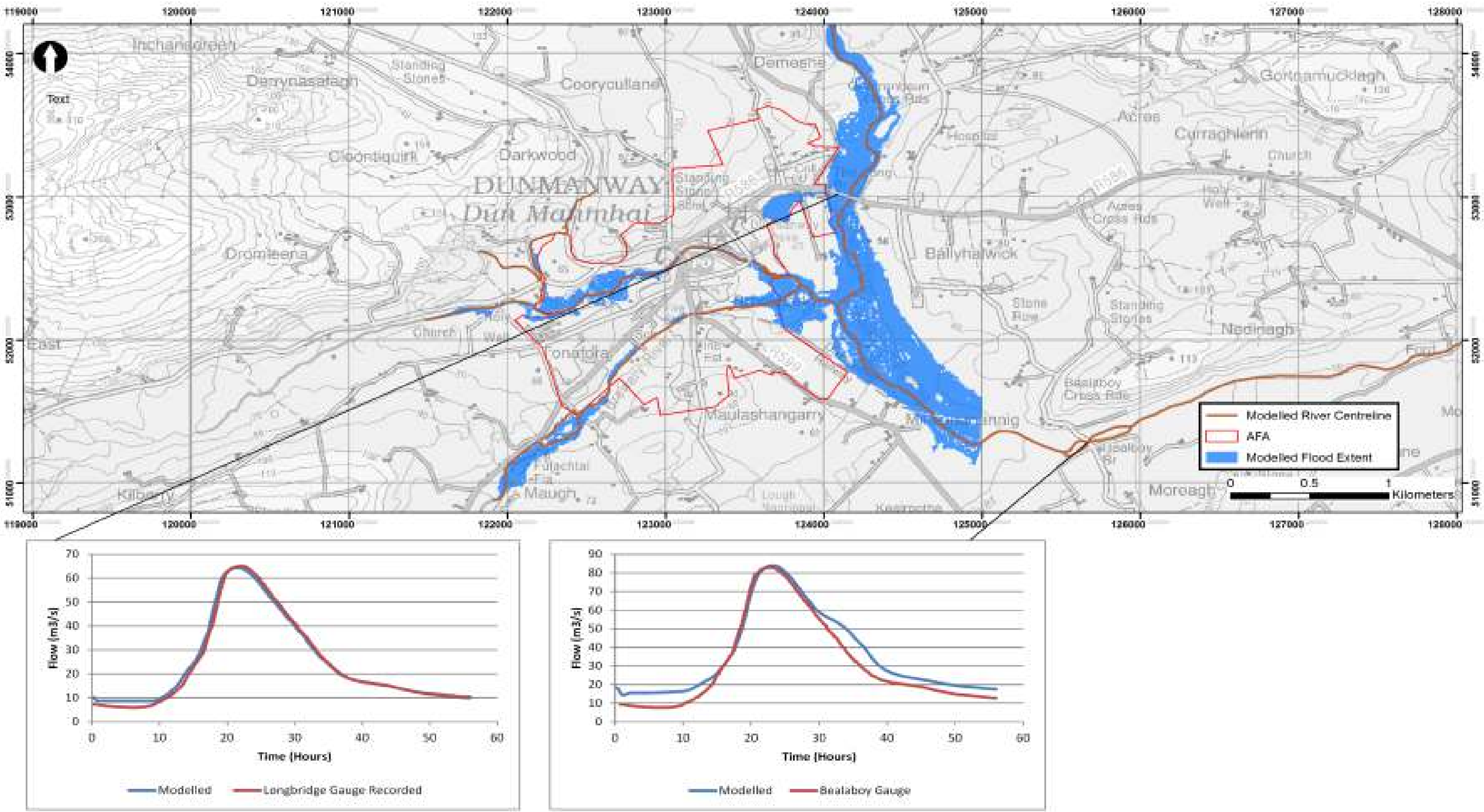
- The gauged level was converted to a flow hydrograph at Long Bridge to form the Bandon inflow.
- The gauged level was converted to a flow hydrograph at Bealaboy to form the target flow.
- The Dirty, Brewery and Dunmanway Lake inflows were scaled to achieve the target flow at Bealaboy.

The hydraulic parameters were adjusted to best match the flood levels in Dunmanway, including: Manning's 'n' and the loss coefficients through Long Bridge. Map 5.1 compares the resultant model extent and levels with the recorded information.

The peak flows were found to be within 2% of the gauged peak flows, and the modelled flood hydrographs matched well with the duration of flooding observed. The model extent and depths match well with reported flooding in 1991, flooding the right bank of the Bandon upstream of Long Bridge, but not affecting any of the properties along the Main Street..

[3] OPW Engineering Services (1991) Dunmanway Flood Alleviation Scheme

Map 5.1: Calibration of Dunmanway to 1 January 1991 Event



Location	Gauged Flow (m3/s)	Modelled Flow (m3/s)	% Difference	Gauged Level (m)	Modelled Level (m)	Difference (m)
Long Bridge	65	64	-1%	57.6	57.5	-0.1
Bealaboy	83	84	2%	53.3	53.3	0.0

5.1.2 12th October 1996

High flows on the Bandon prevented discharge from Dunmanway Lake which consequently raised water levels in the lake and flooded more than 20 properties along Chapel Street. Photographs obtained during the Dunmanway Drainage Scheme Environmental Impact Statement[4] show high levels in Dunmanway (Chapel) Lake, fields flooding around Long Bridge and flooding across Macroom Road. It should be noted that these photographs were after the flood peak and do not necessarily show the maximum flood extent.

The quality of the historic flood data from the post flood report [5] has been reviewed:

- Photographs and Extent
  - Photographs obtained during the Dunmanway Drainage Scheme Environmental Impact Statement[4] show high levels in Dunmanway (Chapel) Lake, fields flooding around Long Bridge and flooding across Macroom Road
- Levels and flows
  - Water levels were taken at Long Bridge and Bealaboy gauges during the event and combined with with gauged levels.
  - The rating developed by this CFRAM Study was then used to convert these levels to a flow hydrograph.

The design hydraulic model was modified as follows to represent the hydrological and hydraulic conditions of this event:

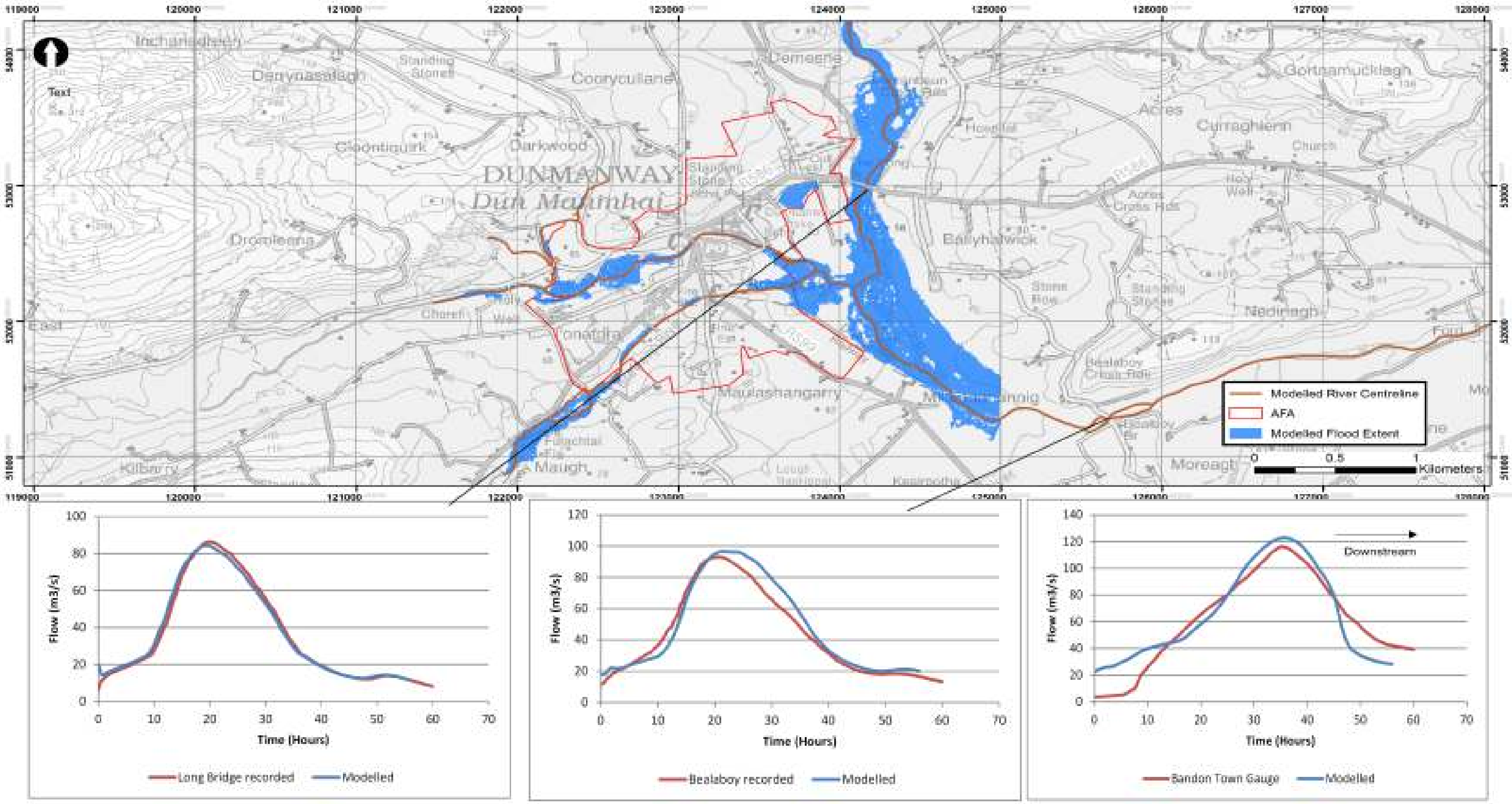
- The gauged level was converted to a flow hydrograph at Long Bridge to form the Bandon inflow.
- The gauged level was converted to a flow hydrograph at Bealaboy to form the target flow.
- The Dirty, Brewery and Dunmanway Lake inflows were scaled to achieve the target flow at Bealaboy.

The hydraulic parameters were adjusted to best match the flood levels and extents in Dunmanway, including: Manning's 'n' and the loss coefficients through Long Bridge. Map 5.2 compares the resultant model extent and levels with the recorded information.
The peak flows were found to be within 4% of the gauged peak flows in Dunmanway, and 6% at the outflow of the model to Bandon Town. The flood hydrograph matches well at Long Bridge. The falling limb is slightly overestimated at Bealaboy due to hydrological assumptions on tributaries which results in a slightly greater flow at Bandon Town. However, the flood volumes and flood extent are similar to observed. The model extent and depths match well with reported flooding in 1996, flooding the right bank of the Bandon upstream of Long Bridge as far as Macroom Road. The modelled flood levels and extent associated with Dunmanway Lake are in agreement with the majority of the reported property flooding. along Chapel Street. However the additional flooding from surface water flooding is not considered in the CFRAM Study model.

[4] RPS (1997) Bandon River Dunmanway Drainage Scheme EIS.

[5] Photographs provide by Cork County Council in conjunction with reference 4 above.

Map 5.2: Calibration of Dunmanway to 12 October 1996 Event



Location	Gauged Flow (m3/s)	Modelled Flow (m3/s)	% Difference	Gauged Level (m)	Modelled Level (m)	Difference (m)
Long Bridge	86	84	-2%	57.6	57.6	0.0
Bealaboy	93	97	4%	53.6	53.5	-0.1
Bandon Town	116	123	6%	N/A	N/A	N/A

5.1.3 6th October 2009

Flash flooding occurred in Schull on the 6th October 2009, inundating shops and businesses along Main Street within 2 hours. A total of 54mm fell within nine hours with newspapers reporting "torrential rain from 10:00 to 12:00 as a result of tropical air mass meeting a polar air mass over Ireland" [6]. The Schull Stream spilled out-of-bank at the Main Street Bridge, before flowing down the road flooding many properties. The newspaper articles suggested similar flooding had occurred once before over the previous 20 years, but no date was specified.

The quality of the historic flood data from the newspaper articles has been reviewed:

- Photographs and Extent
  - Photographs were extracted from the newspaper articles. These are deemed reasonable indicators of areas flooded and mechanisms of flooding.
- Levels
  - Flood depths are difficult to estimate from the available photographs in the newspaper articles, although it is evident flood depths were between 0.2m at the upstream of the High Street and >0.6m at the downstream of the High Street to flood the properties and vehicles shown.

There was no river flow or level data recorded in Schull or at the closest gauge, Ballyhilty (20005), for this event. Therefore, the following steps were undertaken to derive the hydrographs for the ungauged HEPs in the Schull catchment:

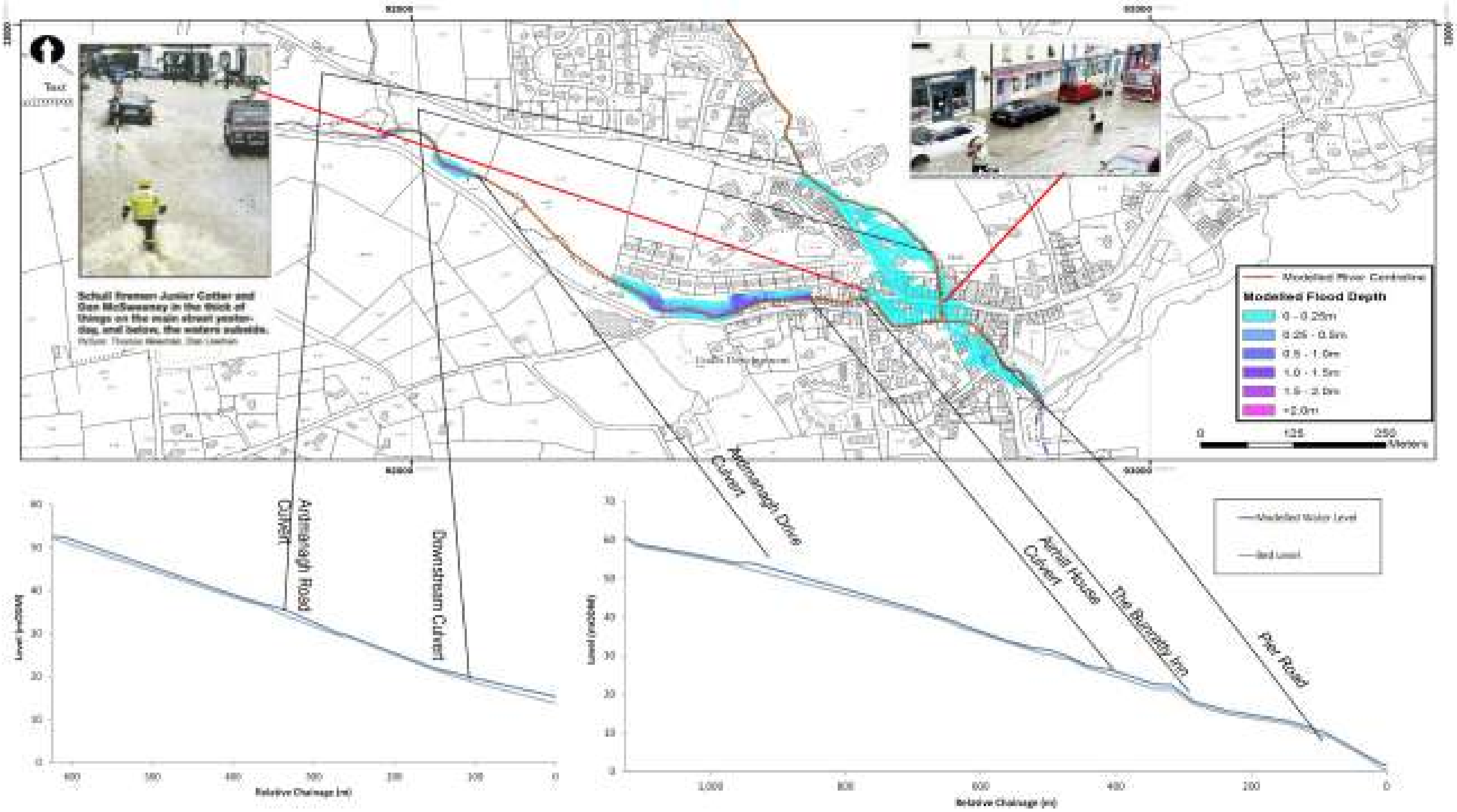
- Transfer the representative rainfall profile from the hourly data at Cork Airport and scale to meet the total rainfall recorded for the Schull area.
- Calibrate the FSSR16 catchment average rainfall parameters for the gauged catchment from Met Eireann DDF results, recorded soil moisture deficit conditions and physical catchment descriptors.
- Apply the transferred rainfall profile and calibrated SPR of 72% to estimate the flow hydrograph at Schull.

The hydraulic parameters were adjusted to best match the flood levels and extents in Schull, including: Manning's 'n' and the manhole loss coefficients under the High Street. Map 5.3 compares the resultant model extent and levels with the recorded information.

The model predicted flow down Ardmanagh Road and out of the manholes to flood the High Street as pictured in the newspaper articles. The modelled flood extent matches well with the areas reported to have flooded. Flood depths matched well with the 0.2m estimate at the upstream of the High Street, but underestimated flood depths at the downstream of the High Street. This local underestimation of depth is because the hydraulic model does not consider the additional surface water flooding and the presence of sandbags at these properties was not considered. I. Otherwise, the Schull model is deemed to match well with the October 2009 flood reports.

[6] Bray, A. and McDonagh, M. (2009) 'Flash flooding brings village to a standstill as river bursts its banks', *Irish Independent*, 7 October 2009 p6.

Map 5.3: Calibration of Schull to 6 October 2009 Event



5.1.4 19th November 2009

Persistent rainfall over the preceding 10 days and heavy rainfall on the 19th November 2009 led to flooding in Innishannon, Bandon and Dunmanway. At Dunmanway, the River Bandon burst its banks, flooding three residential properties at Manch Bridge(downstream), but remained within the flood alleviation scheme within the AFA. Information from the Bandon Flood Relief Scheme public information day indicated that properties along Main Street in Innishannon were flooded. An interview with a local business owner during the Flood Risk Review indicated that the river level reached the back of the gallery building on the south side of Main Street as shown by the video footage.

The quality of the historic flood data has been reviewed from the OPW flood reports and aerial survey data:

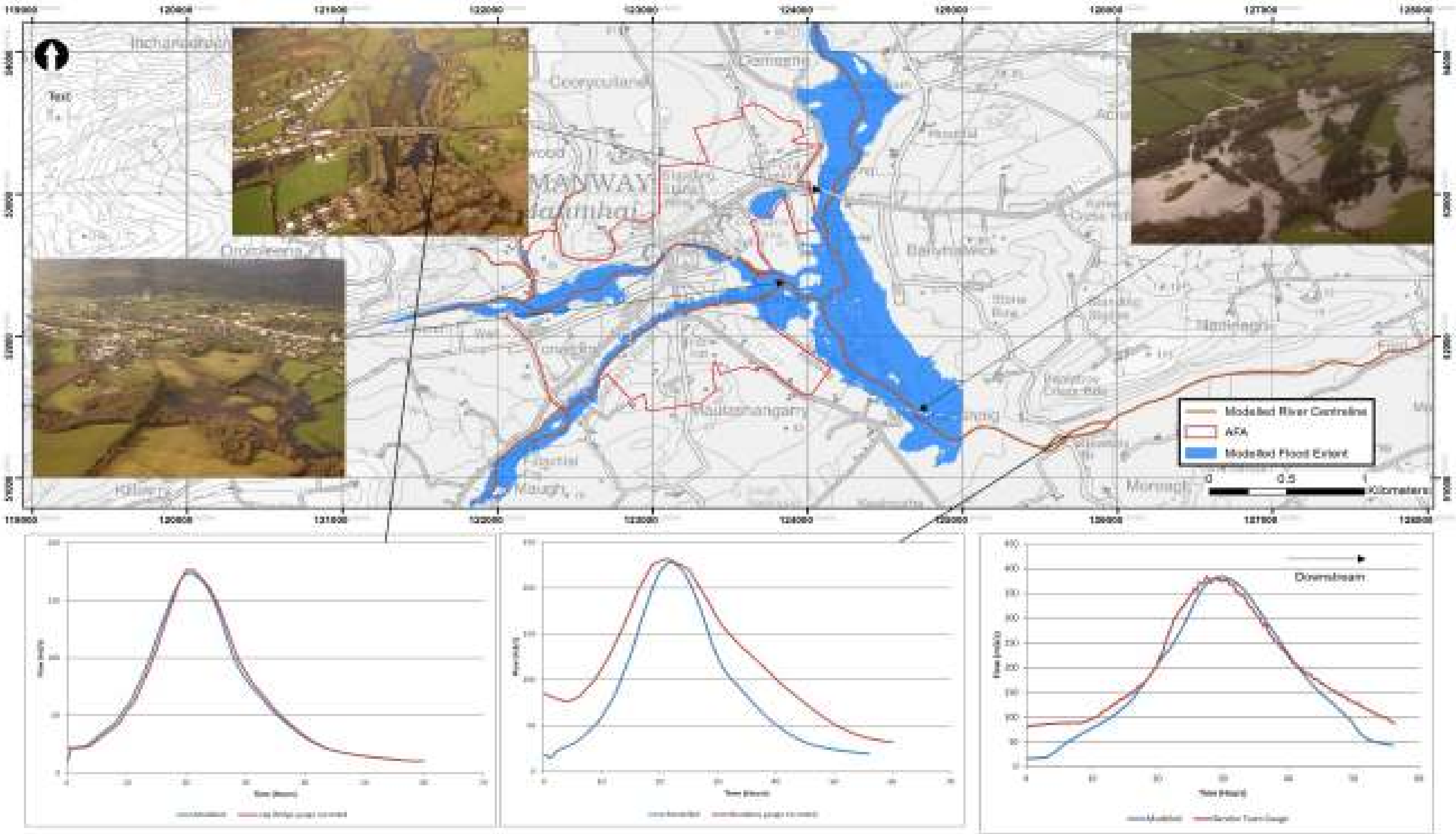
- Photographs and Aerial Footage of Extent
  - No photographs were available in Dunmanway and Innishannon for this event.
  - However, aerial photographs and video footage were available for the Bandon between Innishannon and Dunmanway which was flown at 13:00 on 20 November 2009.
  - Therefore the video footage is deemed representative of the areas flood but not necessarily the maximum flood extent.
- Flood Levels and Flows
  - Gauged levels were available at Long Bridge and Bealaboy gauge which have been used to calibrate the model.
  - The model cannot be calibrated to levels at Bandon Town gauge as the Dunmanway model does not extend to the gauge site in Bandon Town.
  - The rating developed by this CFRAM Study was then used to convert these levels at Long Bridge and Bealaboy to flow hydrographs.
  - The gauged flows at Bandon Town and Curranure have been used to inform the Dunmanway and Innishannon model respectively.

The design hydraulic model was modified as follows to represent the hydrological and hydraulic conditions of this event:

- The gauged level was converted to a flow hydrograph at Long Bridge to form the Bandon inflow.
- The gauged flows at Bealaboy and Bandon Town formed the target flows.
- The Dirty, Brewery and Dunmanway Lake inflows were scaled to achieve the target flow at Bealaboy, and the lower tributaries were scaled to achieve the target flow at Bandon Town.
- The gauged flow at Curranure formed the inflow to the Innishannon model.
- The downstream tributaries were scaled to achieve the flood extent shown in the video footage.

The hydraulic parameters were adjusted to best match the flood levels, flows and extents in Dunmanway including Manning's 'n' values and the food relief culvert coefficients. Maps 5.4 and 5.5 compare the resultant model extent, flows and levels with the recorded information.

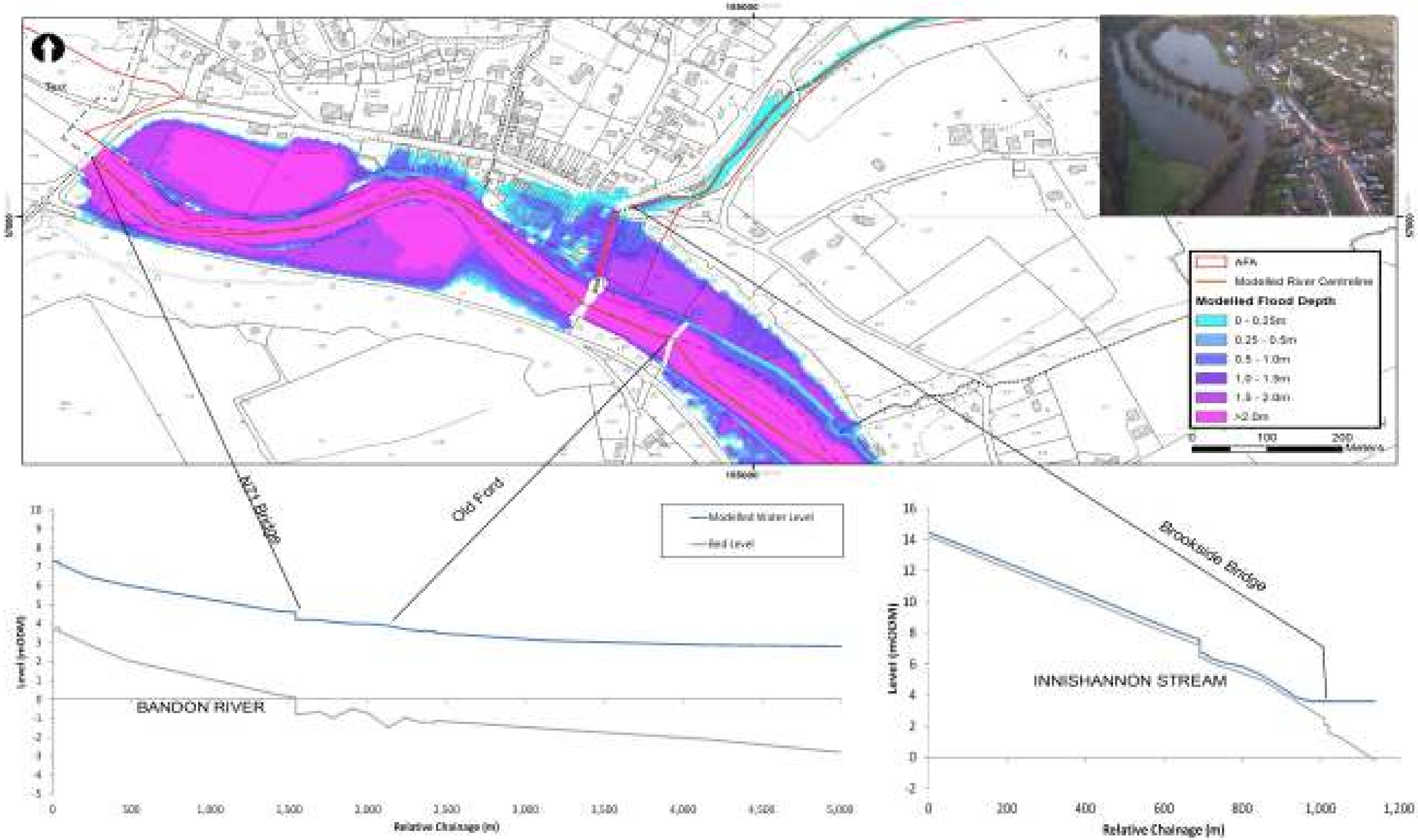
Map 5.4: Calibration of Dunmanway to 19 November 2009 Event



Location	Gauged Flow (m3/s)	Modelled Flow (m3/s)	% Difference	Gauged Level (m)	Modelled Level (m)	Difference (m)
Long Bridge	176	179	1%	58.1	58.0	-0.1
Bealaboy	298	328	-9%	54.3	54.6	0.3
Bandon Town	564	603	7%	N/A	N/A	N/A

Mott MacDonald

Map 5.5: Calibration of Innishannon to 19 November 2009 Event



 296235/IWE/CCW/R019/D June 2016
C:\Users\pig44561\AppData\Roaming\OpenText\OTEdit\EC_EUNAPiMS\c1500321612\296235-IWE-CCW-R019-D UoM20 Hydraulics Report.docx

In Dunmanway, the modelled flows were within 5% of the peak flow and 0.3m of the peak water level. The modelled flood extent matches well with the video footage showing the flood waters contained within the embankments at Long Bridge, flooding at the Dirty River confluence and flooding up to but not over the Old Rail Bridge. The duration of flooding is slightly underestimated at Bealaboy due to assumptions in the tributary inflows downstream of the town, however the flood extent and risk matches well with the flood evidence available.

In Innishannon, the modelled flood extent matches well with the video footage at the playing fields and at the back of Art Gallery on Main Street. The model predicted flood depths of 0.14m at properties south of Main Street, which agrees with depths of 0.1m reported during the Bandon Flood Relief Scheme public information day. Therefore, the Innishannon model calibrates well to the flood mechanisms and extent reported in the November 2009 event. It should be noted that flooding is also reported along Brookside from overland flow which is not considered by the CFRAM study.

5.1.5 15th August 2012

A period of rainfall throughout the night was followed by further intense rainfall during the morning of the 15th August 2012, leading to flooding in Schull. The Schull Stream reached its capacity and flooded a section of the Main Street due to surcharging of a manhole and the paved over channel at the Bunratty Inn. The maximum flood depth of 0.3m was reported on Main Street, resulting in the flooding of 19 properties.

The quality of the historic flood data from the flood event data collection report[7] has been reviewed:

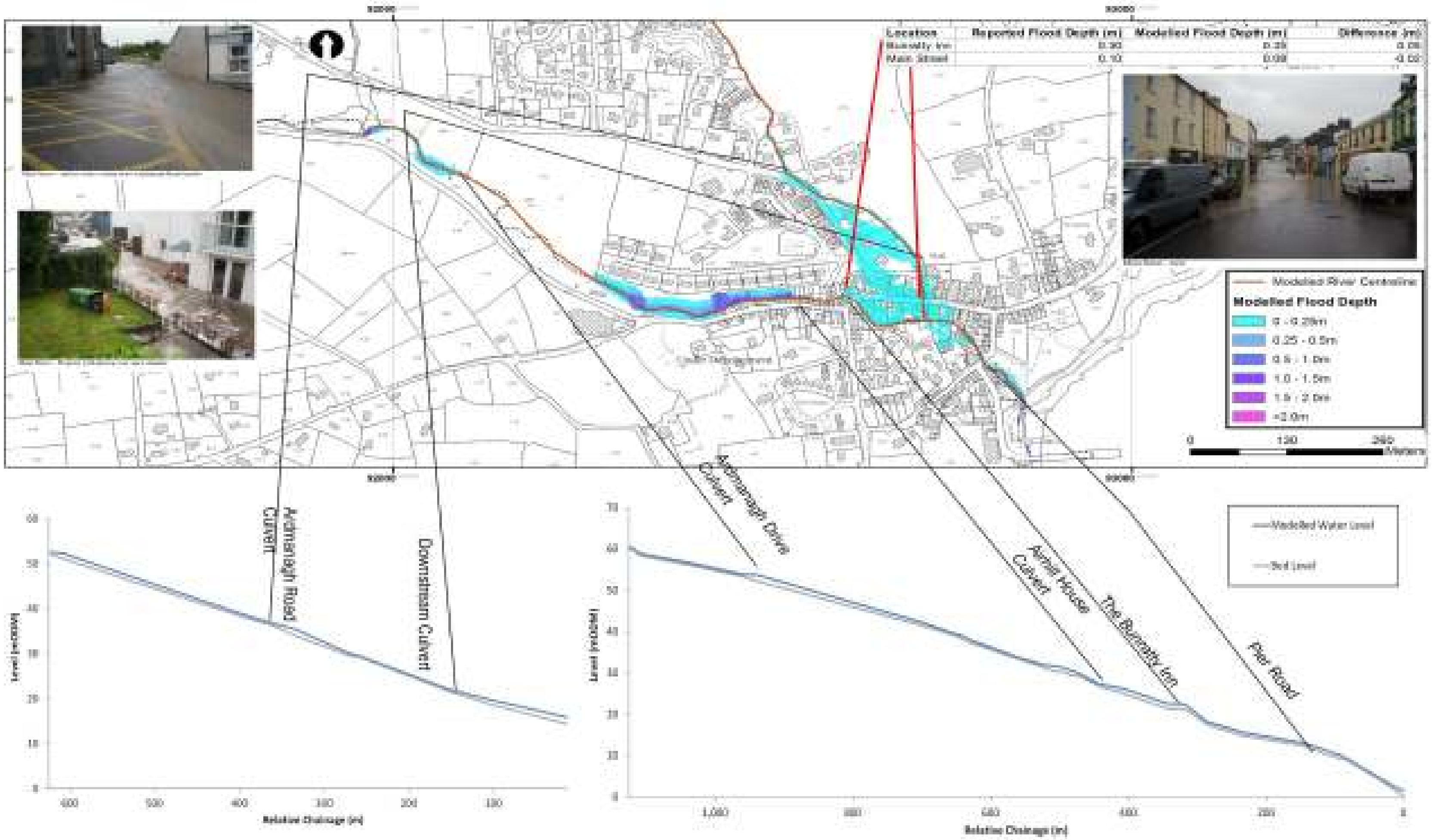
- Photographs and Extent
  - Photographs were collected from residents during the flood event data collection on 16th August 2012.These are deemed reasonable indicators of areas flooded and mechanisms of flooding.
- Levels
  - Flood depths were recorded during compilation of the flood event data collection report and verified with the photographs.
  - The flood depths are deemed to be reliable to 0.1m as these were recorded the day after the event when the impacts of flooding were still evident.

There was no river flow or level data recorded in Schull or at the closest gauge, Ballyhilty (20005), for this event. Therefore, the following steps were undertaken to derive the hydrographs for the ungauged HEPs in the Schull catchment:

- Transfer the representative rainfall profile from the hourly data at Sherkin Island and scale to meet the total rainfall recorded for the Schull area.
- Calibrate the FSSR16 catchment average rainfall parameters for the gauged catchment from Met Eireann DDF results, recorded soil moisture deficit conditions and physical catchment descriptors.
- Apply the transferred rainfall profile and calibrated SPR of 75% to estimate the flow hydrograph at Schull.

[7] Mott MacDonald(2012) Flood Event Data Collection Report – Schull 15.08.12

Map 5.6: Calibration of Schull to 15 August 2012 Event



The hydraulic parameters were adjusted to best match the flood levels and extents in Schull, including: Manning's 'n' and the manhole loss coefficients under the High Street. Map 5.6 compares the resultant model extent and levels with the recorded information.

The model predicted flow down Ardmanagh Road and out of the manholes to flood the High Street as pictured in the newspaper articles. The modelled flood extent matches well with the areas reported to flood. Flood depths matched well with the 0.2m estimate at the upstream of the High Street, but underestimated flood depths at the downstream of the High Street. This local underestimation of depth is because the hydraulic model does not consider the additional surface water flooding and the presence of sandbags at these properties was not considered. Otherwise, the Schull model is deemed to match well with the October 2009 flood reports

5.1.6 Summary

Table 5.2 summarises the calibration run performance, average difference from recorded levels, and tolerance of recorded levels for the three historic events simulated. The average error of the modelled flood levels were within the confidence limits of the recorded levels for the calibration events and matched well with the reported flood extents.

Table 5.2: Summary of Calibration Performance

Event	Reliability of Recorded Levels	Location	Average Error to Recorded Levels/Depths (m)	Root Mean Square Difference
01/01/1991	±0.1m (Gauged)	Dunmanway	-0.05	0.07
06/10/1996	±0.1m (Gauged)	Dunmanway	-0.05	0.07
06/10/2009	±0.25m (photos)	Schull	0.20	0.25
19/11/2009	±0.1m (Gauged) ±0.25m (Interviews)	Dunmanway Innishannon	0.20	0.22
15/08/2012	±0.1m (Survey)	Schull	0.02	0.04

5.2 Sensitivity Analysis

5.2.1 Flow

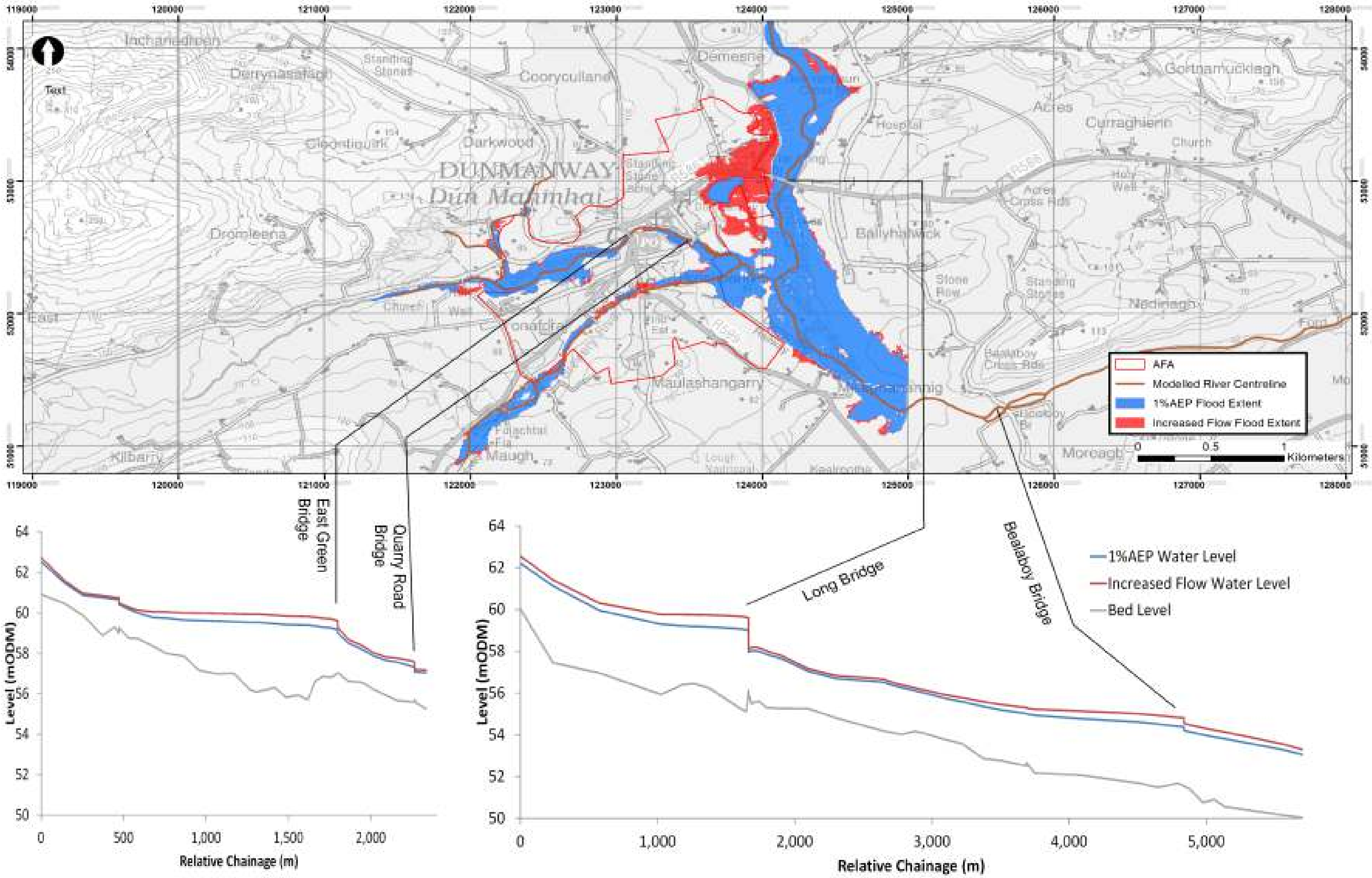
In accordance with CFRAM Guidance Note 22, the 1%AEP design peak flow was raised by 30% to assess the sensitivity to uncertainties in the QMEDrural coefficients, the selection of pivotal sites and the flood growth curves derived in the hydrological analysis. This is approximately equivalent to the flow increase applied to simulate climate change in the High End Future Scenario (HEFS), as the increase in flows due to urbanisation is less than 1%.

Flood level and extent was sensitive to uncertainties in flow in Dunmanway, Innishannon and Schull (Maps 5.7 to 5.9).

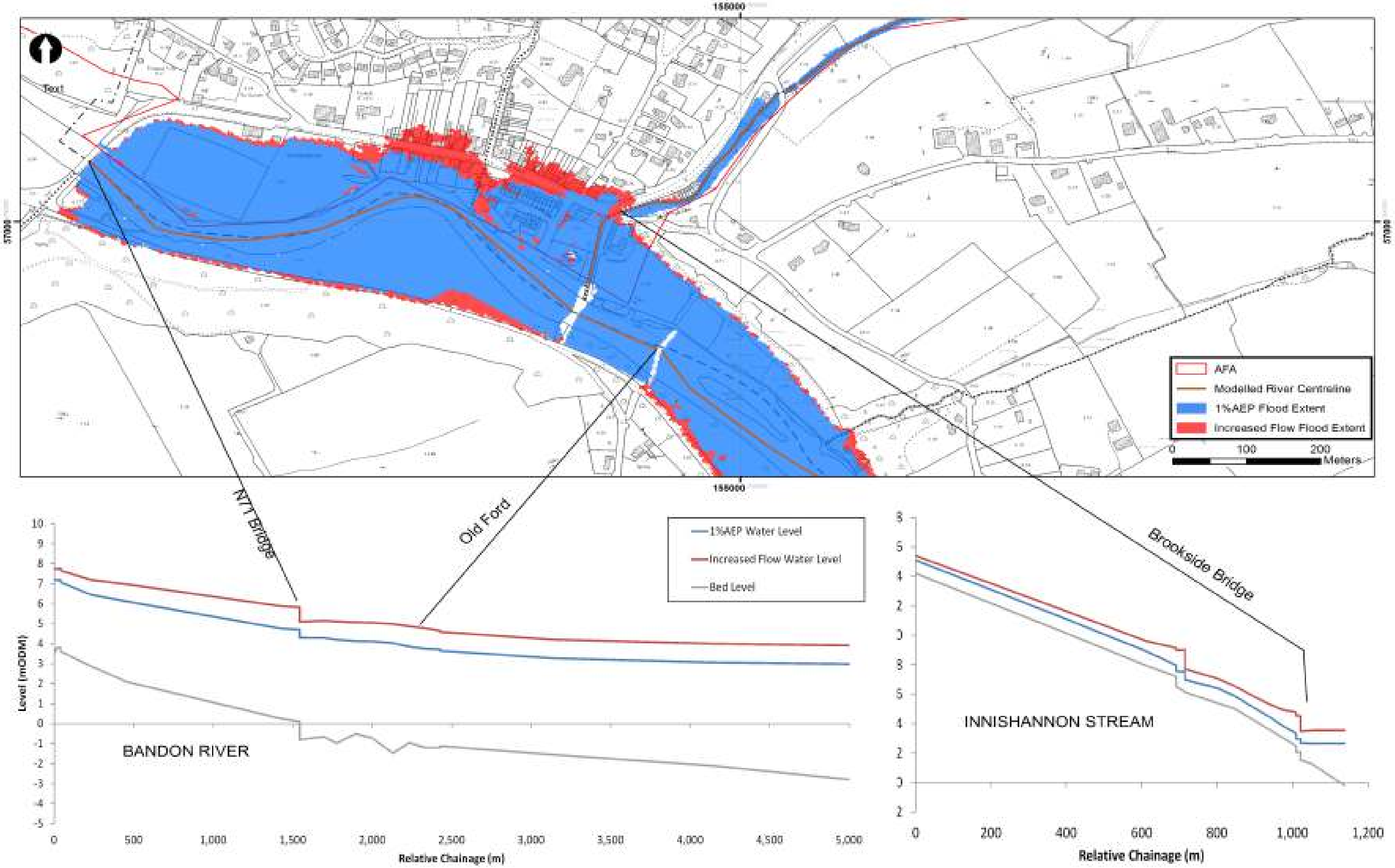
The increased flows exceeded the capacity of the Dunmanway flood alleviation scheme, overtopped the embankments and the Bandon flooded Dunmanway Lake area via Chapel Street. The increased peak flow also lead to greater flooding along the Brewery River at Underhill Commercial Park. In Innishannon, the increased flows raised water levels by over 0.8m to flood more of Main Street. However both these models use gauge records to estimate the design flows, so there is reasonable confidence in the design flows.

In Schull, flood levels increased by 0.4m which exceeded the capacity of the upstream culverts on Schull Stream and Meevane Stream. This resulted in greater flooding to fields upstream of Ardmanagh Drive and greater flooding to properties on the left bank of Meevane Stream flowing down the fields towards the High Street.

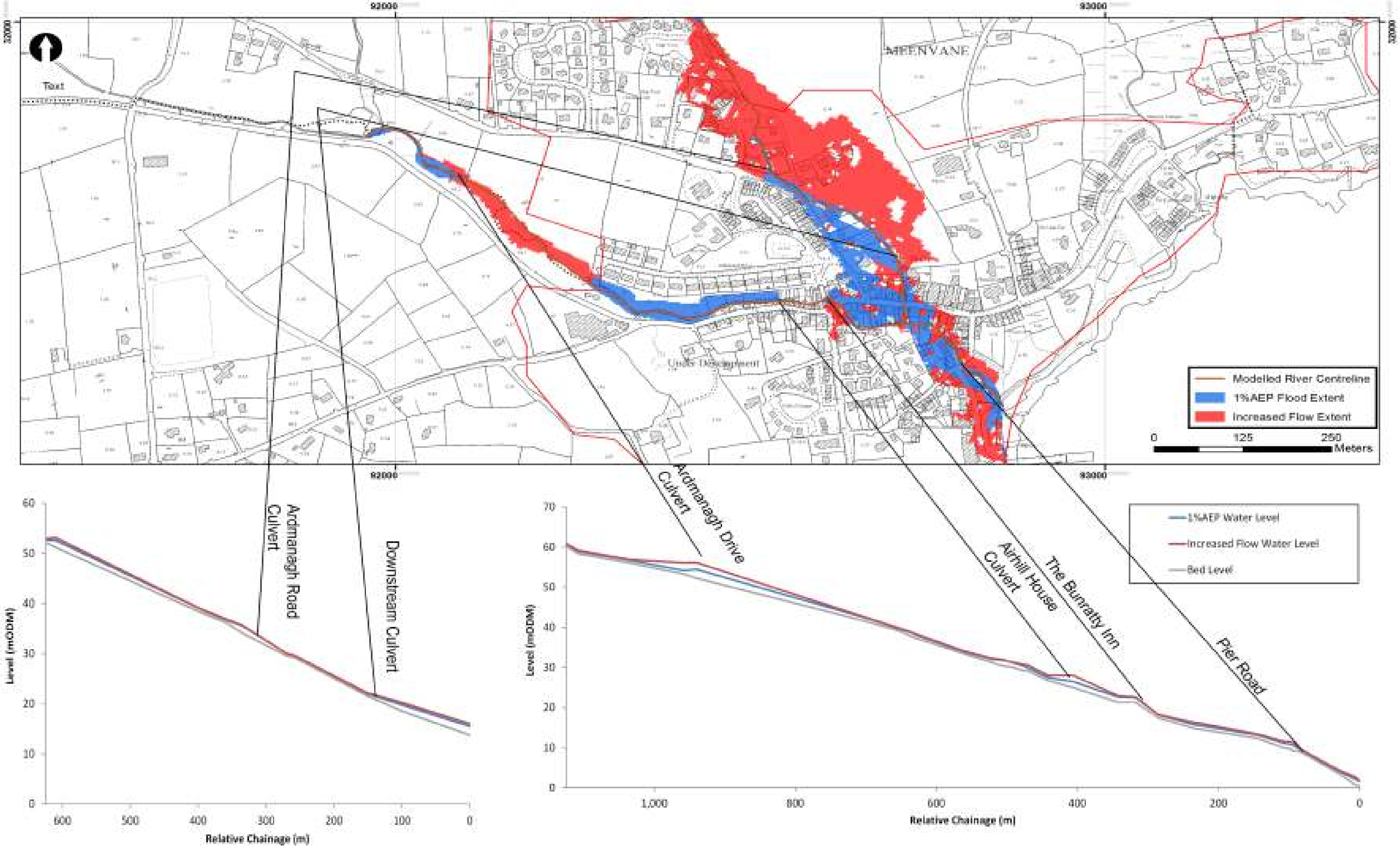
Map 5.7: Sensitivity to Peak Flow – Dunmanway Model



Map 5.8: Sensitivity to Peak Flow-Innishannon



Map 5.9: Sensitivity to Peak Flow-Schull



5.2.2 Level

A sensitivity test was undertaken on downstream water level for the tidally-affected reaches of the Bandon in the Innishannon model, and on the coastal assumptions for the Schull model. This was done to investigate the uncertainties in the estimation of extreme tide plus surge levels extracted from the ICPSS model and the uncertainties in the transformation of water levels along the various bays. The downstream water level was increased by 0.5m to account for these uncertainties. This is broadly equivalent to the sea level increase applied to simulate climate change in the Mid Range Future Scenario (MRFS).

An increased downstream boundary resulted in raised levels by 0.5m through Inishannon up to the Ballymahane tributary, but does not affect flows at Curranure Gauge. This increased flooding at The Lawn and in the playing fields at Innishannon (Map 5.10). Therefore, flood risk in Innishannon was deemed sensitive to the assumptions in the downstream boundary. However, the ICPSS total tide plus surge levels were agreed with OPW as the best estimate of coastal risk in the absence of long term gauge records at Kinsale.

In Schull, an increased downstream boundary level resulted in raised levels up to the downstream face of the Riverside Bridge, but did not affect flows or levels through the AFA upstream (Map 5.11). The increase in downstream level results in a small increase in flooding at Riverside car park but flood risk and extent does not change significantly within the AFA upstream. Therefore, flood risk in Schull was not deemed sensitive to the assumptions in the downstream boundary.

A sensitivity test was also undertaken on downstream water level for the entirely fluvially-affected Dunmanway model. This test investigated the impact of the assumptions taken for the downstream flow-level boundary on flood risk. The gradient in the QH boundary was reduced based on the flattest estimate of the downstream floodplain gradient to investigate the impact of increased backwater upstream.

The flatter downstream boundary at Bandon Town increased water levels between Gurteen and Bandon Town, but did not affect level or flood risk upstream of Gurteen. Therefore flood risk in Dunmanway was not deemed sensitive to the assumptions in the downstream boundary (Map 5.12).

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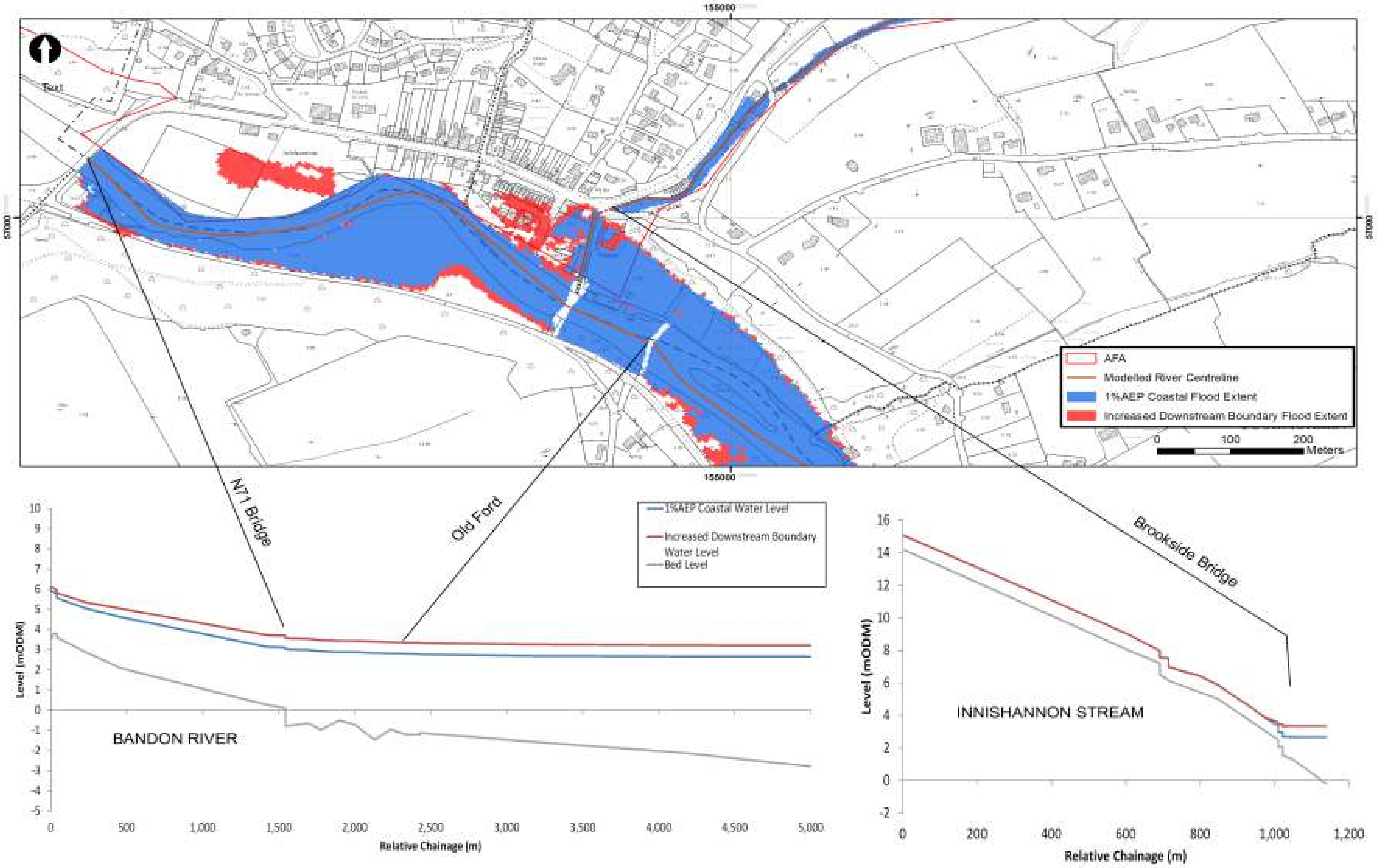
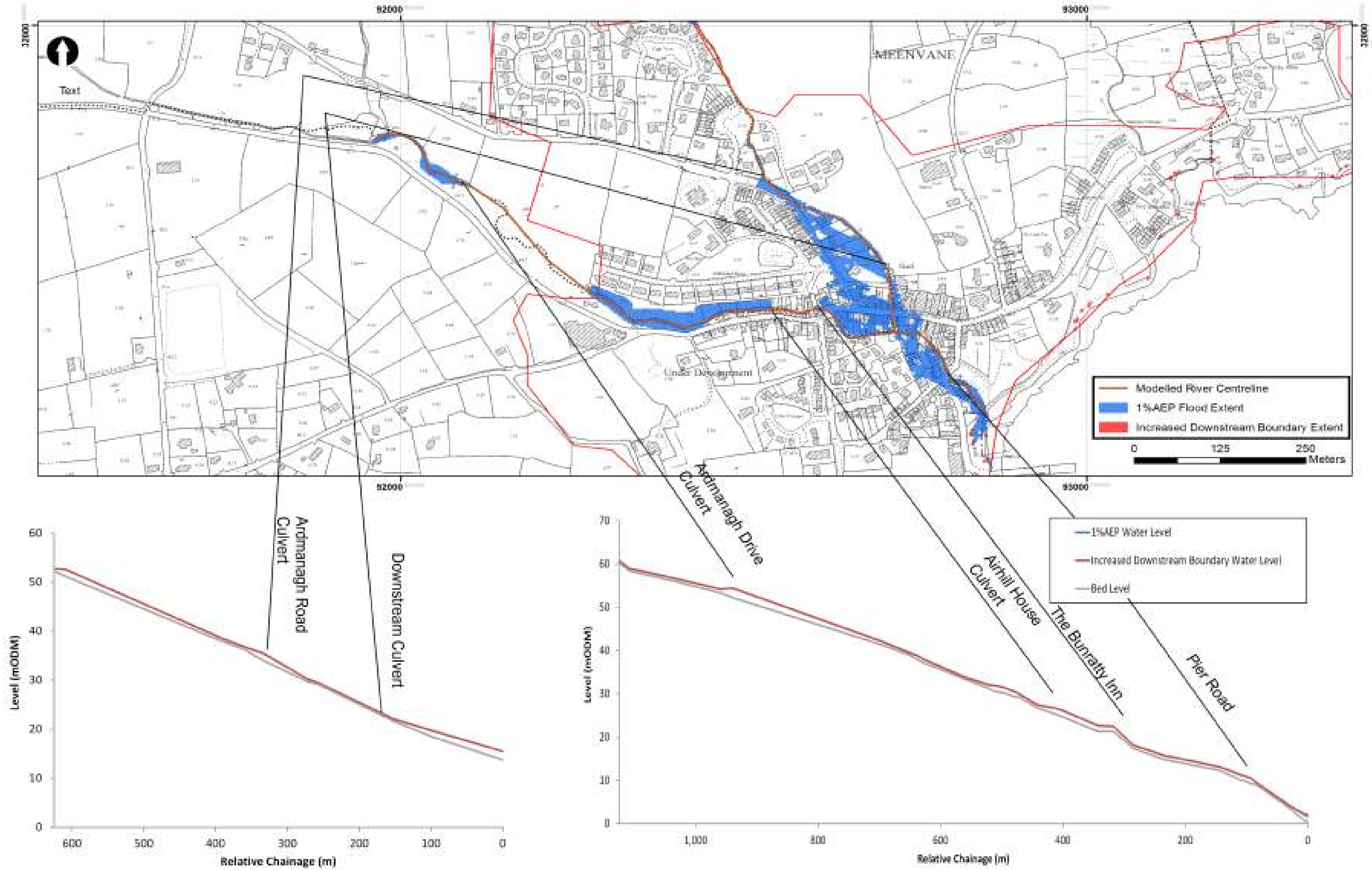
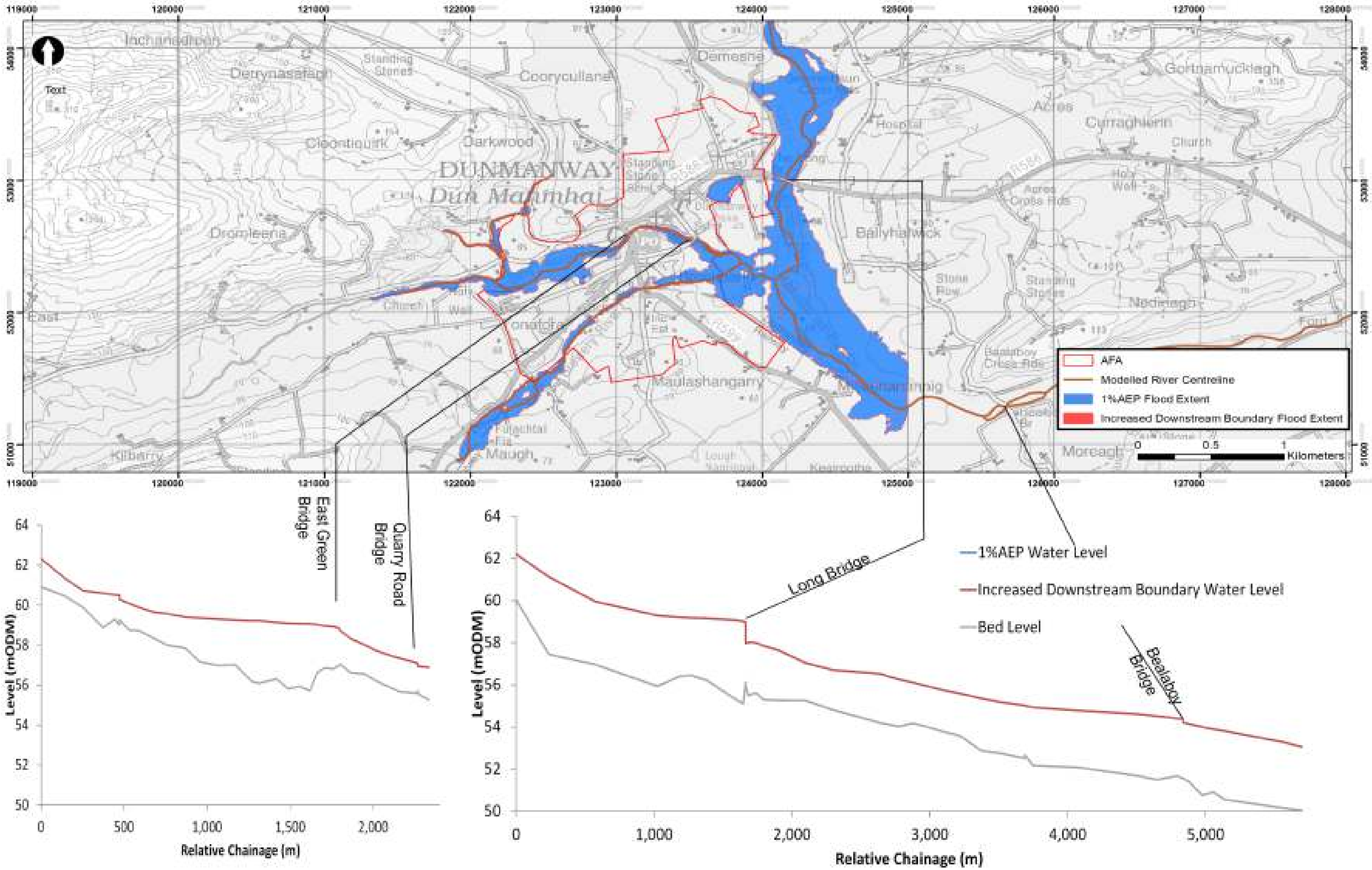
Map 5.10: Sensitivity to Downstream Level – Innishannon Model



Map 5.11: Sensitivity to Downstream Level - Schull Model



Map 5.12: Sensitivity to Downstream Boundary Assumptions - Dunmanway Model



5.2.3 Roughness

In accordance with CFRAM Guidance Note 22, the Manning's 'n' was increased to the next highest value in the recommended ranges for that channel or surface type (Chow 1959) in both the 1D and 2D model components. The Manning's 'n' values were increased in the design model as specified in Table 5.3 and the 1%AEP fluvial event simulated to assess the sensitivity of the predicted flood outline to assumptions in roughness.

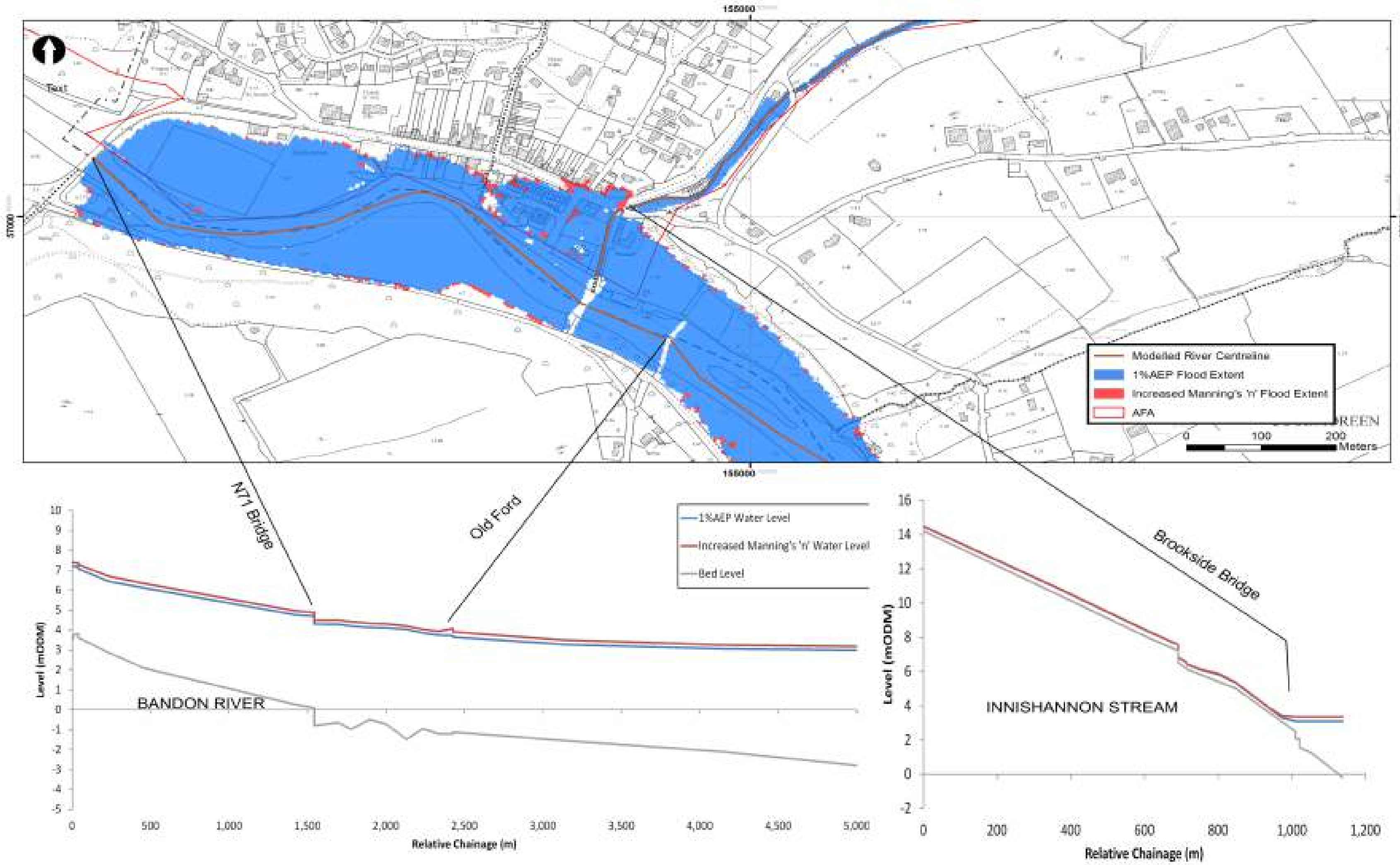
Table 5.3: Sensitivity Manning's 'n' Values

Channel or Surface	Design Manning's 'n'	Sensitivity Manning's 'n'
Active River Channel	0.040	0.045
River Banks/ Medium to Dense Vegetation	0.080	0.100
Buildings	0.200	0.300
Roads and Other Hard Standing	0.033	0.040
Rural/Pasture	0.060	0.080

Flood level and extent were generally not sensitive to the Manning's 'n' values in UoM20. The greatest increase in flood risk attributed to Manning's 'n was predicted in Innishannon (Map 5.13) where flood levels increased by 0.15, but this did not significantly increase flood risk to properties.

The plots for all Manning's 'n' sensitivity tests can be found in the model performance proformas in the relevant Appendices.

Map 5.13: Sensitivity to Increased Manning's 'n' – Innishannon Model



5.2.4 Summary

Table 5.4 summarises the findings of the sensitivity tests undertaken on the design models. Each was deemed sensitive to a parameter if there was a significant increase in flood risk to properties, increased in flooded area (>5%) and increase in water level (±0.2m).

Table 5.4: Summary of Sensitivity Run Performance

Model	Flow		Level		Manning's 'n'	
	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?
Dunmanway	0.32	Yes	0.00	No	0.12	No
Innishannon	0.87	Yes	0.48	Yes	0.15	No
Schull	0.57	Yes	0.08	No	0.04	No

Based on the findings of the sensitivity tests above, the following can be concluded:

- Dunmanway, Innishannon and Schull are all sensitive to the assumptions and uncertainties in peak flow. The sensitivity to peak flow and duration estimates should be considered in the sizing and operation of any flood management options involving the storing of flood waters.
- Innishannon and the Lower Bandon MPW are sensitive to the assumptions and uncertainties in downstream total tide plus surge level up to the Ballymahane confluence. The uncertainty in the total tide plus surge levels should also be considered in the development of any flood embankment/walls to protect against coastal flooding.
- Seasonal changes in vegetation or changes in roughness due to maintenance do not significantly alter flood extent and risk for the 1%AEP event in any of the AFAs in UoM20.

6 Design Event Runs and Model Performance

6.1 Design Scenarios and Event Runs

Table 6.1 outlines the applicable design scenarios to each model in UoM20 and design event runs simulated.

Both the fluvial and coastal scenarios have been simulated for the Lower Bandon in the Inishannon model as this reach has been identified as being at risk from both fluvial and coastal sources. The joint probability between the fluvial and coastal conditions for these scenarios is outlined in Section 3.3 of this report. The model results from the fluvial-dominated event and coastal-dominated event will be combined as part of the flood mapping and post-processing described in Chapter 9 of this report.

No coastal scenarios have been simulated for the Dunmanway model as the Bandon is not tidally influenced upstream of Curranure. No coastal scenarios have been simulated for Schull despite being located on the coast because the AFA was not identified as being at risk from coastal sources by the local engineer during the Flood Risk Review or historic flood reports.

No wave overtopping scenarios have been modelled as the ICWWS did not identify any AFAs at risk from wave overtopping in UoM20.

Table 6.1: Design Event Runs

Source	Scenario	%AEP	Run Name	Dunmanway Model (I23DY)	Inishannon Model (I25IN)	Schull Model (I28SL)
Fluvial	Current	50%	FCD500_D1	✓	✓	✓
		20%	FCD200_D1	✓	✓	✓
		10%	FCD100_D1	✓	✓	✓
		5%	FCD050_D1	✓	✓	✓
		2%	FCD020_D1	✓	✓	✓
		1%	FCD010_D1	✓	✓	✓
		0.50%	FCD005_D1	✓	✓	✓
		0.10%	FCD001_D1	✓	✓	✓
	MRFS	50%	FMD500_D1	✓	✓	✓
		20%	FMD200_D1	✓	✓	✓
		10%	FMD100_D1	✓	✓	✓
		5%	FMD050_D1	✓	✓	✓
		2%	FMD020_D1	✓	✓	✓
		1%	FMD010_D1	✓	✓	✓
		0.50%	FMD005_D1	✓	✓	✓
		0.10%	FMD001_D1	✓	✓	✓
	HEFS	10%	FHD100_D1	✓	✓	✓
		1%	FHD010_D1	✓	✓	✓
		0.10%	FHD001_D1	✓	✓	✓
Coastal	Current	50%	CCD500_D1	N/A	✓	N/A
		20%	CCD200_D1	N/A	✓	N/A
		10%	CCD100_D1	N/A	✓	N/A
		5%	CCD050_D1	N/A	✓	N/A
		2%	CCD020_D1	N/A	✓	N/A
		1%	CCD010_D1	N/A	✓	N/A
		0.50%	CCD005_D1	N/A	✓	N/A
		0.10%	CCD001_D1	N/A	✓	N/A

Source	Scenario	%AEP	Run Name	Dunmanway Model (I23DY)	Inishannon Model (I25IN)	Schull Model (I28SL)
	MRFS	50%	CMD500_D1	N/A	✓	N/A
		20%	CMD200_D1	N/A	✓	N/A
		10%	CMD100_D1	N/A	✓	N/A
		5%	CMD050_D1	N/A	✓	N/A
		2%	CMD020_D1	N/A	✓	N/A
		1%	CMD010_D1	N/A	✓	N/A
		0.50%	CMD005_D1	N/A	✓	N/A
		0.10%	CMD001_D1	N/A	✓	N/A
	HEFS	10%	CHD100_D1	N/A	✓	N/A
		0.50%	CHD005_D1	N/A	✓	N/A
		0.10%	CHD001_D1	N/A	✓	N/A
TOTAL Model Runs				19	38	19

6.2 Model Run Performance

The run performance was investigated for each of the design models for the 1%AEP target event as this represented out-of-bank flooding for the AFAs.

Figures 6.1 to 6.3 show the performance dialog for the 1%AEP fluvial event for the following run performance criteria in the 1D model components;

- The number of iterations per timestep taken to resolve flow and level in the model;
- The convergence of flow and water level in the model within the recommended tolerance of +/- 0.01 m or 0.01 m^3/s between consecutive timesteps;
- The total inflow and outflow from the model components.

The 1D ISIS models were convergent within the recommended tolerances for the majority of the design event in Dunmanway, Innishannon and Schull. The following observations can be made:

- The tolerance is exceeded at 23 and 43 hours in Dunmanway because of high velocity in the steep approach to the Kilbarry Road Bridge. The instabilities are isolated spatially and are outside of the AFA. They do not affect the rest of the event which converges well.

The cumulative mass balance for the 2D model components is shown in Figures 6.4 to 6.6. All the design models were convergent and within the recommended tolerance of ±1% mass error at the peak flow. There is an initial spike in 2D mass error between 1 and 5 hours in Schull as the 2D cells begin to wet at the upstream of the High Street culvert and culverts upstream of the AFA on Schull Stream. However, this reduces to 0.6% before the peak at 8 hours. Therefore the 2D results are deemed convergent and reliable.

Figure 6.1: 1D Convergence Plot – Dunmanway

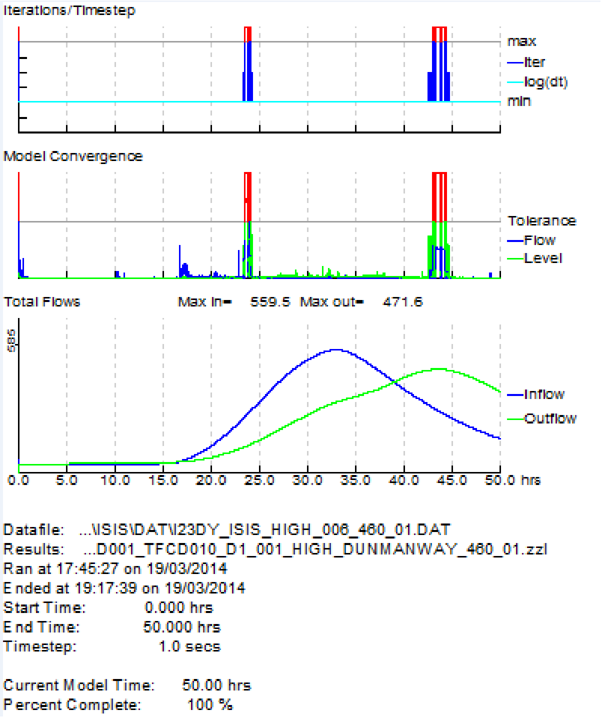


Figure 6.2: 1D Convergence Plot - Innishannon

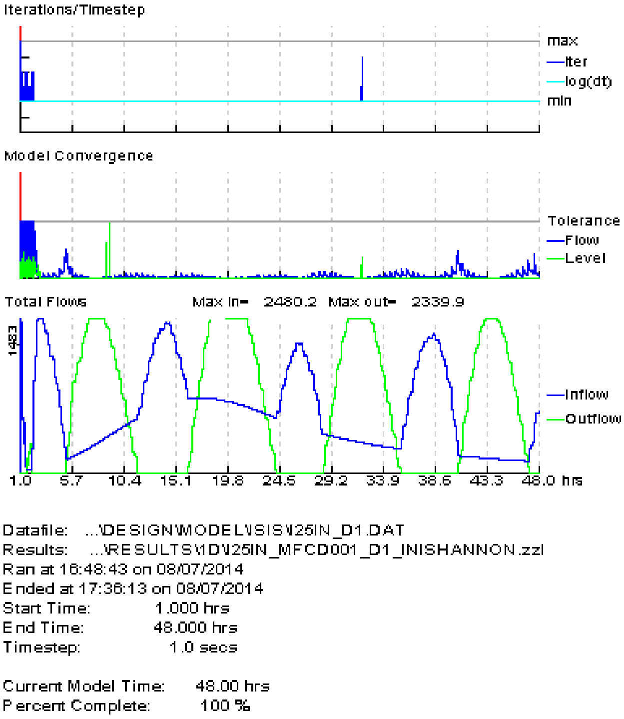


Figure 6.3: 1D Convergence Plot – Schull

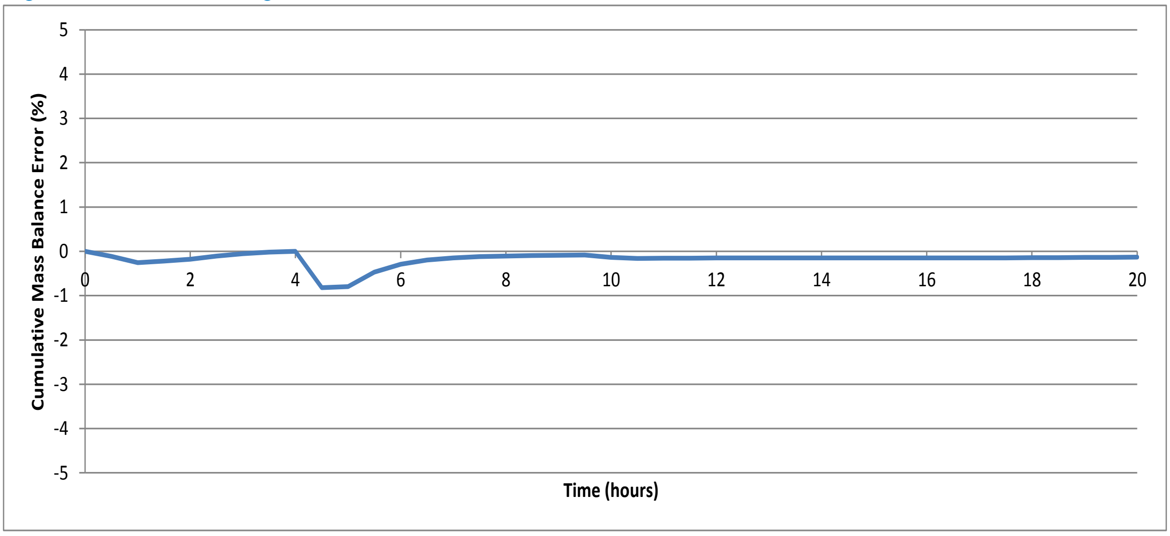


Figure 6.4: 2D Mass Balance Plot – Dunmanway

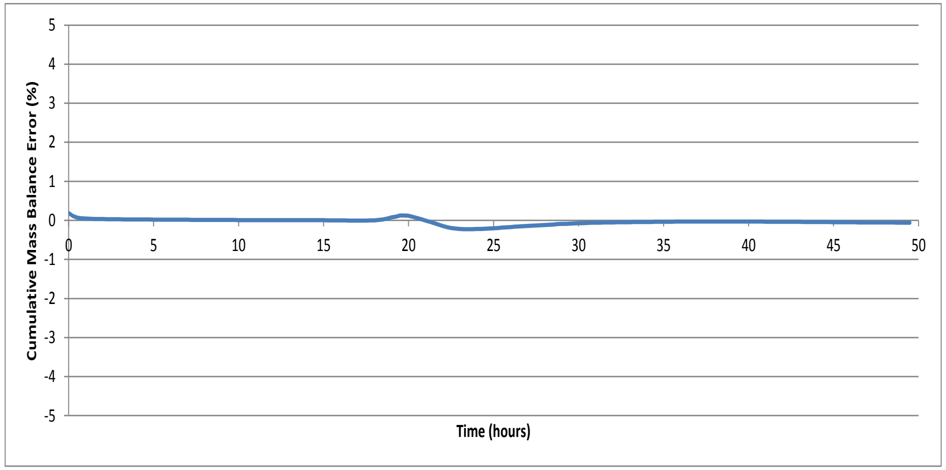


Figure 6.5: 2D Mass Balance Plot - Innishannon

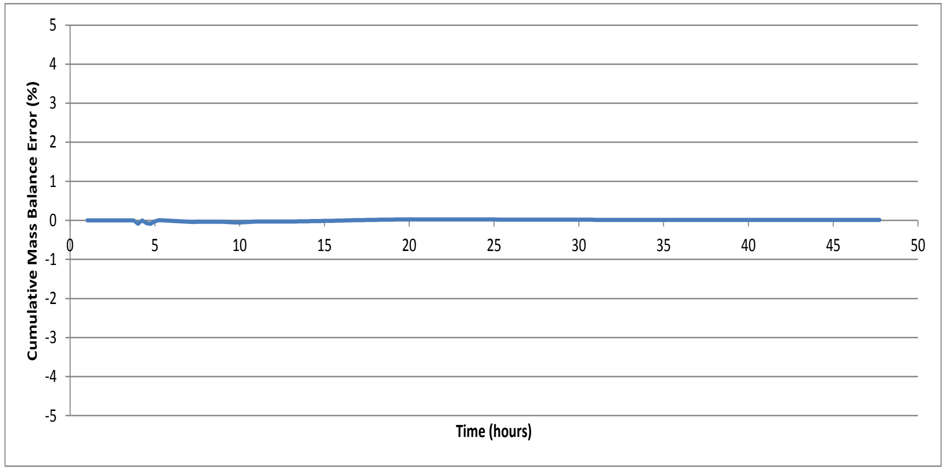
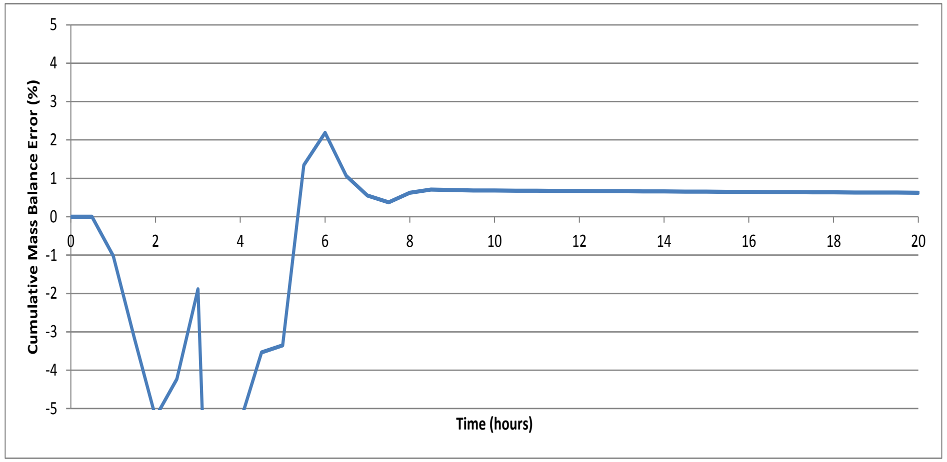


Figure 6.6: 2D Mass Balance Plot – Schull



Tables 6.2 compares the model predicted flows with the design peak flows at the target HEPs for the target 1%AEP event. The model predicted flows have been derived by combining the flows in the 1D channel and across the 2D floodplain to assess the hydrological routing of flows through the catchment. Target flows at HEPs located upstream of confluences were not assessed because these locations are affected by backwater which is not considered in the design hydrology.

The flows in the 1D ISIS channel were combined with 2D flows parallel to the channel where there were out-of-bank flows and compared to the design hydrology.

The modelled flow tended to differ to design flows at HEP locations affected by the tide (i.e. downstream of Rockhouse Creek on the Lower Bandon in the Innishannon model) because the high tide limits the discharge from the River Bandon which is not considered by the design hydrology.

In Dunmanway, the model predicted flows were within 10% of the design flows through the AFA to Bealaboy Bridge. The model tended to underestimate design flows at the downstream HEP for the 0.1%AEP fluvial current event. The phasing has been altered to provide the best estimate for the majority of the model at the 1%AEP target event. Therefore the phasing and joint probability is not optimised for the more extreme 0.1%AEP event.

Table 6.2: Summary of Hydrological Routing Performance for Key Fluvial Current Events

HEP ID	Location	Model Node	10%AEP			1%AEP			0.1%AEP		
			Design Target Flow (m^3/s)	Model Predicted Flow (m^3/s)	Difference (m^3/s)	Design Target Flow (m^3/s)	Model Predicted Flow (m^3/s)	Difference (m^3/s)	Design Target Flow (m^3/s)	Model Predicted Flow (m^3/s)	Difference (m^3/s)
Dunmanway											
20_2126_2	Long Bridge Gauge	20BAND05720E	102	107	5%	151	156	3%	222	225	2%
20_2096_1	Bandon- Dunmanway Lake	20BAND05712H	103	111	8%	153	167	9%	225	246	9%
20_2093_1	Bandon-Dirty	20BAND05619H	128	141	10%	197	206	5%	309	298	-4%
20_2140_2	Bealaboy Gauge	20BAND05406A	131	140	7%	201	204	2%	315	292	-7%
20_754_1	Bandon-Bealascartane	20BAND05285H	138	148	7%	212	213	0%	333	303	-9%
20_742_1	Bandon-Blackwater	20BAND04612B	166	170	2%	256	250	-2%	401	356	-11%
20_664_3	Bandon Downstream	20BAND02921H	184	177	-3%	283	257	-9%	443	368	-17%
Innishannon											
20_652_6	Curranure (20002)	20BAND1994H	204	204	0%	356	356	0%	604	604	0%
20_2232_4	Innishannon Stream Downstream	20INSH0012H	2.5	2.5	0%	4.4	4.4	0%	7.5	7.4	-1%
20_2230_3	Bandon downstream of Inishannon stream	20BAND1760B	231	230	0%	403	395	-2%	683	693	1%
20_147_2+	Bandon at Rockhouse Creek	20BAND1304H	277	248	-11%	483	405	-16%	820	673	-18%
20_2236_5+	Bandon at Ballinadee Creek	20BAND0891H	290	389	34%	506	600	19%	858	854	0%
20_2224_3+	Bandon at Whitecastle Creek	20BAND-0202H	295	1917	550%	515	2070	302%	872	2339	168%
Schull											
20_1990_3	Upstream of Schull/Meevane	SKUL00043I	2.8	2.8	0%	4.2	4.2	0%	6.4	6.4	1%
22_1916_1+	Meenvane Stream Upstream	MEEN00061H	1.3	1.3	-2%	1.9	1.9	0%	2.9	2.9	-1%
22_1916_1	Downstream of Schull/Meenvane	20SKUL00018J	4.1	4.0	-3%	6.1	6.3	3%	9.2	9.7	5%
22_1916_2	Schull Stream Downstream	20SKUL00004E	4.4	4.2	-4%	6.7	6.9	3%	10.0	10.9	9%

7 Assumptions and Limitations

7.1 Assumptions

A number of assumptions were made in the development of the hydraulic model and application of the hydrological inflows. They include:

- The lateral inflows representing the intermediate catchments were assumed to be distributed evenly as rainfall across such a small catchment can be expected to be uniform.
- The peak fluvial flows were assumed to coincide with the peak tidal level at each AFA as a conservative estimate of flood risk. However, it is recognised that the phasing of the river flows and tide will vary event to event.
- The urban drainage network is assumed to be at capacity prior to the start of the event as the worst case scenario as observed in several historic flood events. Therefore, the urban drainage network is not explicitly considered in the design model.
- Model grid size is set at 5 m which was assessed as appropriate for the purpose of the Study. Small urban features, such as fences and walls, have not been considered explicitly as they are not designed to retain water during a flood event. However, the overall impact of these features has been incorporated into the floodplain Manning's 'n'.
- Section data for the cross sections was defined with the hard bed levels. This is because the soft bed or silt is likely to be washed away during a flood.
- It is assumed that water can enter a building above a 0.15m threshold whereupon the water is significantly retarded by the internal structure before exiting the building.
- The "stubby" building approach described above can result in the model calculating reduced flood depths and velocities, along with a greater flood extent as flows are not constricted between buildings.
- In Dunmanway, it is assumed that flapped outfall of Dunmanway Lake will prevent flooding up the tributary until the Bandon River overtops the raised embankment and inundates the channel.
- In Schull, the manhole dimensions have been estimated from site survey and loss coefficients calibrated to achieve the observed flooding. Confined space surveying was part of the survey scope.

7.2 Limitations

There are a number of uncertainties associated with the flow estimation and hydraulic modelling methodology used in UoM20. They include:

- There is uncertainty in the derivation of design flows for small catchments in Schull and Dunmanway Lake. This level of uncertainty must be considered in the interpretation of design flows, flood mapping and in the development of flood mitigation options.
- There is uncertainty in the distribution of flow between the culvert and manholes by Bunratty Inn, Schull due to lack of survey. However, the uncertainty is limited as these loss coefficients have been calibrated to observed flood extents.

- The absence of river flow or continuous water level data in Schull and Inishannon to fully calibrate the hydrological routing and hydraulic model.
- The flood maps produced as part of this Study do not show localised flooding resulting from intense rainfall and where surface flow might exceed the capacity of the urban drainage system. The assessment of such surface water flooding is beyond the scope of the CFRAM studies.
- Groundwater flooding has not been included in assessing the risk of flooding and therefore areas susceptible to groundwater flooding may not be identified in the flood maps. However the PFRA did not identify any of the AFAs in UoM20 as being at risk from groundwater flooding.

8 Flood Mapping Approach

8.1 Approach

The 1D-2D models are configured such that the 1D flows and levels are resolved and hydrodynamically interact with the 2D flows and levels at each timestep. The combined 1D and 2D results were subsequently used to produce the following outputs in accordance with the CFRAM brief:

- Maximum flood depth for each AFA and MPW reach;
- Maximum velocity for each AFA;
- Maximum flood hazard for each AFA;
- Maximum flood extent for each AFA and MPW reach;
- Flood Zone maps for each AFA and MPW reach;
- *Specific Risk Number of Inhabitants – to be provided at a later date;*
- *Specific risk Types of Economic Activity – to be provided at a later date; and,*
- *Specific Risk Density – to be provided at a later date.*

For AFAs, the gridded outputs from the 1D-2D models were used directly or processed to develop the flood maps as discussed below. For MPWs, the maximum water level from the 1D models has been used to derive the flood depth and flood extents. It is important to note that no allowance has been made for the local urban drainage system for either AFAs or MPWs. Therefore, the flood maps assume flooding wherever depth is greater than 0mm.

The Specific Flood Risk Maps (the last three listed above) will be provided at a later date following confirmation of the final methodology.

8.2 Flood Depth and Velocity Mapping

Maximum flood depth and velocity are output directly as GIS grids from the 2D model. The flood depth and velocity maps display the raw model results based on the 5m model grid without the need for any further processing. The flood depth and velocity maps are provided in Schedule 4 of each appendix.

1D water level lines (WLLs) were used to extract depth and velocity information from the 1D river channel in order to produce a seamless flood map. The WLLs plot the maximum water level symmetrically against the flow widths from the centreline in ISIS or ESTRY, which may not be appropriate for asymmetrical cross-sections at meander bends. Therefore, the in-channel water depths presented on the flood maps should be considered in conjunction with the detailed channel survey data presented in the 1D model.

For MPW reaches using a 1D only approach, water levels were assigned to the 1D cross –sections and interpolated to create a water level surface TIN which was then intersected with the DTM to derive flood depths. Any isolated or disconnected areas of flooding were manually reviewed to check whether the water level had overtopped the raised feature, such as a road embankment. The isolated flooding was removed if the maximum water level was below the raised feature crest. Conversely, the previously isolated flooding was connected if the maximum water level was above the raised feature crest.

8.3 Flood Hazard Mapping

The flood hazard was also output direct from the 2D model results, whereby flood hazard is a function of depth and velocity which is calculated for every time step to derive the maximum flood hazard based on modified guidance from the DEFRA FD2320with the removal of debris factor:

$$Flood\ Hazard = Depth\ x\ (Velocity + 0.5)$$

When interpreting flood hazard maps, it is important to consider that the flood hazard rating value has been calculated at each time-step based on concurrent depth, velocity and debris factor. The maximum flood hazard rating value is maximum of these concurrent flood hazard values but does not necessarily coincide with both the maximum depth and maximum velocity. This is produced directly by the TUFLOW model and requires no post-processing to derive flood hazard.

Debris factor has not been considered given the uncertainties associated with variable debris factors based on the underlying land use.

The flood maps categorise the resultant flood hazard values into four broad classes (Table 8.1) which are presented on the flood hazard maps provided in Schedule 4 of each appendix.

Table 8.1: Flood Hazard Categories

Flood Hazard Value	Degree of Flood Hazard	Description
<0.75	Low	Caution - "Flood zone with shallow flowing water or deep standing water"
0.75-1.25	Moderate	Dangerous for some (vulnerable social groups such as children and the elderly) - "Danger: Flood zone with deep or fast flowing water"
1.25-2.00	Significant	Dangerous for most people - "Danger: flood zone with deep fast flowing water"
>2.00	Extreme	Dangerous for all - "Extreme danger: flood zone with deep fast flowing water"

Source: DEFRA FD2320 Table 2 Hazard to People

8.4 Flood Extent and Zone Mapping

The maximum flood extent was derived from the maximum flood depth grid and converted to a closed polygon. Any isolated or disconnected areas of flooding from initial water levels were removed. However, the 2D model simulates all active flow paths, so wet cells are connected at the maximum flood extent. The GIS processing automatically simplifies the polygon to a smoother outline but this does not differ from the modelled grid extent. No additional processing was undertaken to remove dry islands so that the flood outlines matched the modelled grids.

There is one formal flood defence scheme at Dunmanway which contains the 1%AEP event. Therefore, undefended scenarios were simulated to derive Flood Zone A and B without the scheme in place. The

defended areas were provided by OPW. No other formal or informal effective flood defences were identified in the other AFAs considered. Therefore, the flood zone outlines are the same as the flood extents for Inishannon and Schull. The 1%AEP and 0.1%AEP fluvial extents were used to represent Fluvial Flood Zone A and B respectively. The 0.5%AEP and 0.1%AEP coastal extents were used to represent the coastal Flood Zones A and B respectively.

8.5 Combined Flood Source Mapping

The Lower Bandon in the Inishannon model is subject to flooding from both fluvial and tidal influence. Therefore, the fluvial-dominant flood extent was merged with the tidal-dominant flood extent to produce the maximum flood extent from both sources. It should be noted that this does not represent a target %AEP assessed in the joint-probability, but provides a useful summary of the maximum extent from both sources.

8.6 Flood Risk (Assessment) Mapping

8.6.1 General Flood Risk Maps

The potential adverse consequences (risk) associated with flooding in each of the AFA's was assessed and mapped against four risk receptor groups:

- Society (including risk to people)
- The Environment
- Cultural Heritage
- The Economy

Maps were produced by overlaying flood extents for key AEP events on GIS datasets for each of the four receptor groups listed above. Depending on the density of the receptors at each AFA, separate maps were prepared for each receptor or combined on a single map.

8.6.2 Specific Flood Risk Maps

Specific Flood Risk maps are required for key indicators. These include the following:

- Indicative Number of Inhabitants
- Types of Economic Activity
- Economic Risk Density

8.6.2.1 Indicative Number of Inhabitants

For each AFA, the study area was broken into a number of grids, each 10,000m^2 (i.e. 1 ha). The population density per ha was calculated by summing the number of residential properties within each grid and multiplying by an average occupancy rate determined by the Central Statistics Office.

8.6.2.2 Types of Economic Activity

Each property within an AFA was assigned a use, which was based on the property survey. The types of economic activity were identified and represented on a map with flood extents for key AEP events overlain.

8.6.2.3 Economic Risk Density

The maximum depth of flooding was extracted for each building polygon for the full range of AEP events using the results of the hydraulic modelling and flood mapping. The depth of flooding was multiplied by the area of the property and the unit cost of damage per m^2. The selected unit cost is dependent on the property type which was determined through a property survey. The unit cost of damage for different property types was taken from the Multi-colour Manual published by the Flood Hazard Research Centre.

The Annual Average Damage (AAD) methodology will be confirmed at a later date. Once calculated, the AAD for each property within each a 10,000m^2 (i.e. 1 ha) grid will be summed and represented on a map providing the economic risk density (€ AAD / ha).

9 Model and Mapping Results

9.1 Overview

The modelling undertaken has confirmed that flood alleviation scheme in Dunmanway protects the Chapel Street area from flooding in the 1%AEP event as designed. Flood risk remains for Chapel Street but only in the extreme 0.1%AEP fluvial flood event.

Based on the model predicted results and flood maps, the greatest current fluvial flood risk in UoM20 is located at Schull with over 50 properties at risk from the 1%AEP fluvial current extent, although depth of flooding is relatively low (0.2 to 0.5m). Up to 20 properties in Schull are at risk of flooding on a more regular basis as observed in the recent flash flood events in 2009 and 2012.

Flooding in Innishannon is driven by high flows along the Bandon, affecting over 30 properties in the 1%AEP fluvial current event. Innishannon is also affected by extreme tidal conditions although this does not cause significant flooding to properties.

The following sections summarise the key findings for each AFA to highlight the flooding issues identified in the flood maps. A more detailed assessment of receptors at risk and implications for these receptors is discussed in the subsequent Flood Risk Review.

9.2 Dunmanway AFA

Map 9.1 summarises the fluvial flood risk in Dunmanway for the 10%, 1% and 0.1%AEP design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Overtopping of the flood defence embankments at Macroom road and at Church View in extreme fluvial events to flood properties along Chapel Street and around Dunmanway Lake.
- Overtopping of Sacksville Bridge and Underhill Bridge on the Dirty and Brewery rivers respectively to flood adjacent properties in School Road, bypassing the bridge in the most extreme fluvial events.

The key thresholds and areas affected by flooding in Dunmanway are:

- 50%AEP event inundates the floodplain upstream of Long Bridge and flows through the flood relief culverts but does not exceed the protection provided by the flood alleviation scheme.
- 2%AEP fluvial current event overtops Sacksville Bridge on Dirty River and the bridge at Underhill Commercial Estate on Brewery River.
- 0.5%AEP fluvial current event begins to inundate critical infrastructure at the water treatment works at Long Bridge.
- 0.5%AEP fluvial current event overtops the raised embankments upstream of Long Bridge and adds to flow along the Dunmanway Lake Tributary.
- 0.1%AEP fluvial current event exceeds the capacity of the the flood alleviation scheme entirely to flood properties along Chapel Street from overtopping of the embankment upstream of Long Bridge and downstream by Church View.
- 0.1%AEP fluvial current exceeds the capacity of the access bridge at Rock Springs on the Dirty River Tributary.

The greatest risk to life is associated with deep and potentially fast flowing water upstream of the flood relief culverts at Long Bridge which is classed as extreme in 1%AEP fluvial current event but does not affect properties. The greatest risk to life at properties is located along Chapel Street and the Dunmanway Lake area in the extreme 0.1%AEP fluvial current event.

The critical structures in determining flood risk include:

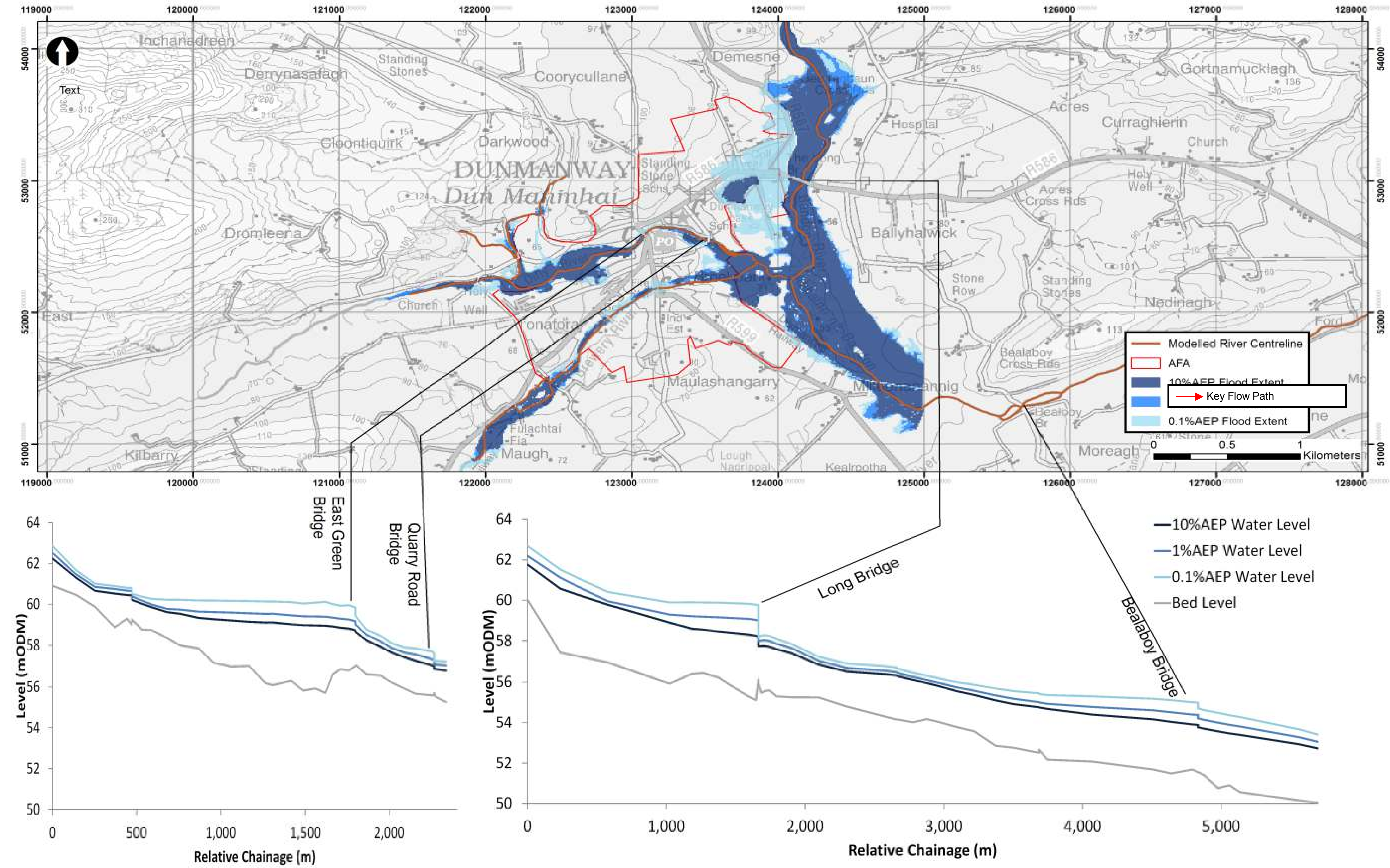
- The flood alleviation scheme comprising of the raised embankments, flood relief culverts on the Bandon River and flapped outfall on the Dunmanway Lake tributary.
- Sacksville Bridge on the Dirty River.
- Underhill Bridge on the Brewery River.

The areas flooded are consistent with flood reports before and after construction of the flood alleviation scheme. The model has been calibrated to three events at Long Bridge, Bealaboy Bridge and Bandon Town gauges. Therefore there is reasonable confidence in the flood mapping in Dunmanway based on the information available at the time of this study.

The following recommendations for flood risk management option development can be made:

- The flood embankment upstream of Long Bridge at Macroom Road could be improved to protect against the 0.1%AEP if required.
- Flood storage is a feasible option on the Bandon, Brewery and Dirty rivers upstream of the AFA given the available floodplain and position in the catchment to capture and store excess flows.
- Flood warning on the Bandon is likely to be effective as the time to peak is over 6 hours ensuring sufficient lead time for flood warning actions.

Map 9.1: Summary of Fluvial Flood Risk – Dunmanway



9.3 Innishannon AFA

Map 9.2 summarises the fluvial flood risk in Innishanon for the 10%, 1% and 0.1%AEP fluvial design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Overtopping of the left bank to flood properties along The Lawn and Main Street.
- Overtopping at the downstream face of Brookside Bridge on Innishannon Stream in extreme events through the access gate on the right bank.
- The N71 Bridge is not overtopped in any fluvial current or tidal current scenario modelled.

The key thresholds and areas affected by flooding in Innishannon are:

- 10%AEP fluvial current event floods the playing fields from the River Bandon. Pluvial flooding may occur more frequently, but this is not considered in the CFRAM Study.
  10%AEP fluvial current event begins to flood along The Lawn but does not affect properties until the 2%AEP fluvial current.
- 1%AEP fluvial current event floods the N71 along Main Street from Brookside Bridge and The Lawn.
- 2%AEP coastal current event begins to flood the road at The Lawn but properties are not affected.

The greatest risk to life associated with flooding in Innishannon is associated with deep and fast flowing water on the playing fields and at the back of the houses on Main Street. Flood water spills over Main Street in the 1%AEP and 0.1%AEP fluvial current which may pose a hazard to road users.

The critical structures in determining flood risk include:

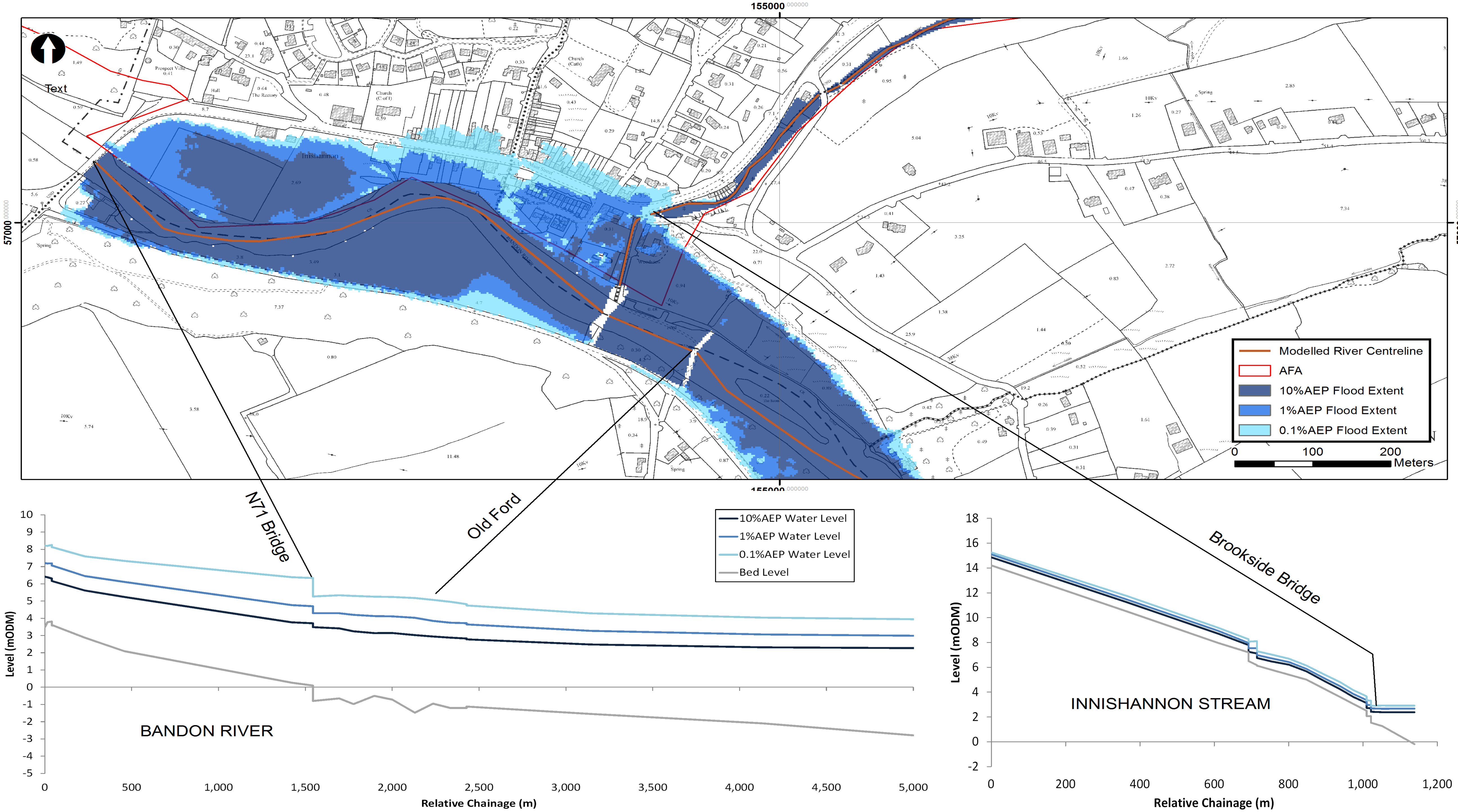
- The capacity of the channel at the old ford crossing and the island downstream.
- The access gate on the downstream of Brookside Bridge.

The areas flooded are consistent with flood reports and interviews with residents. The model has been validated against observed flooding in November 2009. Therefore there is reasonable confidence in the flood mapping in Innishannon based on the information available at the time of this study.

The following recommendations for flood risk management option development can be made:

- Flood warning on the Bandon is likely to be effective as the fluvial time to peak is over 6 hours and the tidal conditions can be predicted several days in advance.

Map 9.2: Summary of Fluvial Flood Risk – Innishannon



9.4 Schull AFA

Map 9.3 summarises the fluvial flood risk in Schull for the 10%, 1% and 0.1%AEP fluvial design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Surcharging of the manhole and paved-over Schull Stream by the Bunratty Inn and flowing down the High Street and Pier Road.
- High flows exceeding the culvert capacity on the Meevane Stream, flowing down Ardmanagh Road and adding to flooding along the High Street.
- Overtopping of the left bank upstream of Pier Road Bridge to flood Pier Road cottages.

The key thresholds and areas affected by flooding in Schull are:

- 20%AEP fluvial current event exceeds the capacity of the manholes by the Bunratty Inn on Schull Stream and downstream culvert on the Meevane Stream to cause flooding along the High Street.
- 5%AEP fluvial current event exceeds the capacity of the Pier Road Bridge to flood Pier Road Cottages.
- 0.5%AEP fluvial current event floods properties on the left and right banks of Meevane Stream upstream of Ardmanagh Road
- 0.1%AEP fluvial current event exceeds the capacity of the culvert upstream of Ardmanagh Drive on Schull Stream to flood fields upstream of the AFA but does not affect additional properties.

The greatest risk to life is associated with fast flowing water from the Bunratty Inn down the High Street in the 1%AEP and greater magintude events. Flood water along the High Street and Ardmanagh Road may also form a hazard to road users.

The critical structures in determining flood risk include:

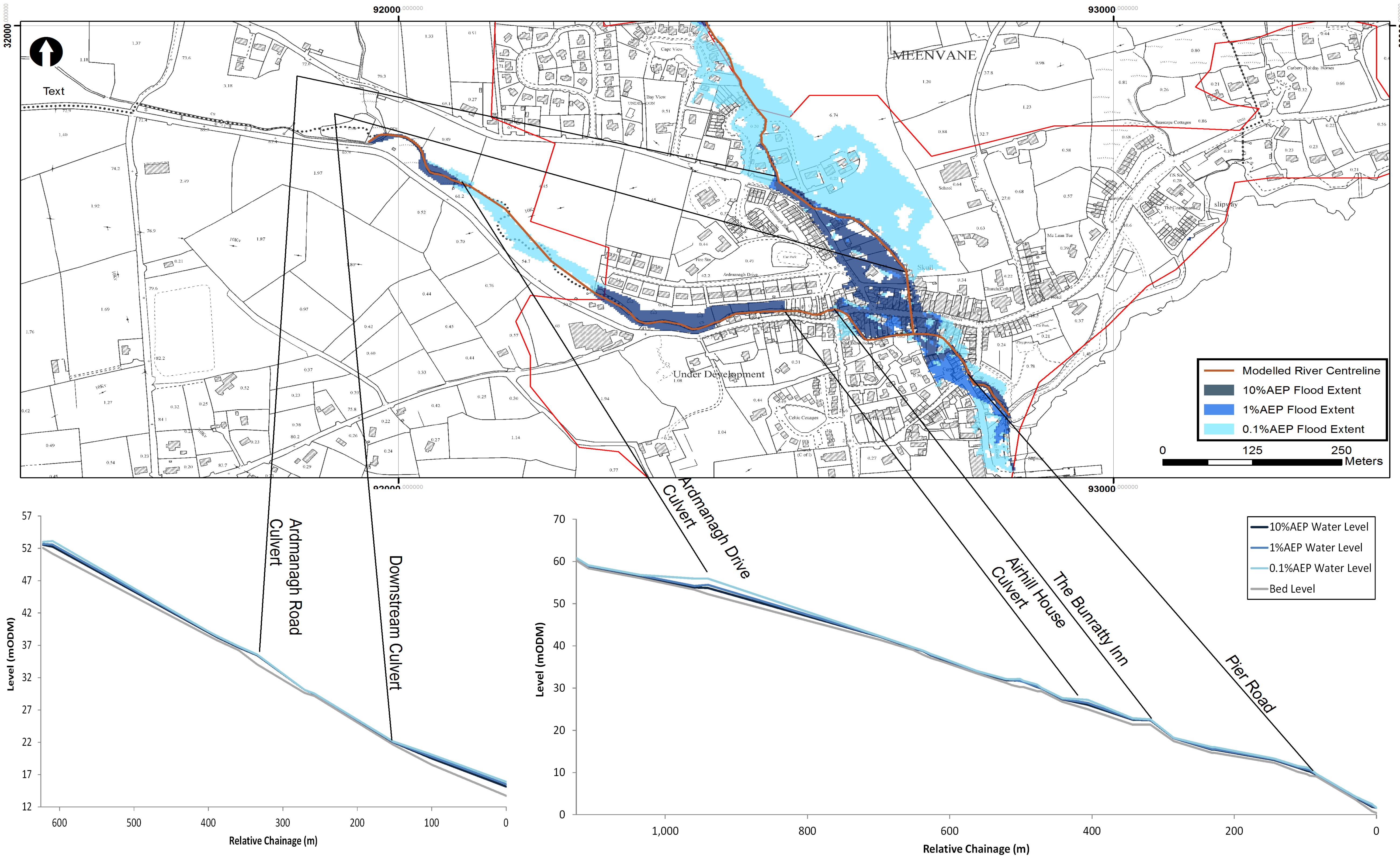
- The High Street culvert on Schull Stream and associated manholes.
- Culverts on the Meevane Stream.
- Pier Road Bridge on Schull Stream.

The areas modelled as flooded are consistent with flood reports and interview with residents. The model has been validated against observed flooding in November 2009. Therefore there is reasonable confidence in the flood mapping in Schull based on the information available at the time of this study.

The following recommendations for flood risk management option development can be made:

- Increased conveyance at the key structures identified is likely to reduce flood risk.
- Flood warning for Schull is likely to be ineffective as the flood risk is driven by a small flashy catchment.

Map 9.3: Summary of Fluvial Flood Risk – Schull AFA



10 Summary and Recommendations

10.1 Key Findings

The hydraulic analysis for UoM20 has developed three hydraulic models to assess current and future flood risk from the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1%AEP fluvial and tidal flood events. The design flood levels and flows were then processed to map flood extent, flood depth, flood velocity and flood hazard in the three AFAs and flood extent and depth maps for two reaches of MPW.

Historic flood events

- The Dunmanway model was calibrated to the January 1991, October 1996 and November 2009 flood event and recurring flood events.
- Modelled flows in Dunmanway were found to within 6% of gauged flows, and calibrated well with the recorded extents of flooding with the flood alleviation scheme in place for the November 2009 event.
- The Innishannon model was validated against the November 2009 event due to lack of spot levels to undertake full calibration.
- The Innishannon model outputs matched with reported flooding at the playing fields and flood depths at affected properties along Main Street.
- The Schull model was calibrated to the October 2009 and August 2012 flash flood events to represent the surcharging of the manholes at the Bunratty Inn and flow along Ardmanagh Road and the High Street. Flood depths were slightly underestimated at the downstream of the High Street because the model does not consider the additional flooding from surface water (pluvial).

Sensitivity test results

- The modelled flood risk in Dunmanway, Innishannon and Schull are all sensitive to the assumptions and uncertainties in peak flow. The sensitivity to peak flow and duration estimates should be considered in the sizing and operation of any flood management options involving the storing of flood waters.
- Innishannon and the Lower Bandon MPW are sensitive to the assumptions and uncertainties in downstream total tide plus surge level up to the Ballymahane confluence. The uncertainty in the total tide plus surge levels should also be considered in the development of any flood embankment/walls to protect against coastal flooding.
- Seasonal changes in vegetation or changes in roughness due to maintenance do not significantly alter flood extent and risk for the 1%AEP event in any of the AFAs in UoM20.

Model and mapping results

The hydraulic modelling and mapping results were analysed for the design scenario under current conditions, the mid range future scenario and high end future scenario. The key findings are summarised below.

- Of the AFAs assessed in UoM20, the greatest fluvial flood risk is located in Schull with over 50 properties at risk from the 1%AEP fluvial current extent, although depth of flooding is relatively low (0.2 to 0.5m).
- Up to 20 properties in Schull are at risk of flooding on a more regular basis as observed in the recent flash flood events in 2009 and 2012.
- Flooding in Innishannon is driven by high flows along the Bandon, affecting over 30 properties in the 1%AEP fluvial current event.
- Innishannon is also affected by extreme tidal conditions, although this does not cause significant flooding to properties.
- The flood alleviation scheme in Dunmanway protects the Chapel Street area from flooding in 1%AEP as designed.
- Flood risk remains for Chapel Street but only in the extreme 0.1%AEP fluvial flood event.

10.2 Recommendations

The following recommendations can be drawn from the key findings above for the subsequent flood risk assessment, preliminary option development and FRMP:

- The uncertainty and sensitivity to peak flow and duration estimates should be considered in the sizing and operation of any flood management options in Bandon, Innishannon and Schull.
- The uncertainty in the total tide plus surge levels should also be considered in the development of any flood embankment/walls to protect against coastal flooding in Innishannon.
- Increased maintenance of channels as an independent measure is unlikely to manage flood risk at the 1%AEP for any of the AFAs assessed. It may be more effective for more frequent events and/or in combination with other measures.
- The capacity of the culverts in Schull should be carefully considered for increased conveyance options to reduce flood risk upstream.

The following recommendations can be drawn from the hydraulic analysis for future analysis in the UoM20:

- It is recommended that post-flood surveys are continued for all significant future flood events where properties and/or infrastructure are affected. Data should be collected shortly after the event and include: sources of flooding, timing of overtopping, any actions taken and at what time, blockages of structures, flood levels in the channel and on the floodplain and accompanying photographs.
- It is recommended that surface water flooding and the interaction of flooding with the urban drainage network is investigated in Schull and Innishannon, given the history of pluvial flooding.

Glossary

AAD	Annual Average Damage: Average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
AEP	Annual Exceedance Probability; this represents the probability of an event being exceeded in any one year and is an alternative method of defining flood probability to 'return periods'. The 10%, 1% and 0.1% AEP events are equivalent to 10-year, 100-year and 1000-year return period events respectively.
AFA	Area for Further Assessment – Areas where, based on the Preliminary Flood Risk Assessment and the CFRAM STUDY Flood Risk Review, the risks associated with flooding are potentially significant, and where further, more detailed assessment is required to determine the degree of flood risk, and develop measures to manage and reduce the flood risk.
AMAX	Annual Maximum Flood
CFRAM	Catchment Flood Risk Assessment and Management – The 'CFRAM' Studies will develop more detailed flood mapping and measures to manage and reduce the flood risk for the AFAs.
DTM	Digital terrain model; elevation of the bare ground surface without any objects like plants, buildings and man-made structures.
EU	European Union
EPA	Environmental Protection Agency
FRMP	Flood Risk Management Plan. This is the final output of the CFRAM study. It will contain measures to mitigate flood risk in the AFAs.
FRR	Flood Risk Review – an appraisal of the output from the PFRA involving on site verification of the predictive flood extent mapping, the receptors and historic information.
FSU (WP)	Flood Studies Update (Work Package) (2008 to 2011)
FSR	Flood Studies Report (HR Wallingford, 1975)
GIS	Geographical Information Systems
HA	Hydrometric Area. Ireland is divided up into 40 Hydrometric Areas.
HEFS	High-End Future Scenario to assess climate and catchment changes over the next 100 years assuming high emission predictions from the International Panel on Climate Change.
HEP	Hydrological Estimation Point
HPW	High Priority Watercourse. A watercourse within an AFA.
ICPSS	Irish Coastal Protection Strategy Study (2012)
ICWWS	Irish Coastal Water Level and Wave Study (2013)
IFSAR	Inter-ferometric Synthetic Aperture Radar used to derive ground elevation remotely from satellite platforms.

ING	Irish National Grid system, Ordnance Survey of Ireland
LiDAR	Light and Detection Ranging used to derive ground elevations from ground based or aerial platforms.
MPW	Medium Priority Watercourse. A watercourse between AFAs, and between an AFA and the sea.
MRFS	Mid-Range Future Scenario to assess climate and catchment changes over the next 100 years assuming medium emission predictions from the International Panel on Climate Change.
ODM	Ordnance Datum Malin.
	The current geodetic datum of Irish National Grid which references the mean sea level at Malin Head between 1960 and 1969.
OPW	Office of Public Works, Ireland
OSi	Ordnance Survey Ireland
PFRA	Preliminary Flood Risk Assessment – A national screening exercise, based on available and readily-derivable information, to identify areas where there may be a significant risk associated with flooding.
QMED	Median annual flood used as the index flood in the Flood Studies Update. The QMED flood has an approximate 50%AEP.
SAAR	Standard average annual rainfall 1961 to 1990
SEA	Strategic Environmental Assessment. A high level assessment of the potential of the FRMPs to have an impact on the Environment within a UoM.
SW CFRAM	South Western Catchment Flood Risk Assessment and Management study
UoM	Unit of Management. The divisions into which the RBD is split in order to study flood risk. In this case a HA.
UPO-ERR Gamma Curve	Unit-Peak-at-Origin Gamma curve coupled with an Exponential Replacement Recession curve. Developed in the Flood Studies Update Work Package 3.1 Hydrograph Width Analysis to derive design flood hydrographs.
WFD	Water Framework Directive. A European Directive for the protection of water bodies that aims to, prevent further deterioration of our waters, to enhance the quality of our waters, to promote sustainable water use, and to reduce chemical pollution of our waters.