South Western CFRAM Study

Final Hydraulics and Flood Mapping Report,
Unit of Management 21

June 2016

The Office of Public Works

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Issue and revision record

Revision	Date	Originator	Checker	Approver	Description	Standard
A	Nov 2013	M Piggott	R Gamble	R Gamble	Draft	
B	May 2014	M Piggott	R Gamble	R Gamble	Draft Final	
C	August 2014	M Piggott	R Gamble	R Gamble	Draft Final 2 Minor amendments to text	
D	June 2016	M Piggott	B O'Connor	B O'Connor	Revised to Final status	

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 21. The model results and flood maps from this report inform the subsequent strategic environmental assessment and flood risk management plans.

Three 1D-2D ISIS-TUFLOW hydraulic models have been developed for Durrus, Bantry and Kenmare to assess fluvial and coastal flood risk for various flood probabilities. The river channels have been modelled using 1D ISIS software to calculate flows and head loss 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. A 2D TUFLOW model was developed to assess coastal flood risk in Castletown Bearhaven as it was not deemed to be at risk from fluvial flooding.

The Bantry and Kenmare models were calibrated to flood events of 17th October 2012 and 23rd October 2008 where sufficient data enabled full calibration of the hydraulic parameters. Sensitivity tests were undertaken on flow, downstream level and Manning's 'n' for all models. An additional sensitivity test on the assumptions on the utility pipe at Finnihy Bridge, Kenmare was also undertaken.

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 extent, flood zone, flood depth, flood velocity and flood hazard 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.

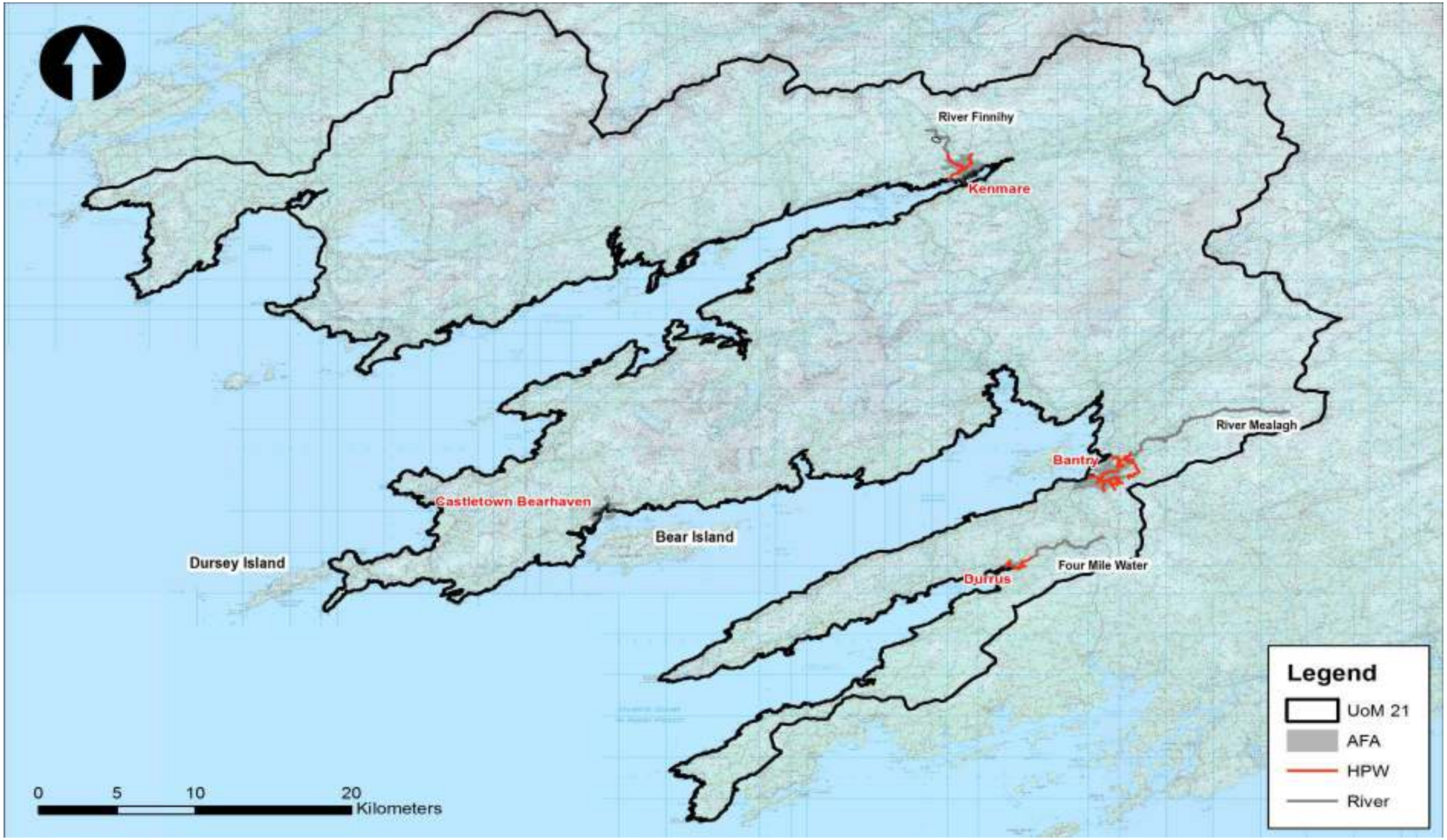
The South Western River Basin District is split into five Units of Management (UoM). These Units follow watershed catchment boundaries and do not relate to political boundaries. The Units are as follows;
- The Blackwater catchment (UoM18)
- The Lee / Cork Harbour Catchment (UoM19)
- The Bandon / Skibbereen Catchment (UoM20)
- The Dunmanus / Bantry / Kenmare Bay Catchment (UoM21)
- The Laune / Maine / Dingle Bay Catchment (UoM22)

Map 1.1 displays the extent of UoM21 which is the subject of 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;
- Development and assessment of flood risk mitigation options; and,
- Development of the Flood Risk Management Plans (FRMPs).

Map 1.1: Unit of Management 21



1.2 Report Structure

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 to 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 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 use a number of other acronyms and technical terminology which are defined in the glossary included at the end 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 21 and the confidence in each dataset based on the review in the following sections. The specific details of the data used for each model are included in the model Appendices.

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 in UoM21	OPW	As part of this study 2012-2013
Detailed Digital Terrain Models	Filtered LiDAR data for AFAs	OPW	2012
National Height Model	IFSAR coarse elevation data with national coverage	OPW	2010
OSI Mapping	Building footprints and vector data of land cover	OSI	2010

2.2 Geometric Survey Data

As part of this study, extensive river channel survey was undertaken of all the High Priority Watercourses (HPWs) in UoM21. The extent of this work which was carried out between September 2012 and April 2013 by Murphy Surveys Ltd. is shown on Map 2.1. 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 to this report. The detailed survey specification and data is available in a separate survey 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 to the watercourse 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.

In UoM21, 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 specific checks carried out on 10% of the cross sections. Levels from the river channel cross sections were checked against the Digital Terrain Model (DTM) as described in Section 2.3. The average difference between the levels from the surveyed cross sections and equivalent levels from the DTM was found to be 0.18mm.

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 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 was assessed to be appropriate for use without further adjustment. The final DTMs for each model are displayed in the model geoschematics in the Appendices.

2.4 Land Cover Data

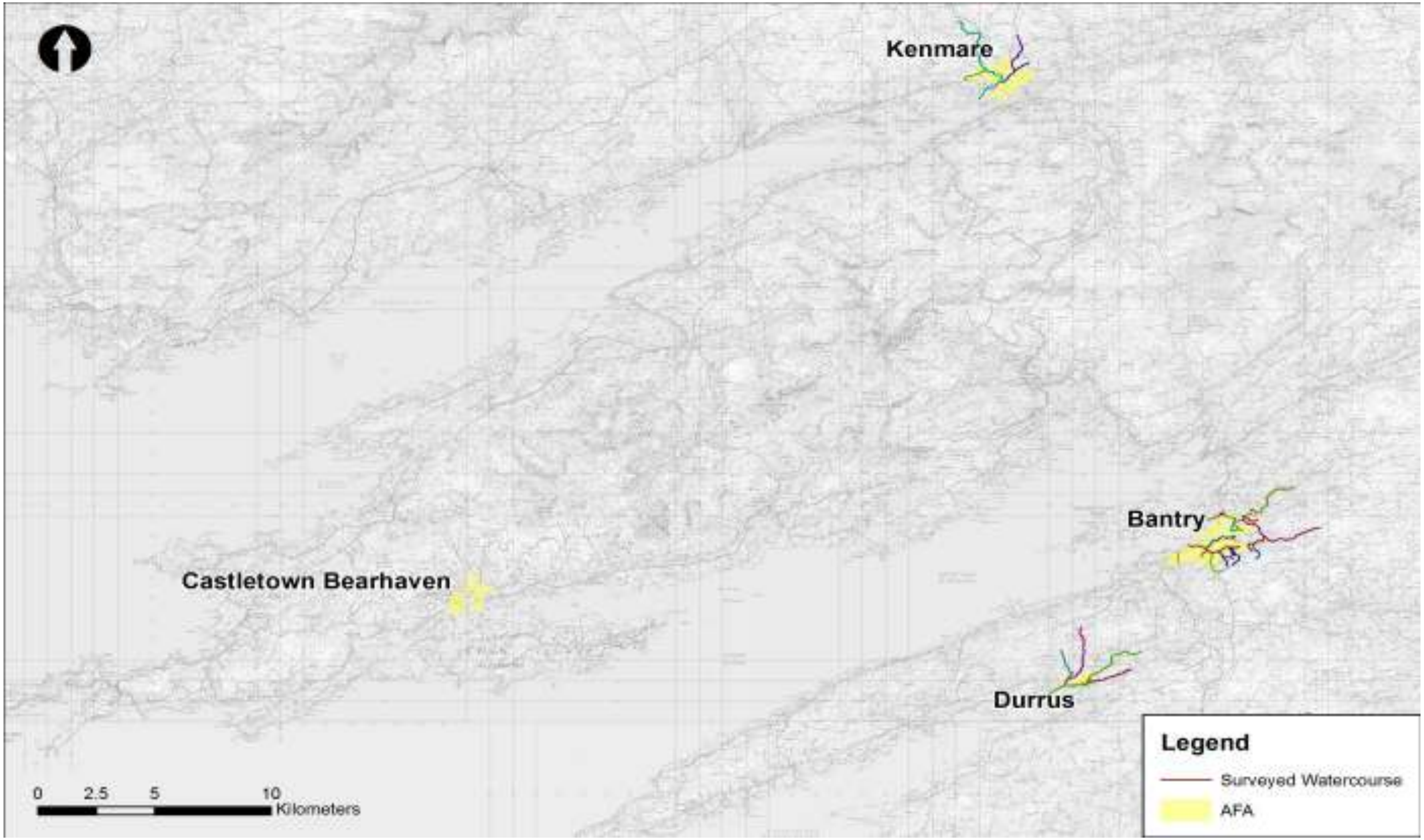
The various types of surfaces in the AFAs were assessed from the following data sources to inform the hydraulic roughness parameters for modelling:

- Building footprints derived from OSI mapping
- 1:1000,1;2500 and 1:5000 vector OSI Mapping
- Surface cover detailed in the geometric survey and survey photographs
- Site visits

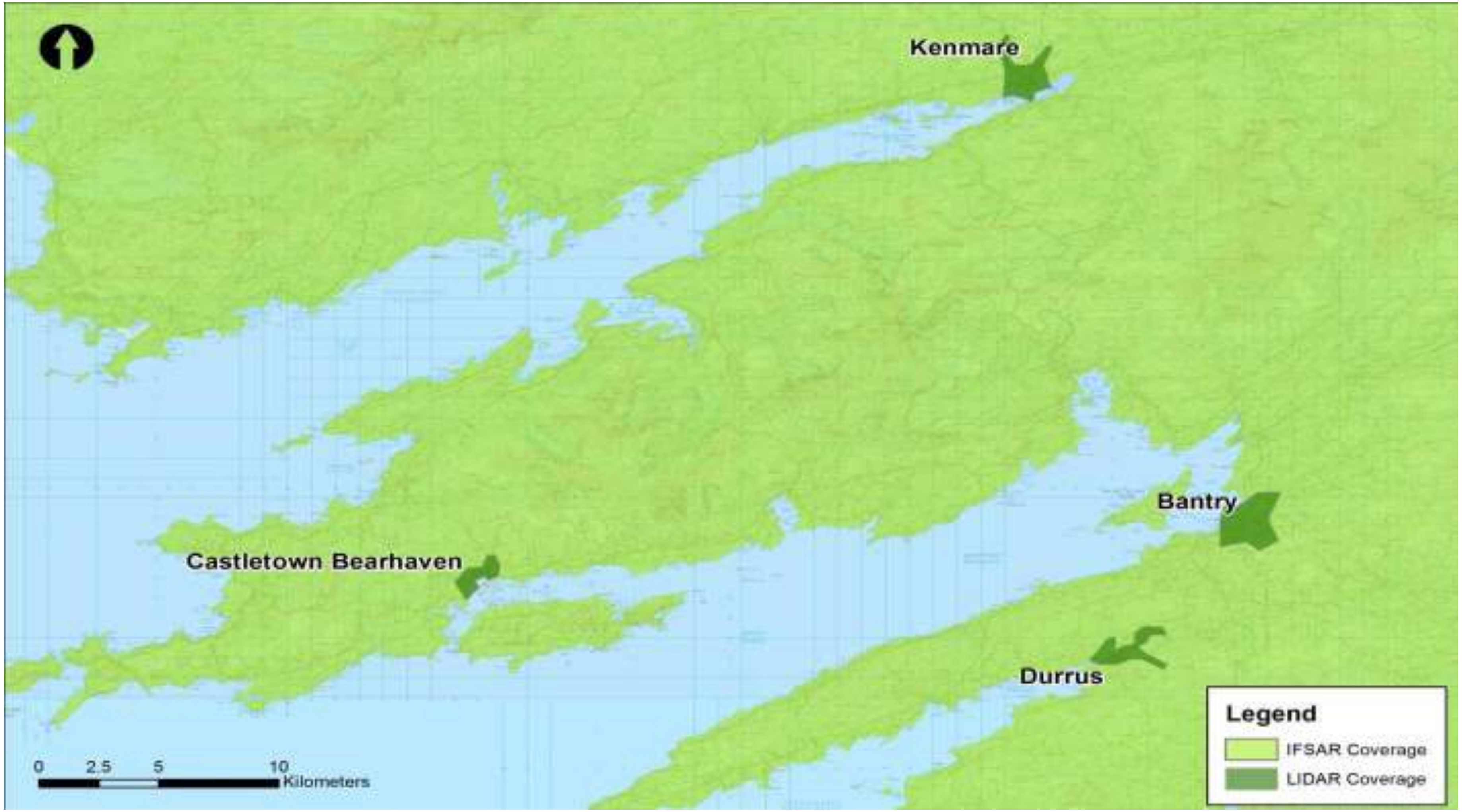
The mapping datasets were used in the first instance to classify land cover within each AFA into broad surface types of: river bed and standing water; river banks; dense vegetation; pasture, parkland and arable; buildings; and, hard-standing urban areas. The land cover was subsequently refined during the model build process using the survey 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 Coverage in UoM21



Map 2.2: LiDAR Coverage in UoM21



3 Hydrological Approach

3.1 Summary of Design Hydrology

As part of the earlier UoM21 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 along the modelled watercourses with at least one HEP within each AFA. HEPs were located 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 20005, 20006, 21001, 21004 and 22006. Table 3.1 summarises the design peak flows for each catchment in the UoM21 AFAs for ease of reference.

Table 3.1: UoM21 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
Bantry AFA									
21_5827_3	21004(Inchiclogh gauge)	87.5	106.8	120.6	135.3	157.1	176.0	197.3	258.5
21_6258_3	Mealagh downstream	96.8	118.1	133.3	149.6	173.7	194.5	218.1	285.7
21_7225_2	Bantry downstream	6.40	7.91	8.99	10.14	11.85	13.33	14.99	19.79
21_7668_2	Dromacoosane downstream	2.68	3.37	3.86	4.38	5.15	5.83	6.58	8.76
Castletown Bearhaven AFA –No fluvial flood risk was identified so design flows are not required.									
Durrus AFA									
21_8044_2	Four Mile Water Downstream	17.01	21.21	24.22	27.42	32.17	36.27	40.92	54.26
21_6225_2	Ahanegavanagh stream	6.25	7.87	9.04	10.28	12.12	13.71	15.51	20.68
Kenmare AFA									
22_3116_4	Finnihy downstream	37.15	46.21	52.70	59.61	69.86	78.72	88.74	117.53
22_3958_2	Lissaniska downstream	3.05	3.78	4.30	4.85	5.67	6.39	7.19	9.50
22_3425_9	Gortamullin east downstream	0.39	0.48	0.54	0.61	0.70	0.79	0.89	1.17

The design hydrographs for each inflow HEP 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 these 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 UoM21 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
Kenmare	ICPSS Point SW8 – ICWWS	2.12	2.22	2.28	2.35	2.43	2.49	2.56	2.70
Castletown Bearhaven	ICPSS point S3	1.99	2.09	2.16	2.22	2.31	2.38	2.45	2.60
Bantry	ICPSS point S6	2.14	2.25	2.33	2.42	2.52	2.60	2.68	2.86
Durrus	ICPSS point S9	2.09	2.20	2.28	2.36	2.46	2.54	2.62	2.79

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.

Wave overtopping volumes were derived for each flood defence section identified in the ICWWS using the EurOtop wave overtopping calculations. The use of wave overtopping volumes is appropriate where the flood extent would be limited by the volume overtopping the defence, often in locations where the defences are above the coastal floodplain. The horizontal projection of water levels plus half wave height can overestimate flood extent and risk as it does not recognise wind waves are intermittent not continuous. The horizontal projection of water levels plus half wave height can also underestimate flood risk as it does not consider wave run-up which can overtop defences even when the water level and half wave height is below crest level.

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 assessed for UoM21 was guided by the methodology set out in Flood Studies Update Work Package 3.4. In all cases, the tributaries in UoM21 were hydrologically similar to the main watercourse. The joint probabilities used are provided in Table 3.3. Given the smaller size of these catchments, it is reasonable that the same storm effects the entire catchment.

The extreme flow estimates at Inchiclogh gauge 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 joint probability between total tide plus surge levels and extreme waves has been considered separately under the ICWWS study. The resultant combinations have been assessed using wave overtopping equations and found the two main critical scenarios:

- Castletown Bearhaven: Vertical wall: Lowest still water level combined with largest wave height.
- Kenmare: Various shoreline types: Highest still water level combined with smallest wave height.

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

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%

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 upper-most 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 Dromacoosane catchment in Bantry AFA 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 shifted to consider the typical timing differences within the catchment. In UoM21, the inflow hydrographs were shifted uniformly within each hydrological catchment to ensure a physically realistic single storm event in these small coastal catchments (< 30 km^2) (Table 3.4).

The performance of the modelled peak flows compared with the design peak flows is discussed in Section 6.2 of this report.

Table 3.4: Phasing of Inflows

AFA	Sub-catchment	Time Shift Applied to the Inflow Hydrographs to Achieve the Design Peak Flows at the target HEPS (Hours)
Durrus	Four Mile Water	23.00
	Ahanegavanagh	10.80
Castletown Bearhaven	N/A Coastal Risk Only	
Bantry	Dromacoosane	8.00
	Bantry	9.25
	Mealagh	5.50
Kenmare	Finnihy	2.25

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 fluvial flow in Bantry, Durrus and Kenmare. This phasing is a conservative assumption of combined flood risk in line with the joint-probability analysis set out 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)
Bantry	Full tidal boundary at the downstream of the Mealagh, Bantry and Dromacoosane catchments.	0
Castletown Bearhaven	Full tidal boundary along the	0

Model	Downstream Condition	Time Adjustment to Coincide Peak Tide with Peak Flow (Hours)
	coast/haven.	
Durrus	Full tidal boundary at the downstream of Four Mile Water.	-10
Kenmare	Full tidal boundary along the coast and at the downstream of the River Finnihy.	-11

3.5 Critical Storm Duration

In UoM21, the design storm duration has been derived from the time to peak and SAAR applying the FSSR16 approach. The storm duration was adjusted to produce the critical hydrograph for each AFA assuming a single design storm event. The longer duration was adopted as a conservative estimate for the design scenario where the critical duration for independent sub-catchments varied within an AFA. This ensured a physically realistic single storm event in these small coastal catchments (< 30 km^2). Table 3.6 outlines the resultant critical durations for each AFA used for the design scenarios.

Table 3.6: Critical Storm Durations for Rainfall-Runoff Inflows

AFA	Sub-catchment	Area (km^2)	Theoretical Critical Duration (Hours)	Critical Duration Adopted (Hours)
Durrus	Four Mile Water	32.5	13.6	13.6
	Ahanegavanagh	6.4	7.9	13.6*
Bantry	Dromacoosane	2.4	4.5	12.9*
	Bantry	4.2	7.5	12.9*
	Mealagh	54.5	12.9	12.9
Kenmare	Finnihy	31.5	12.6	12.6
Castletown Bearhaven	Not Applicable – Coastal flooding only			

* adjusted to match the longer duration within the AFA and apply a single design event

4 Hydraulic Modelling Approach

4.1 Schematisation

Table 4.1 outlines the general approach for each AFA in UoM21. Map 4.1 presents the areas and reaches modelled.

Table 4.1: UoM21 Model Approach

Model ID	AFA	Approach	No. Models	Area Modelled (km^2)	Length Modelled (km)	Upstream Limit	Downstream Limit
I29DS	Durrus	1D/2D ISIS/TUFLOW	1	2.2	4.3	095419,042630 093956,042304	093068,041350
I30BY	Bantry	1D/2D ISIS/ESTRY/ TUFLOW	1 *	10	14.2	101050,048450 101664,050254	098961,048537 099772,049870
I31CN	Castletown Bearhaven	2D TUFLOW	1	1.6	N/A	N/A	N/A
I32KE	Kenmare	1D/2D ISIS/TUFLOW	1	2.3	4.7	090072,071935	090025,070199

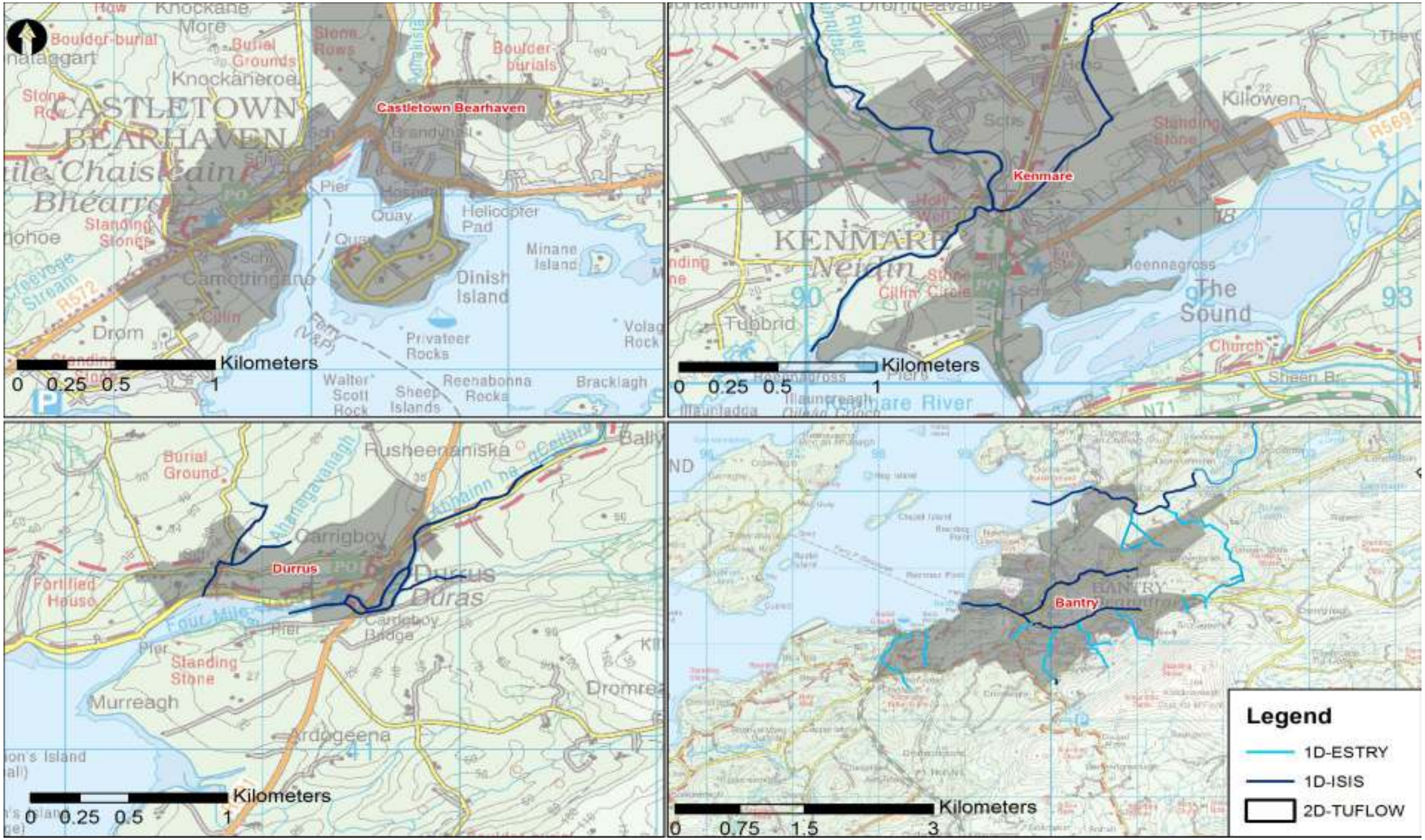
* The Bantry model can be separated into the 3 sub-catchments (Mealagh, Bantry and Dromacoosane) as the flows do not interact between subcatchments. However, it has been combined into a single 1D-2D model with separate elements to streamline processing for the purposes of this study.

A hydrodynamically linked one-dimensional (1D) and two-dimensional (2D) approach has been taken for Durrus, Bantry and Kenmare. However, a 2D only approach has been taken for Castletown Bearhaven as the AFA is subject to coastal flood risk only. The majority of HPWs have been modelled in ISIS one-dimensional modelling software (version 3.6.0) to simulate in-bank flows as it 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 along Milleencoola West, Ardnageehy, Knocknavaghaea East, Sheskin, Carignagat, Dromleigh, Kilnruane and the entire Dromacoosane catchment in Bantry. 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 reaches modelled in ISIS using the ISIS-TUFLOW-PIPE link which allows flow and water levels to be exchanged. The ESTRY-ISIS network is then connected with the 2D TUFLOW model of the floodplain to simulate flood hazard in the AFAs. Knocknavaghaea West (KNWT) was not modelled as the catchment was less than 1km^2 and the local engineers did not identify a flow path or watercourse along the west of Millbrook Estate.

TUFLOW two-dimensional modelling software (version 2012-AC-05) 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 coastal flooding, such as found in Castletown Bearhaven, as it is able to simulate the multi-directional flow paths as the sea overtops the quaysides, coastal roads and sea walls. 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 UoM21. However, the following modifications were made during the modelling process for open channel sections:

- Additional river channel sections have been interpolated for the tributaries in Bantry to stabilise flow over the steep gradients based on the surveyed and DTM slope.
- Right bank levels were also raised at the upstream of the Reenrour tributary to improve the 1D-2D interface and reduce circulation in a small flooded area at the upstream limit.
- Right bank levels were lowered at the back of Glengariff Road to represent the variable garden walls and banks following discussions with the local engineers on site.
- In Durrus, the bed levels were modified around the confluence of Four Mile Water North and South to remove localised scour holes in order to stabilise the model. This does not affect water level because the higher bed levels downstream of the confluence control the level.

The river channel gradient, width and shape can vary rapidly on the approach and exit of bridges which is not necessarily representative of the broader 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 upstream or downstream reach than those directly at the structure face.

The exception are the bridges in Kenmare where the survey section immediately upstream of the bridges through the town centre was deemed to be representative of the upstream gradient and channel shape due to the short distance between the bridge structures.

The surveyed left and right floodplains beyond the bank crests were deactivated in the 1D models in order to avoid double counting the floodplain volume with 2D floodplain model. The deactivation point on each cross-section tried to avoid rapidly changing the channel width to minimise instability in the 1D model and circulation around a jagged 1D/2D interface.

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 section 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 Finnihy Lissaniska Stream
Urban channel or natural channel with river silts	0.040 to 0.045	River Mealagh Bantry River and tributaries Dromacoosane Stream Four Mile Water
Light brush and/or grass during winter	0.060 to 0.075	Four Mile Water River Mealagh Bantry River Dromacoosane Stream
Dense vegetation year round	0.075 to 0.080	River Finnihy Lissaniska Stream

Source: Chow 1959

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 AFA. 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 model 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 Photograph and Schematic

The simplification of structures in UoM21 is discussed in the following sections. There were no operable structures within the UoM21 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 UoM21:

- 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, with 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, with 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 most widely in UoM21, with only Chapel Street Bridge in Bantry and East Park Lane Bridge in Kenmare using the Bernoulli Loss approach to better simulate the head losses and stabilise the transition to orifice flow. The Bernoulli Loss K values were estimated using standard industry guidance (Chow 1959), combined with engineering judgement of similar structures. The K values were then adjusted during the calibration process to achieve the observed water levels and flooding in this area.

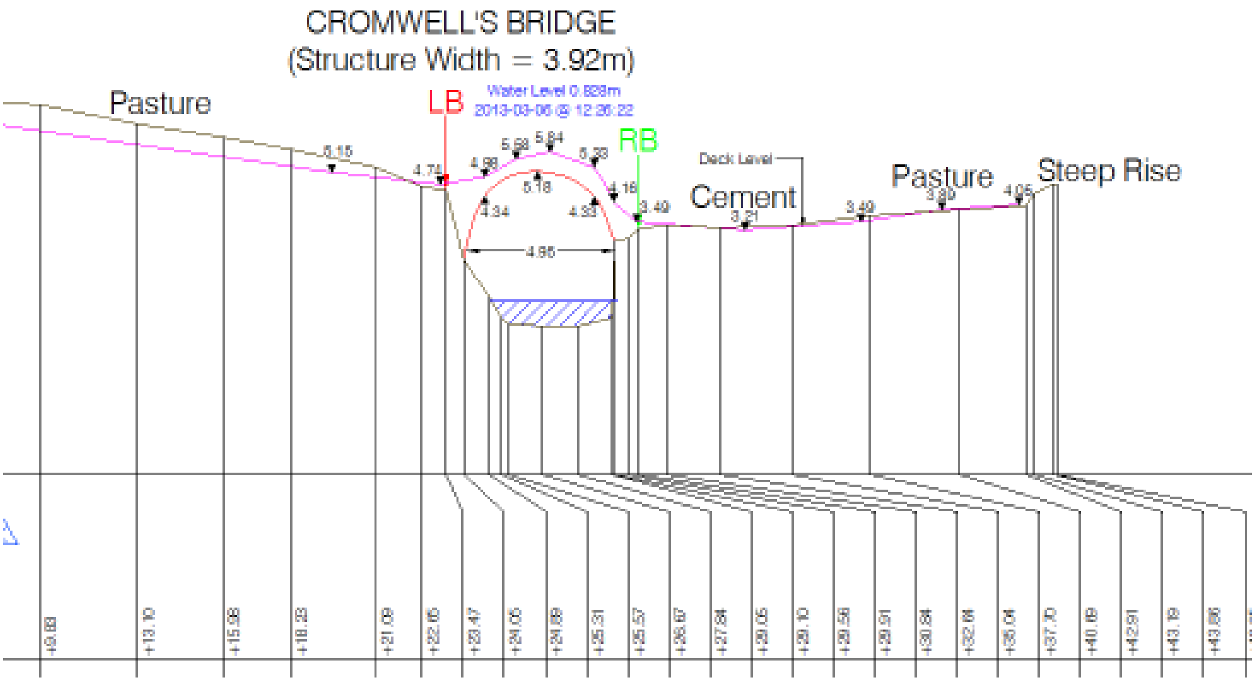
Photo 4.1: Finnihy Bridge

Captured: 08 March 2013

In UoM21, there were a number of bridges with utilities crossing immediately upstream or under the bridge structure, obstructing the bridge flow and increasing head loss before the soffit was reached (Photo 4.1). The modelled soffit elevation was lowered to the pipe soffit level because the turbulent flow above this level would be turbulent and the gap liable to blockage. This is a conservative estimate of head loss for flood mapping purposes. However, a sensitivity test on this assumption was undertaken for Kenmare as described in Chapter 5. All pipes below soffits have been considered in modelling assumptions for those bridges.

Two footbridges at the golf course and Cromwell’s Bridge in Kenmare were not modelled because the bankfull level at the section or upstream was at or lower than the springing level of the bridge. In the example of Cromwell’s Bridge the bank level upstream was 2.6 mAOD and the springing level was at 2.86mAOD and the bridge did no constrict the channel with piers below the springing level. Therefore, the bridge structure would by-passed before the water level was affected by the bridge structure (Figure 4.2).

Figure 4.2: Cromwell's Bridge Kenmare



Culverts

Culverts were modelled in ISIS using; i) a culvert inlet to simulate losses associated with the constriction of flow at the entrance; ii) appropriate sized and shaped conduit units; 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.

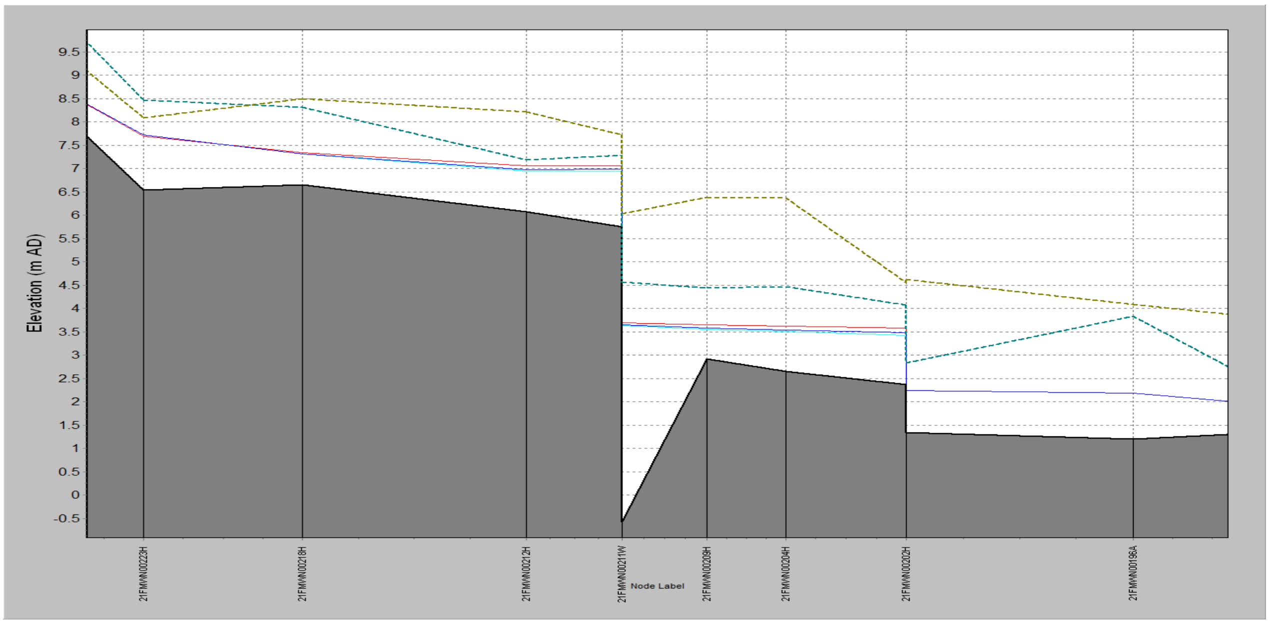
Culverts modelled in ESTRY on the Bantry tributary reaches were based on the survey structure dimensions and contraction and expansion losses calculated using the recommended coefficients from Capacity Charts for the Hydraulic Design of Highway Culverts (Henderson, 1996). Losses associated with trash 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 Bantry, and other informal weirs/natural bed drops such as the waterfalls in Kenmare and Durrus, 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.

The waterfalls in Durrus were modelled as spill units with a weir coefficient of 1.5. It is not easy to verify the spill coefficient for non-formal weirs without flow data. Decreasing spill coefficients to represent greater flow inefficiencies resulted in a maximum 0.05m increase with a coefficient of 1.3 and a maximum of 0.12m with a coefficient of 1.0 for in-bank flows at the waterfalls (Figure 4.3). However, the level difference reduces to < 0.05m once out-of-bank as the floodplain became the effective weir crest. The 50%AEP peak water level is already at bankfull around the weirs, therefore the impact of the spill coefficient on flood risk and thresholds of flooding is negligible

Figure 4.3: Sensitivity of the Water Level Profile to the Spill Coefficient Used at Durrus Waterfalls



Water Level 1.5 Spill Coefficient at Waterfalls
Water Level 1.3 Spill Coefficient at Waterfalls (assuming less efficiency than an formal weir structure)
Water Level 1.0 Spill Coefficient at Waterfalls (assuming less efficiency than an formal weir structure)
Bed Level
Left Bank
Right Bank

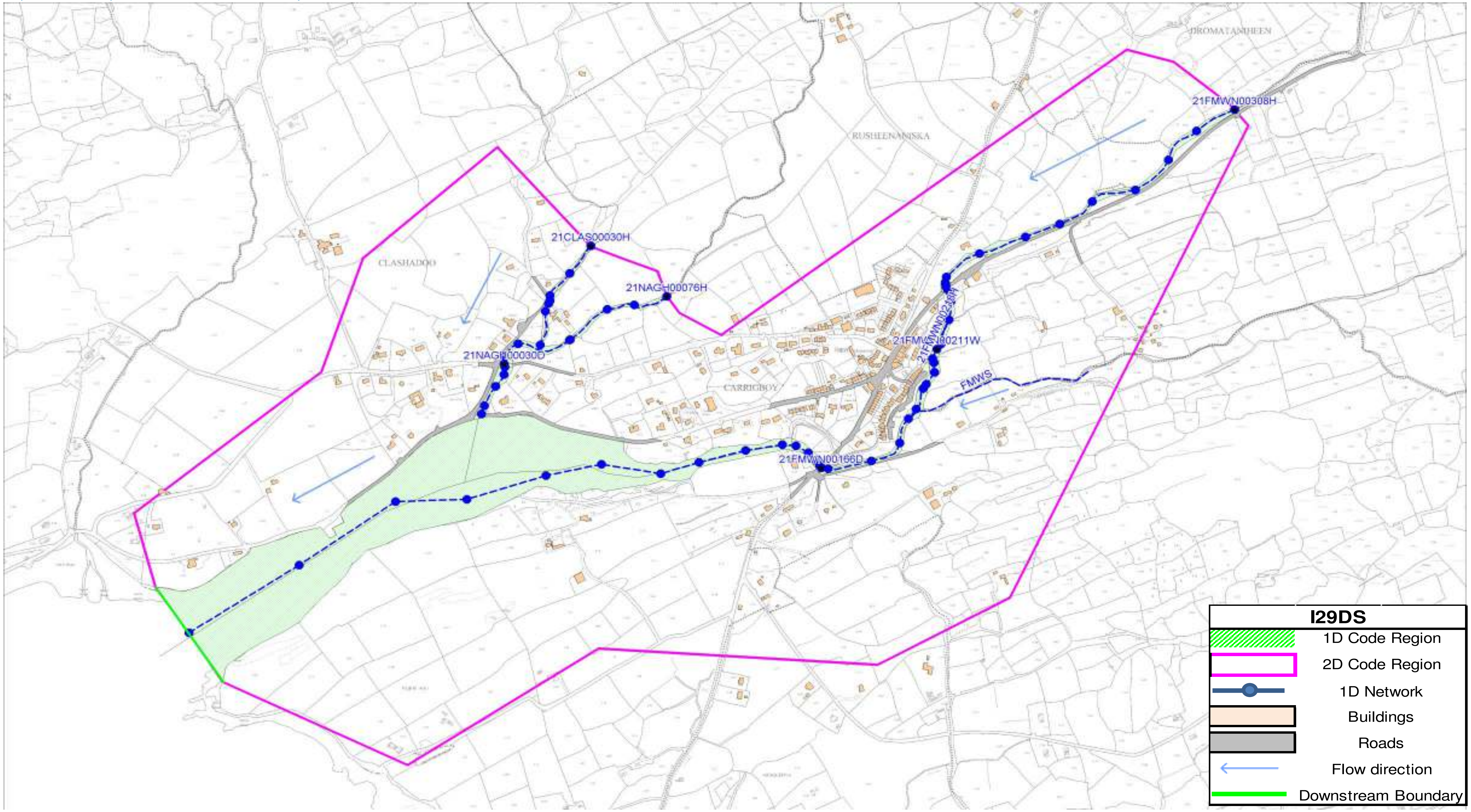
4.4 Floodplain

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 adequately representing the complex urban nature of these AFAs whilst avoiding overly long run times. Map 4.2 presents an example for Durrus.

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 at a sub-grid level to ensure representative conveyance between buildings and along key ditches. Other 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 the Durrus Hydraulic Model



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. The buildings were assigned a Manning's 'n' value of 0.20 to simulate the storage and reduction in velocity through the buildings once water was above the threshold value of 0.15m.

Syme (2008)[2] tested different methodologies of representing buildings including blocking out, Manning's 'n' and cell blockage approaches. Syme found the increase in water levels due to the different representation of buildings were all within 0.04m of each other with a standard deviation of 0.03m (Table 3.2 Syme 2008).

The blocked out methodology presents a more "visually correct" representation of flow paths around the building but does not simulate the effects of storage within the building and does not produce a representative flood level. Therefore, the Manning's 'n' approach combined with the building threshold approach has been selected to represent the impact of building whilst providing a representative flood level for subsequent damage calculations. 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, such as sandbags, has been considered when comparing model results with historic flood extents.

The roads in UoM21 are typically 8 to 12 m 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'. 4.3

[2] Syme (2008) Flooding in Urban Areas - 2D Modelling Approaches for Buildings and Fences. Engineers Australia, 9th National Conference on Hydraulics in Water Engineering. Darwin Convention Centre, Australia 23-26 September 2008

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 UoM21 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.

In Bantry, the orifice linearization was increased from 0m (Default) 0.1m to stabilise the transition to orifice flow of the many steep culverts. 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[3].

Table 5.1: Selection of Calibration Events

Event	Model	Source of Flooding	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
23/10/2008	Kenmare	Fluvial	1	0	3	3	3	10	Calibrate main channel and coastal flood risk to large event data. Smaller tributaries in Kenmare should take note of uncertainties due to blockage.
17/10/2012	Bantry	Coastal	1	0	3	3	3	10	Calibrate main channel and coastal flood risk to large event data. Smaller tributaries within the Bantry catchment and the Mealagh catchment should take note of uncertainties due to blockage.

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

There were reports of road flooding during the 2000 and 2004 events in Bantry. However, there was no information on the extent of flooding, properties affected or levels to calibrate the hydraulic model to. Changes to the urban drainage systems since 1983 and limited hydrometric data made it difficult to undertake a full calibration for the severe flooding in Bantry in 1981 and 1983. However, the local engineer's report's that the Mealagh valley was flooded regularly and the business park was flooded once every 5 to 10 years have been used to validate the flood extents of the more frequent %AEP events.

Sensitivity analysis has been used to further assess hydraulic parameters for Bantry and Kenmare and to validate Castletown Bearhaven and Durrus models where there was insufficient data to calibrate the hydraulic model.

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

5.1.1 23rd October 2008

On the afternoon of the 23rd of October 2008 Kenmare Main Street was flooded to depths of over 0.5m. The event resulted in water coming out-of-bank along the Finnihy River and the Lissaniska Stream, affecting 37 residential and 11 commercial properties. The Kenmare Area Office distributed 300 sandbags during the event to mitigate property flooding. A private contractor, who was undertaking construction works at Kenmare Heritage Trail footbridge, also removed the metal parapet during the event to alleviate flood levels upstream.

The quality of the historic flood data from the post flood report [4] has been reviewed:

- Photographs
  - The photographs were taken during the flood with date and time marked.
  - These are deemed to be an accurate representation of flooding snapshots during the event.
- Levels
  - Peak water levels were marked by local authority staff immediately after the flood.
  - These levels are deemed to be reliable as they were observed immediately after the event, but the wrack marks could be influenced by local wash (natural or traffic) or capillary action on plaster walls. Therefore, the levels are deemed accurate to within 0.1m.
- Extent
  - It is not entirely clear how the flood outline was identified from the report but it is assumed that it was drawn from a combination of site observations, the photographs and levels referred to above.
  - The extent is deemed to be accurate, but may differ from the model assumptions as it considers the effects of sandbags.

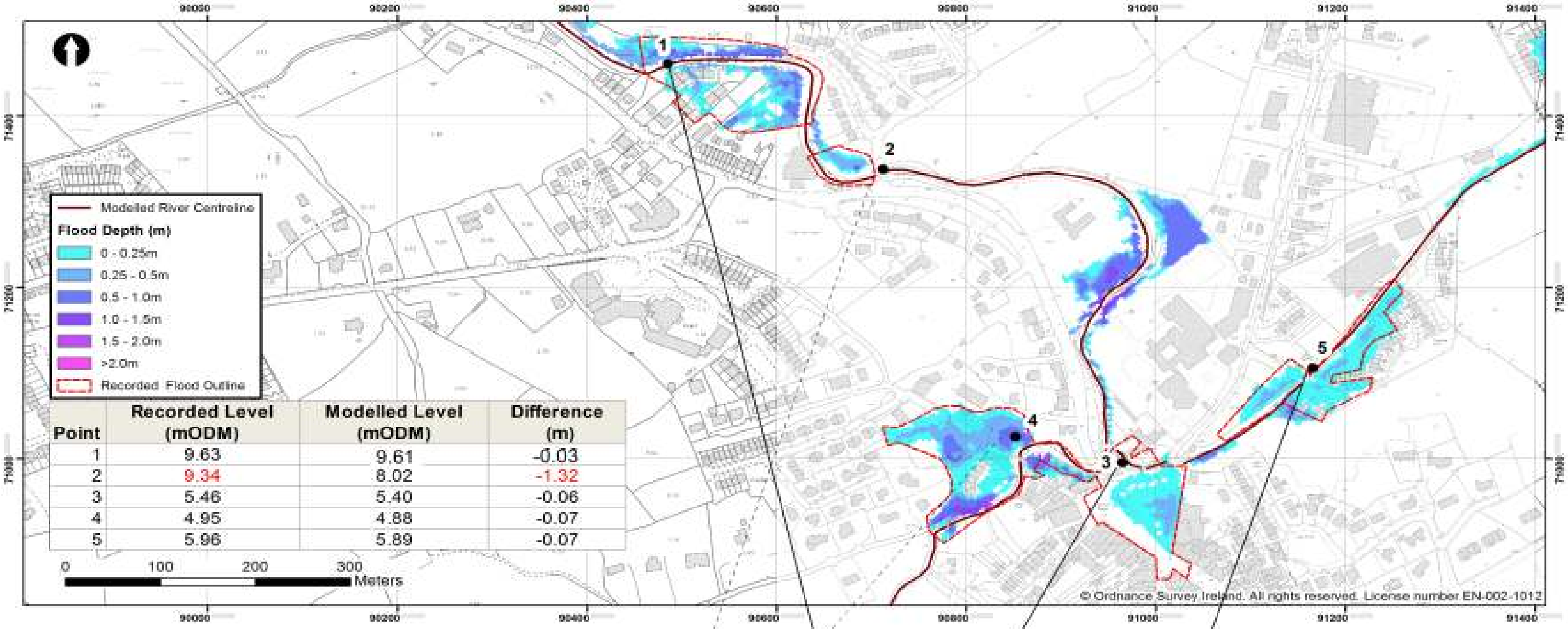
The design hydraulic model was modified as follows to represent the hydrological and hydraulic conditions of this event:

- The rainfall profile was transferred from Valentia Observatory and hydrographs produced using the FSSR16 rainfall-runoff approach with percentage runoff increased to 77% to represent the saturated conditions indicated in the Met Eireann observed SMD measurements, and phased to meet the target levels at Finnihy Bridge.
- The design tide plus surge curve was scaled and phased to meet the predicted tide from the Admiralty Tide Tables at Kenmare. Observed data at Castletown Bearhaven indicated the surge residuals were negligible for this period. Therefore the predicted astronomic tide is applicable.
- The Kenmare Heritage Trail footbridge was partially blocked to represent the constructions works which were being undertaken at that time and replicate the backwater upstream. The spill coefficient over the bridge parapet was increased to represent the improved efficiency of flow over the bridge with the removal of the railings.

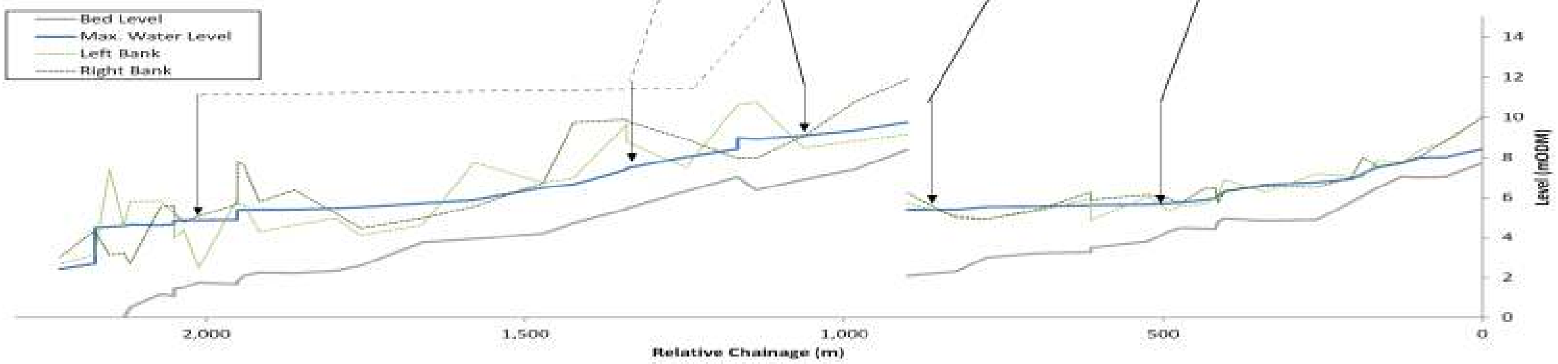
The hydraulic parameters were adjusted to best match the flood levels and extents in Kenmare, including: Manning's 'n', the loss coefficients over the stepping stone "weir", and, the loss coefficients at the Creamery Bridge. Map 5.1 compares the resultant model extent and levels with the recorded information.

[4] Kerry County Council (Feb 2009) Report on Flooding in Kenmare Town on 23rd October 2008.

Map 5.1: Calibration of Kenmare Model to 23 October 2008 Event



Point	Recorded Level (mODM)	Modelled Level (mODM)	Difference (m)
1	9.63	9.61	-0.03
2	9.34	8.02	-1.32
3	5.46	5.40	-0.06
4	4.95	4.88	-0.07
5	5.96	5.89	-0.07



The calibrated model results match well with the recorded flood outline, levels and photographs at the Creamery Bridge and Scarteen Park. The inclusion of the stepping stones infill survey (April 2014) improved the calibration of the flood extent at Finnihy Banks Estate. It should be noted that the flood extent in this area was partly caused by pluvial flooding as well from surface water runoff down the road. The CFRAMS model does not consider pluvial flooding. The peak flows was roughly equivalent to the 10%AEP on the Finnihy. However, the blockage at footbridge increased flooding in the town which was more similar to the 2%AEP design flood extent.

There were two locations where the model results differed to the recorded outline:

Photo 5.1: Wrack Mark at Riverside Villas Bridge

Source: Kerry County Council 2008 Post-Flood Report

Riverside Villas Bridge (Point 2)

The modelled flood level is significantly below the recorded level, although the flood extent seems to be a reasonable match. The survey and hydraulic model indicate that there is little head loss at this structure until water levels reach the soffit. However, the channel gradient downstream is relatively steep, and there is no obvious mechanism to limit flow downstream and cause backwater that would raise water levels to the bridge soffit. The recorded flood level was based on wrack marks in the field upstream of the bridge and is noted in Photo 5.1 as only being accurate to 1m in horizontal plane. This could equate to 0.3 to 1m difference in elevation on the steep ground depending where the surveyed level was taken. Furthermore, the recorded flood outline was found to intersect the DTM at approximately 8.5mODM. Therefore, there is up to 0.8m uncertainty in the observed level which should be considered when comparing with the modelled results.

The discrepancy at this location does not affect performance upstream of the Steeping Stone's weir or other locations downstream in the model.

Convent (500m downstream of Riverside Villas Bridge)

The model predicts flooding in the grounds of the convent and on the opposite bank up to the School boundary. Whilst this flooding was not reported at the time, interviews with the convent during the flood risk review indicated that the low-lying areas of the garden were frequently inundated. Therefore, it is not unreasonable to expect this location to flood during this extreme flood.

5.1.2 17th October 2012

A large storm surge of approximately 0.8m above the predicted tide resulted in flooding of Wolfetone Square in Bantry. The coastal walls have a number of openings where the rising tides flowed through to flood a number of properties along The Quay and Bridge Street.

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

- Photographs
  - The photographs were taken of locations flooded 26 days after the event.
  - These are deemed to be accurate representation of locations flooded.
- Flood Depths
  - Depth of flooding was recorded at the properties flooded 26 days after the event based on wrack marks and interviews with owners, and are considered accurate to within 0.1m due to the time elapsed after the event and use of wrack marks.
  - In some cases, steps into the property limited the progression of flooding into the building.
- Extent
  - The recorded extent has been identified from interviews with the local residents in the flood report.
  - This method may not necessarily pick up the maximum extent, which would have occurred in the early morning before most residents could observe it.

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

- The recorded tide plus surge conditions at Castletown Bearhaven tidal gauge were transferred to Bantry based on the design water level profile in Bantry Bay. A peak water level of 2.3mODM was estimated, which is just less than the design 10%AEP total tide plus surge level.
- There were no reports of river flooding for this event. Therefore in-bank flows have been applied for the fluvial inputs.
- The openings in the sea wall along The Quay mean that the wall above road level is an ineffective flood defence and thus has not been represented in the model as per the CFRAMS brief. This assumption is appropriate as the calibration event is significantly above the threshold level (>0.3m) and tidal flood risk is level, not volume, dependent in Bantry.

The hydraulic parameters were adjusted to best match the flood levels and extents in Bantry including Manning's 'n' values. Map 5.2 compares the resultant model extent and levels with the recorded information.

The calibrated model results match well with the recorded flood outline and reported flow routes along the The Quay and into Wolfetone Square. There is a 0.05 to 0.1m discrepancy in level at the southern end of Wolfetone Square, which in level results in more extensive flooding along New Street and Glengarriff Road. The fluvial flows do affect levels in the harbour. Furthermore, inspection of the TUFLOW timestep

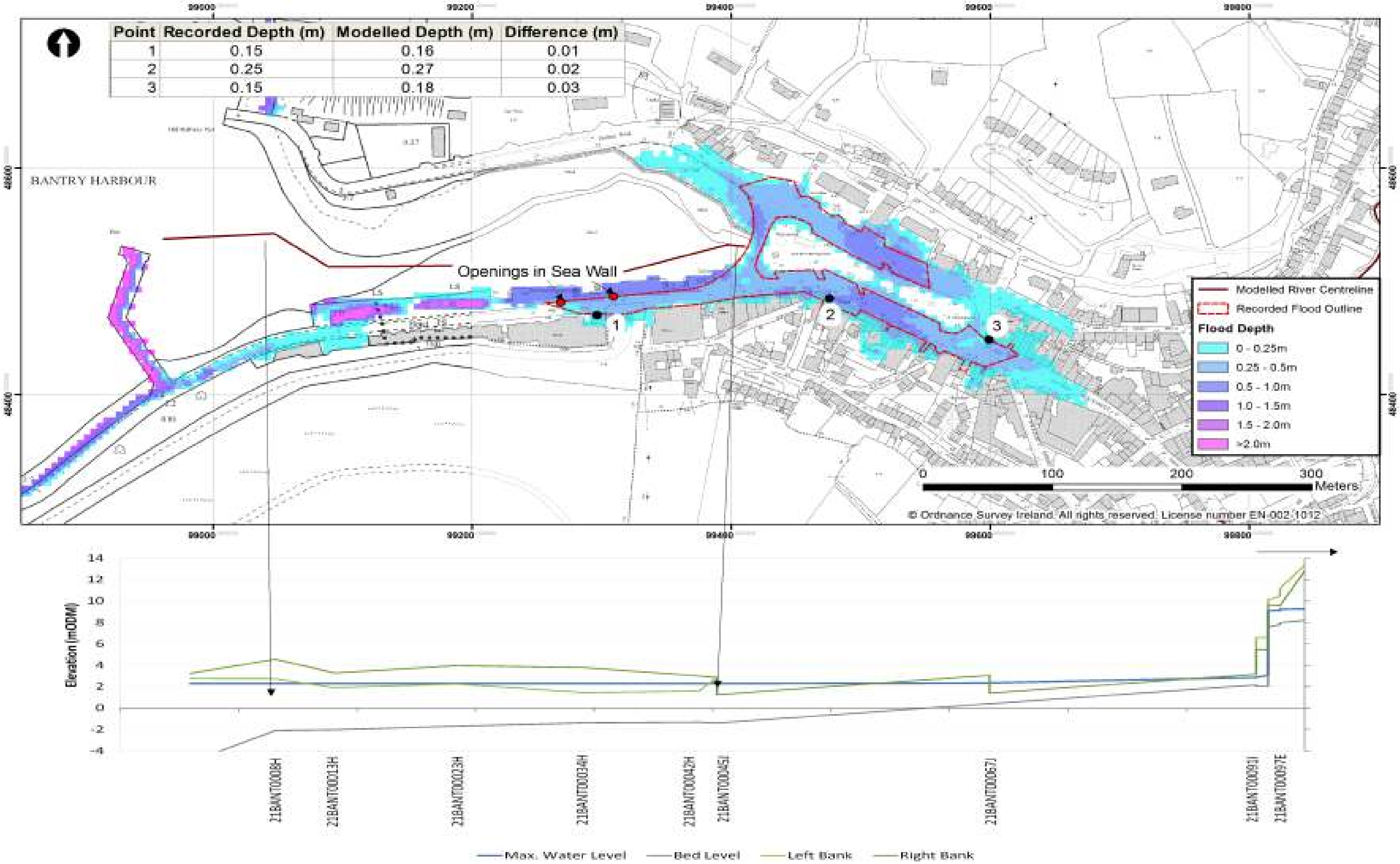
[5] Mott MacDonald (Nov 2012) 17 October 2012 Flood event Report Form, Bantry.

results indicates that the flooding along New Street and Glengarriff roads results entirely from the overtopping of the quayside and the river banks are not overtopped.

This discrepancy in the total tide plus surge level may result from transfer of the water level from Castletown Bearhaven to Bantry differing to the design profile assumed in the ICPSS analysis. However, the water level is within 0.01m of the recorded level at the quayside. Therefore, the water level transfer is deemed reasonable for this event.

However, the discrepancy between modelled and recorded levels increases inland to New Street. The pumped urban drainage system is not considered in the model which may result in the modelled extent being larger than the observed. There is also uncertainty in the recorded levels which are reliant on the accuracy of DTM which has a RMSE of +/-0.2m.

Map 5.2: Calibration of the Bantry Model for 17 October 2012 Event



Point	Recorded Depth (m)	Modelled Depth (m)	Difference (m)
1	0.15	0.16	0.01
2	0.25	0.27	0.02
3	0.15	0.18	0.03

5.1.3 Summary

Table 5.3 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 required ±0.1m of the recorded levels for the calibration events.

Table 5.2: Summary of Calibration Performance

Event	Reliability of Recorded Level and Extents	Location	Absolute Difference to Recorded Level/Depth (m)	Average Error to Recorded Levels/Depths (m)	Root Mean Square Difference
28 October 2008	Extents deemed to be reasonably accurate in areas of property flooding but consider sandbags. Levels reliable within 0.1m except at Riverside Villas which suggest a much flatter water level profile than observed in the river survey	Kenmare, Rose Cottages (minimum difference)	+0.06	0.07 (-0.32 if Riverside Villas Bridge is included)	0.07 (0.59 if Riverside Villas Bridge is included)
		Kenmare, Riverside Villas Bridge (maximum difference)	-1.32		
17 October 2012	Extents deemed to be reasonably accurate in areas of property flooding Levels subject to +/-0.2m based on DTM	Bantry, Quayside (minimum difference)	+0.01	0.03	0.03
		Bantry, New Street (maximum difference)	+0.1		

The Kenmare model matched well in and around the centre of Kenmare town, but under predicted the flood level at Riverside Villas Bridge due to uncertainty in the recorded level at this location. Flood levels around the bridge should be treated with caution, however the flood extent is likely to be reliable as the floodplain is relatively constrained.

The Bantry model tended to slightly over-predict flood risk along New Street and Glengarriff Road. However, this is likely to be due to the water level profile from Castletown Bearhaven being different to the design water profile for this event. Sensitivity analysis on the downstream level has been undertaken in the following section to assess this uncertainty.

5.2 Sensitivity Analysis

5.2.1 Flow

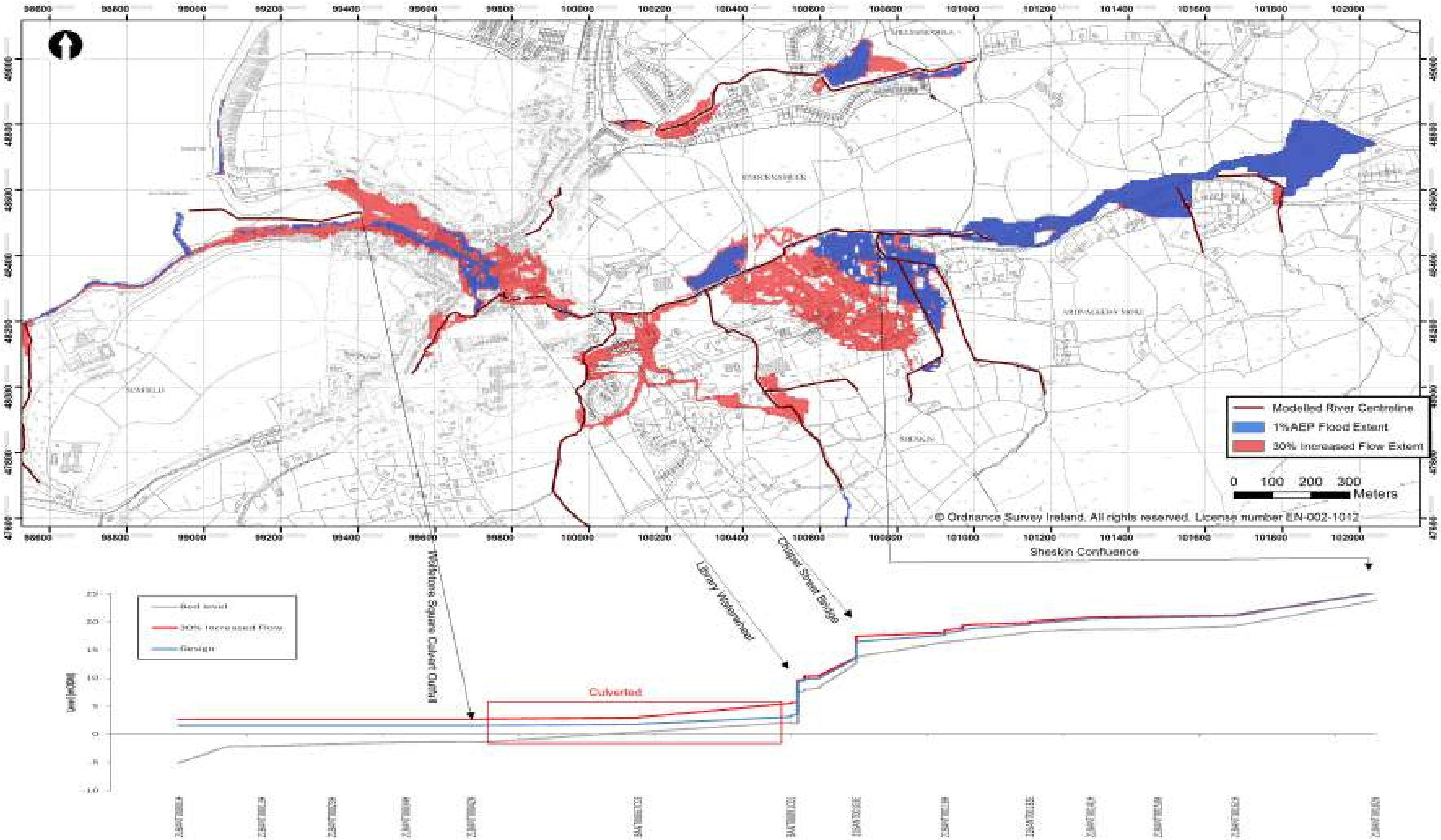
In accordance with CFRAM Guidance Note 22, the 1%AEP design peak flow was raised by 30% to assess the sensitivity to uncertainties in the QMEDrural coefficients, the selection of pivotal sites and the flood growth curves derived in the hydrological analysis. This is approximately equivalent to the flow 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%.

In UoM21, the Bantry River (Map 5.3) and Finnihy River (Map 5.4) were the most sensitive to assumptions in peak flow due to the limited capacity of the various bridges and culverts along these watercourses. The increased flows exceed the capacity of these structures, spill over the river banks and flow rapidly down the roads to flood a greater extent than the design scenario.

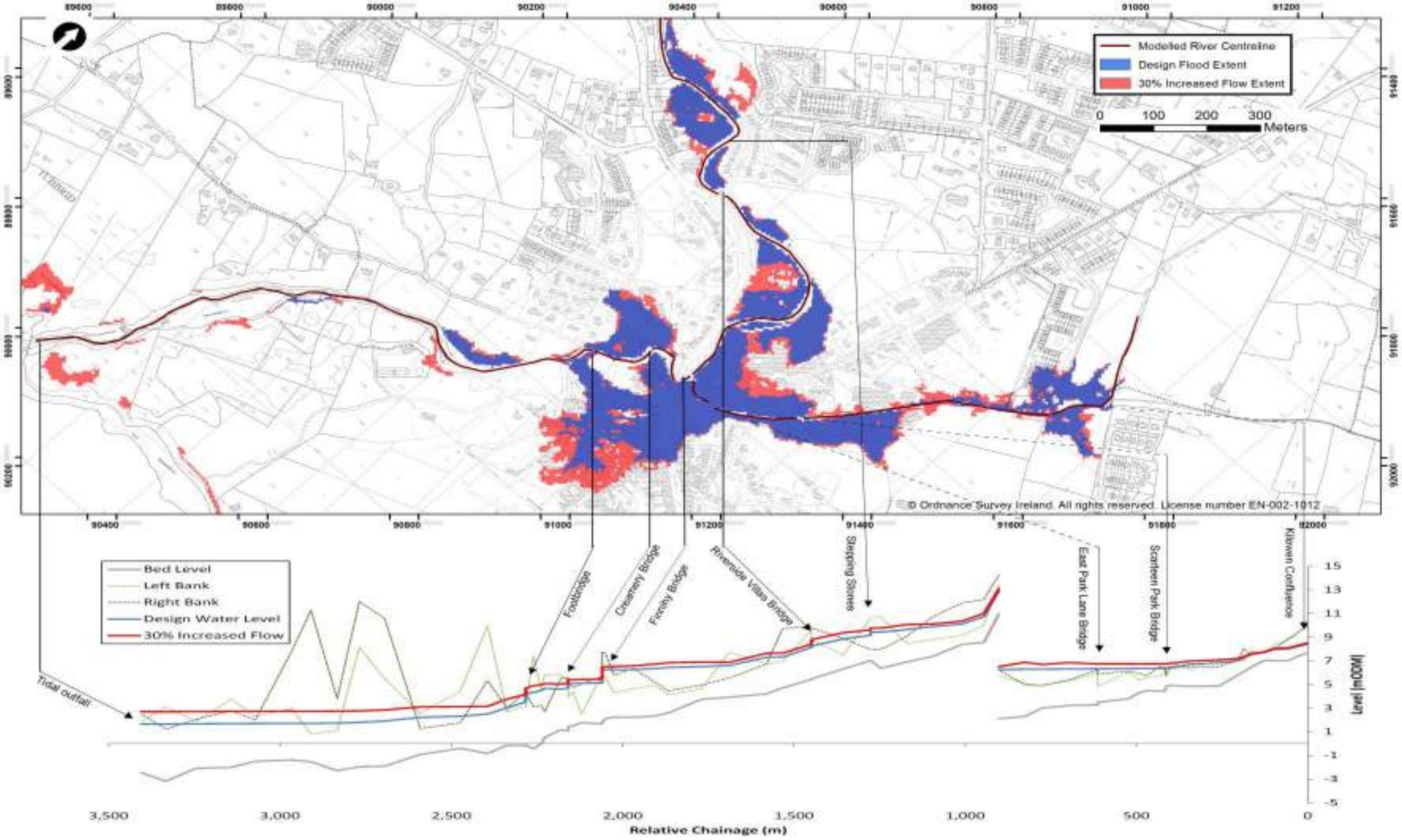
Durrus, the Dromcarra catchment and the Mealagh catchment are less sensitive to the assumptions in peak flow, as their narrow floodplains are already inundated in the design scenario. Therefore, the increase in water level does not significantly increase areas at flood risk, although depth of flooding and risk to life increases slightly with the more extreme conditions.

The plots for all flow sensitivity tests can be found in the model performance proformas in the relevant Appendices.

Map 5.3: Sensitivity to Peak Flow – Bantry Town Model



Map 5.4: Sensitivity to Peak Flow-Kenmare



5.2.2 Level

A sensitivity test was undertaken on downstream water level for tidally-affected AFAs in UoM21 (i.e. Bantry, Castletown Bearhaven and Kenmare). 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).

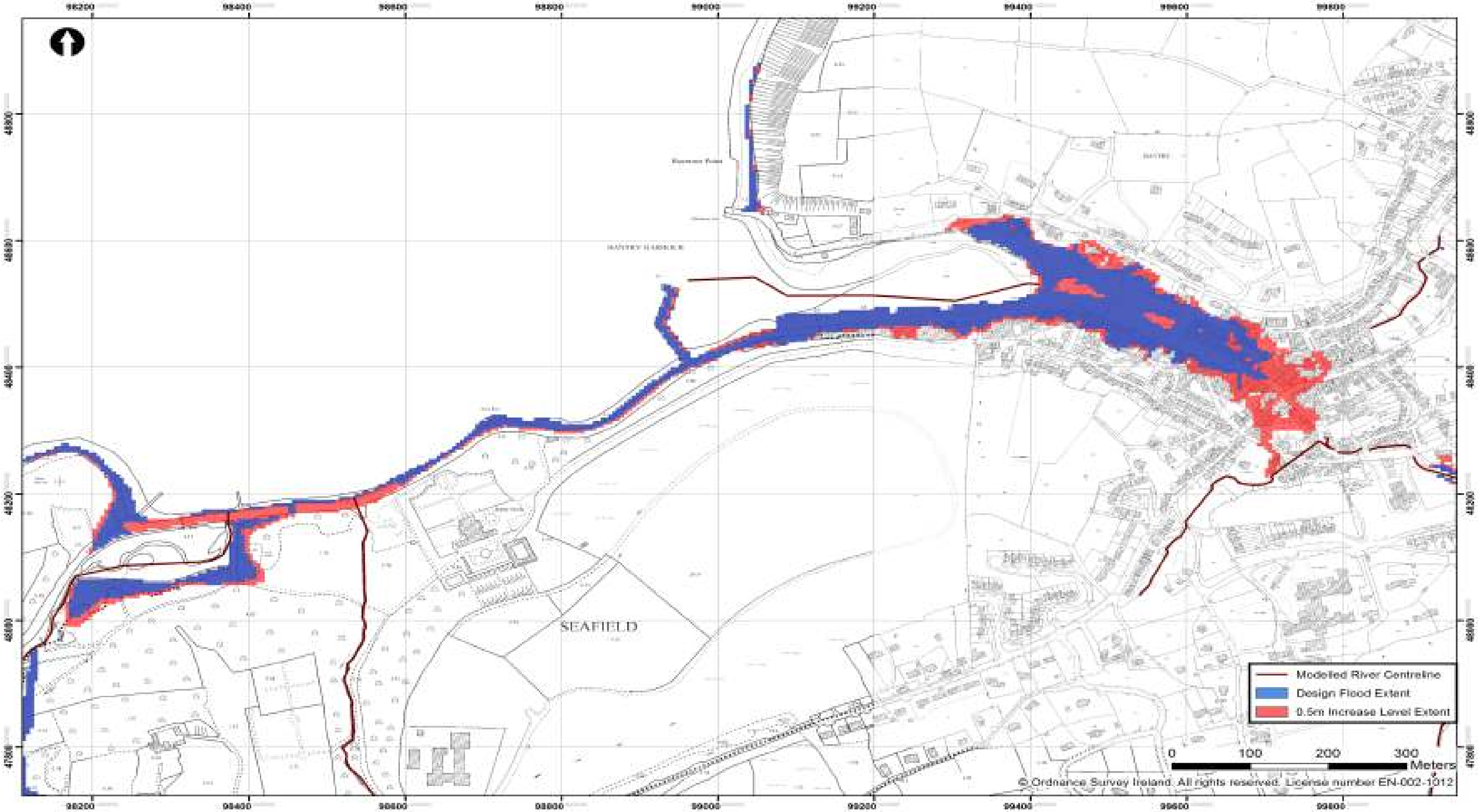
In UoM21, flood level and extent was sensitive to the downstream level in Bantry Town catchment (Map 5.5), Castletown Bearhaven (Map 5.6) and the Reenaross and Pier Road areas at Kenmare (Map 5.7). The increase in water level results in more extensive coastal flooding as more water spills over the quayside for a longer duration.

The Drommcarra catchment and Mealagh catchment in Bantry were less sensitive to the downstream level, as the tidal conditions do no interact with the river flows upstream due to steep bed gradients. A sensitivity test on downstream level was not undertaken for Durrus, as flood risk in the town is not tidally affected.

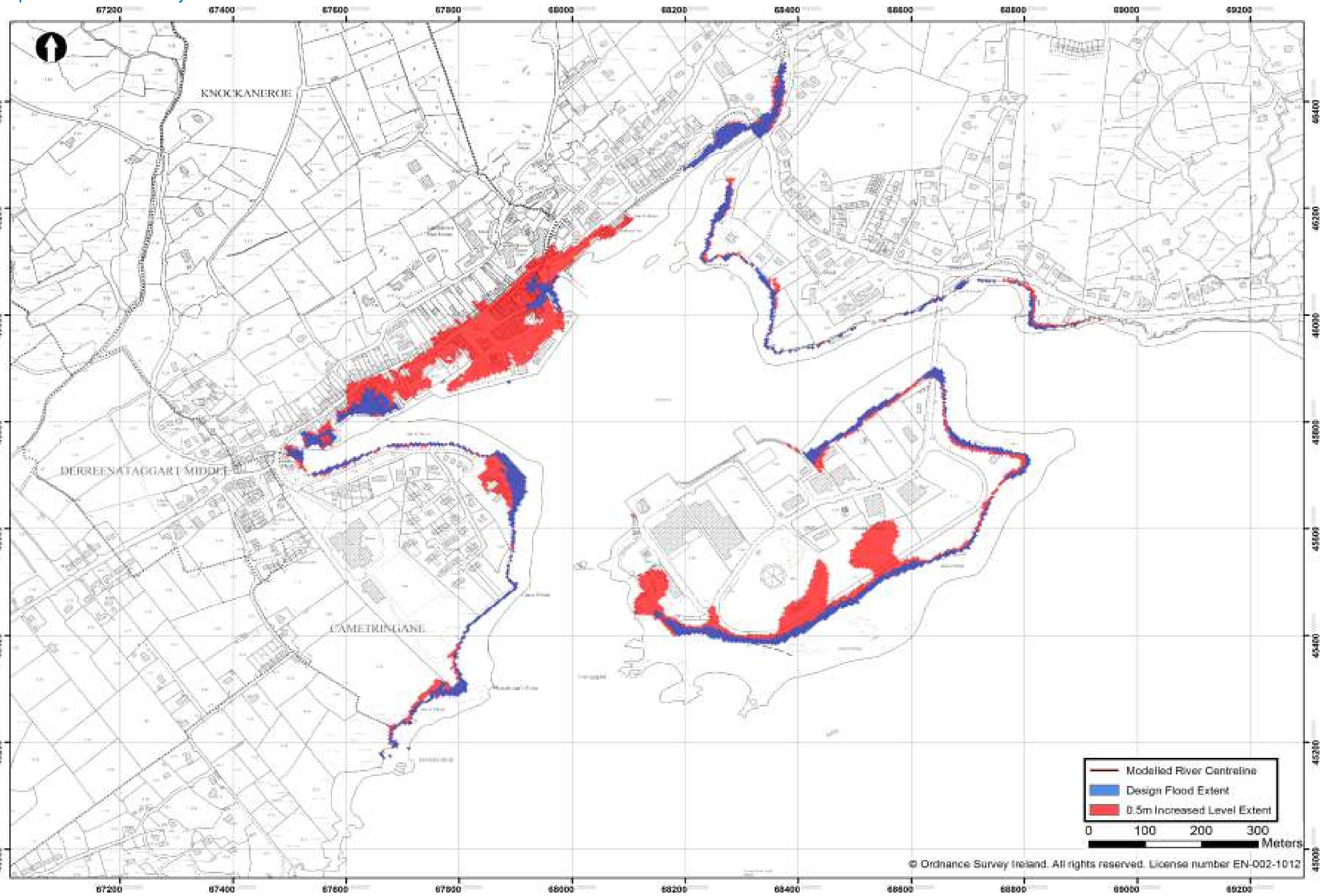
Water level gauging in Bantry Habour is recommended to verify the ICPSS profile between Castletown Bearhaven and Bantry, thereby improving the confidence in the extreme total tide plus surge levels at Bantry.

The plots for all level sensitivity tests can be found in the model performance proformas in the relevant Appendices.

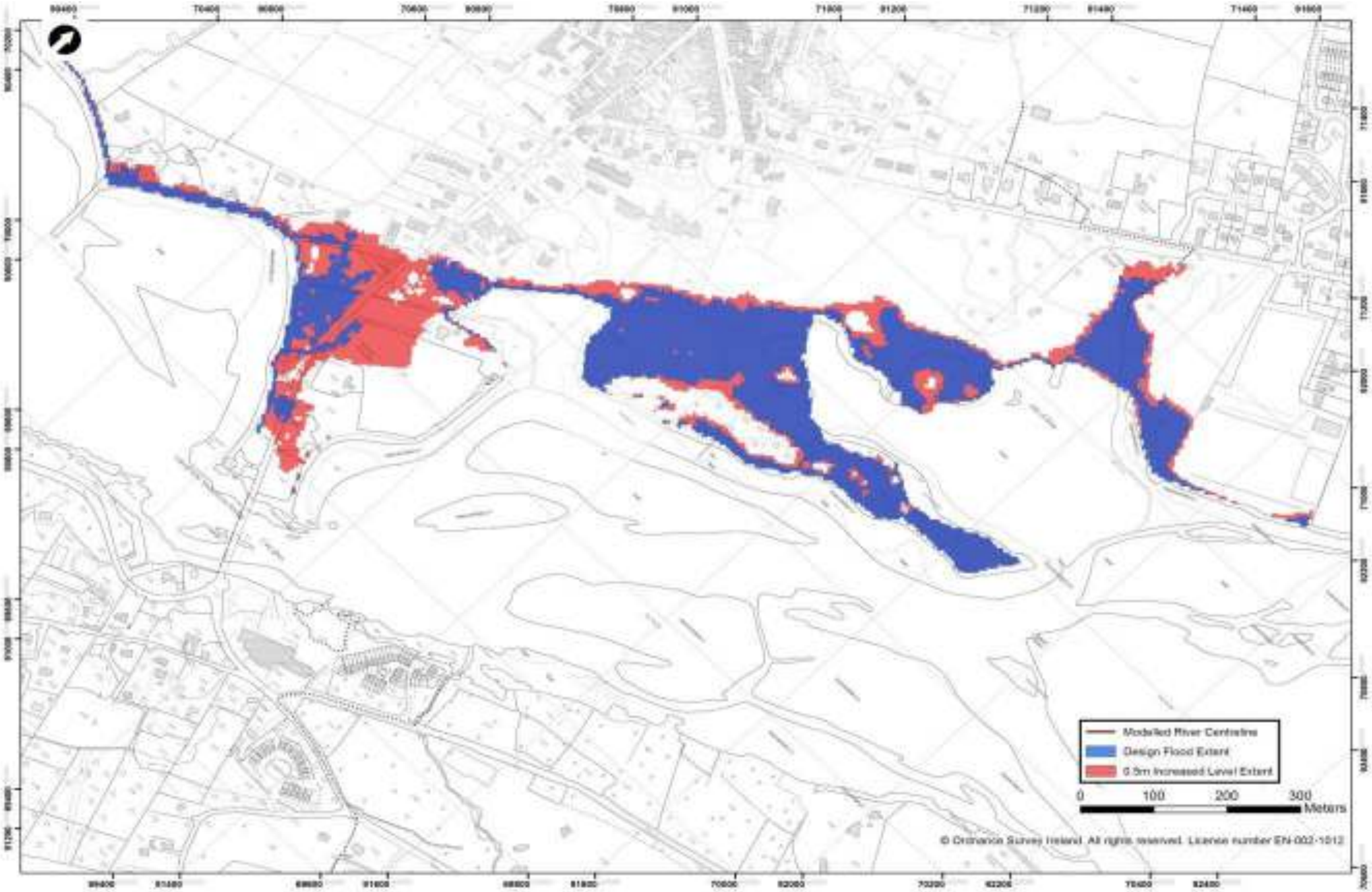
Map 5.5: Sensitivity to Downstream Level – Bantry Town Model



Map 5.6: Sensitivity to Downstream Level - Castletown Bearhaven Model



Map 5.7: Sensitivity to Downstream Level - Kenmare 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 in Kenmare	0.050	0.055
Active River Channel in Durrus and Bantry	0.040	0.045
River Banks/ Medium to Dense Vegetation	0.080	0.100
Buildings	0.200	0.250
Roads and Other Hard Standing	0.030	0.035
Rural/Pasture	0.060	0.080

In UoM21, flood level and extent were not sensitive to the Manning's 'n' values assigned to any of the models. The greatest increase in flood risk attributed to Manning's 'n was predicted in Durrus (Map 5.8) upstream of School Road. However, the typical increase in water level was less than 0.2m and did not increase flooding to any properties, roads or environmentally-protected features.

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

5.2.4 Pipe Obstruction at Finnihy Bridge

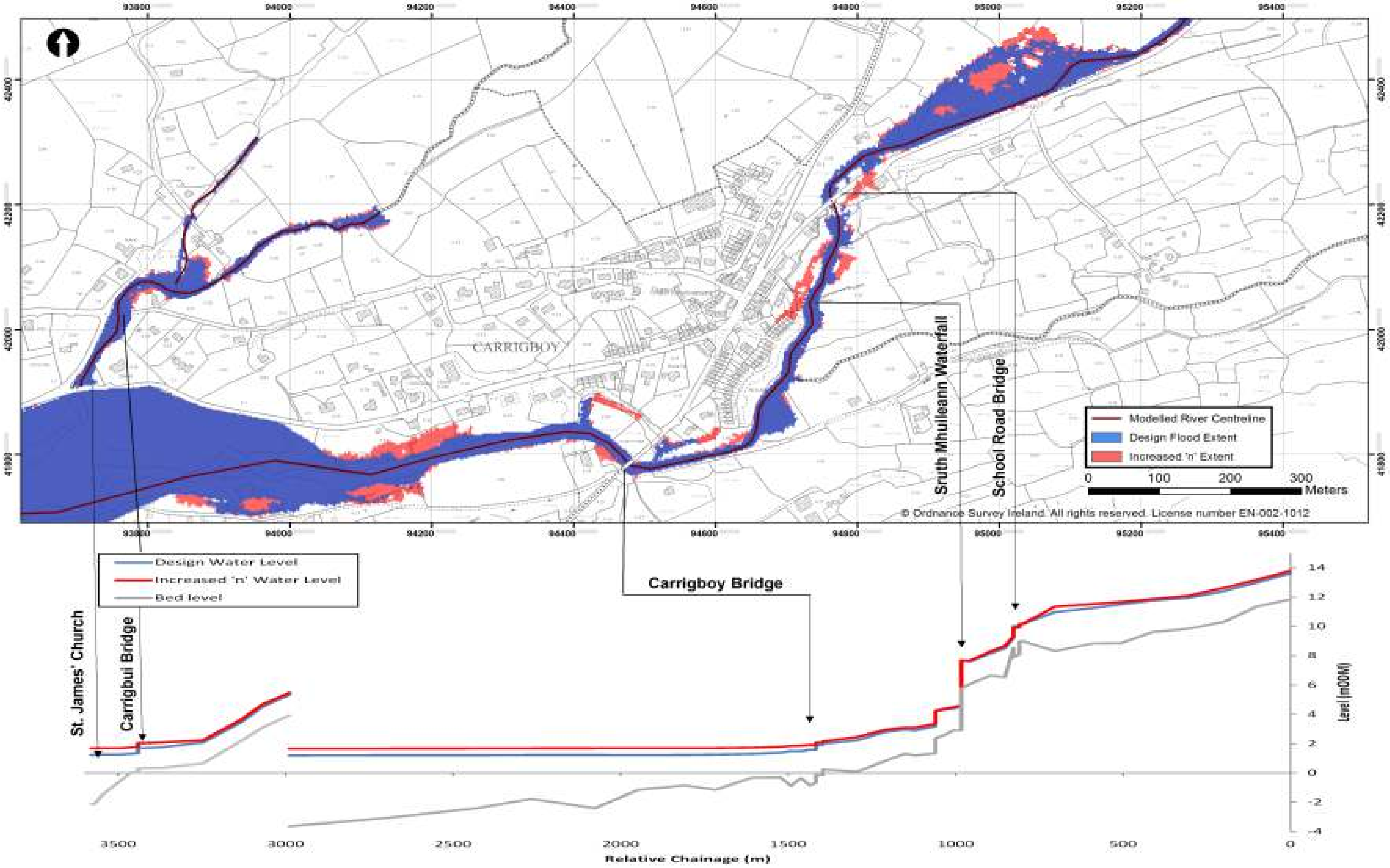
The Kenmare design model assumes a worst case scenario at Finnihy Bridge where the opening above the utility pipe on the upstream face becomes blocked and therefore effectively lowers the soffit and capacity of the bridge. This is a worst case scenario to provide a conservative estimate of flood risk to the town. Therefore, a sensitivity test was undertaken with the upstream utility pipe entirely removed to establish the impact on flood risk. Map 5.9 compares the 1% AEP fluvial current event with pipe (design) and the 1%AEP fluvial current event with the pipe entirely removed.

The removal of the pipe on the upstream face of Finnihy Bridge decreases flood levels upstream on the Finnihy to the Convent and upstream on the Lissaniska to Scarteen Park. This reduction in backwater significantly reduces the flood risk to Rose Cottages, Market Street and Bridge Street in the centre of Kenmare.

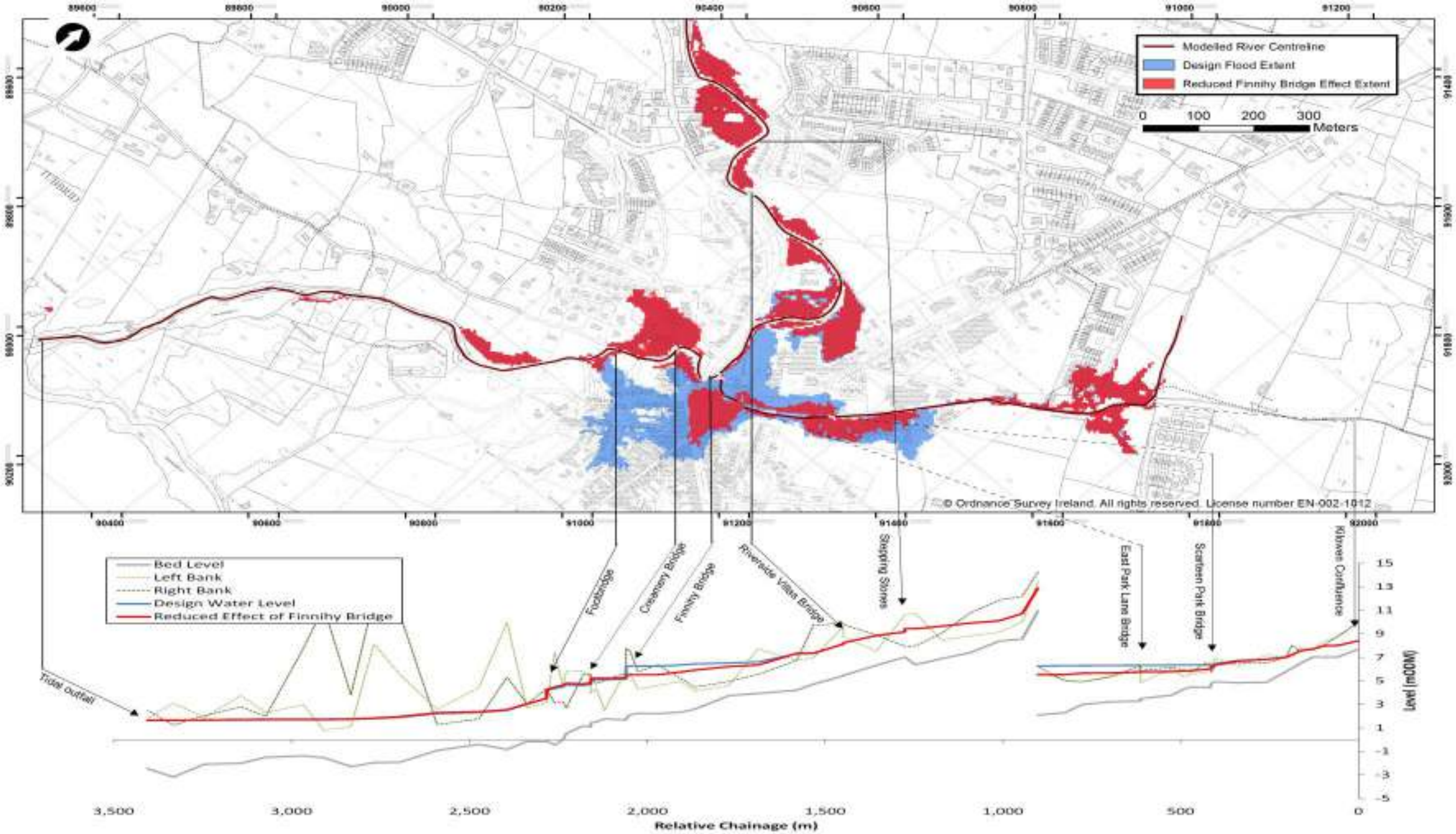
Conversely, the flow through the bridge increases from 58 to 69 m^3/s (+20%) and flood levels increase by 0.12m downstream of Finnihy Bridge to the footbridge. However, the increase in conveyance in the downstream reach does not significantly increase flood extent as the floodplain at the Creamery is already inundated.

In reality, the amount of head loss due to the obstruction caused by the pipe and any debris that gets caught against it will be somewhere between the two scenarios tested. Therefore, the effective capacity of Finnihy Bridge should be carefully considered when interpreting flood maps, deriving flood risk management options and assessing any future flood events.

Map 5.8: Sensitivity to Manning's 'n' – Durrus Model



Map 5.9: Sensitivity to Finnihy Bridge Head Loss Assumptions – Kenmare Model



5.2.5 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 flooded area (>5%) and increase in water level (±0.2m). In some cases there is a significant increase in level but this does not result in a significant increase in flood risk and extent, such as the Mealagh catchment.

Table 5.4: Summary of Sensitivity Run Performance

Model	Flow		Level		Manning's 'n'		Finnihy Bridge	
	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?
Durrus	0.09	No	N/A		0.16	No	N/A	
Bantry – Dromcarra	0.14	No	0.23	No	0.11	No	N/A	
Bantry-Mealagh	0.34	No	0.14	No	0.10	No	N/A	
Bantry - Bantry	0.65	Yes	0.45	Yes	0.08	No	N/A	
Castletown Bearhaven	N/A		0.55*	Yes	0.01	No	N/A	
Kenmare	0.32	Yes	0.55*	Yes	0.13	No	-0.33	Yes

RMSD is Root Mean Square Difference.
*RMSD for open coast is the absolute increase in water level i.e. 0.55m.

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

- Bantry Town catchment and Kenmare AFAs are sensitive to assumptions and uncertainties in peak flow. The uncertainty and sensitivity to peak flow and duration estimates should be considered in the sizing and operation of any flood management options using storage of flood waters.
- Bantry Town catchment, Castletown Bearhaven and Kenmare AFAs are sensitive to the assumptions and uncertainties in downstream water level. 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 UoM21. However, the reduction in roughness of the channel through maintenance activities may improve channel capacity and/or conveyance for events which are closer to the threshold of flooding.
- Kenmare AFA is sensitive to the assumptions taken for the blockage of the pipe at Finnihy Bridge. The effective capacity of Finnihy Bridge should be carefully considered when interpreting flood maps, deriving flood risk management options and assessing any future flood events.

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 UoM21 and design event runs simulated.

Both the fluvial and coastal scenarios have been simulated for Bantry and Kenmare as these AFAs have 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 Durrus 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. The hydraulic model of Durrus has been extended down to the open sea and a tidal boundary applied directly to the model. However the tidal influence does not affect the AFA as the channel in Durrus is significantly above the extreme sea levels.

No fluvial scenarios have been simulated for Castletown Bearhaven as the AFA was not identified as being at fluvial flood risk. No fluvial inflows have been applied to the Castletown Bearhaven hydraulic model.

Only the current 10%AEP and 5%AEP wave overtopping scenarios were simulated for Kenmare as the more frequent event resulted in less than 1m^3/s overtopping the vulnerable sections for less than 1 hour. For the less frequent events, the total tide plus surge level overtopped the crest levels which are already simulated under the coastal design event runs. Similarly, the wave overtopping volumes in Castletown Bearhaven were negligible under current conditions and only became significant in the 0.5%AEP and 0.1%AEP events under the Mid range future scenario. The wave overtopping was negligible in comparison to the mechanism 1 flooding (overtopped by the total tide plus surge) under the High End future scenario and therefore was not simulated.

Table 6.1: Design Event Runs

Source	Scenario	%AEP	Run Name	Durrus Model (I29DS)	Bantry Model (I30BY)	Castletown BearHaven (I31CN)	Kenmare Model (I32KE)
Fluvial	Current	50%	FCD500_D1	✓	✓	N/A	✓
		20%	FCD200_D1	✓	✓	N/A	✓
		10%	FCD100_D1	✓	✓	N/A	✓
		5%	FCD050_D1	✓	✓	N/A	✓
		2%	FCD020_D1	✓	✓	N/A	✓
		1%	FCD010_D1	✓	✓	N/A	✓
		0.50%	FCD005_D1	✓	✓	N/A	✓
		0.10%	FCD001_D1	✓	✓	N/A	✓
	MRFS	50%	FMD500_D1	✓	✓	N/A	✓
		20%	FMD200_D1	✓	✓	N/A	✓
		10%	FMD100_D1	✓	✓	N/A	✓
		5%	FMD050_D1	✓	✓	N/A	✓
		2%	FMD020_D1	✓	✓	N/A	✓
		1%	FMD010_D1	✓	✓	N/A	✓
		0.50%	FMD005_D1	✓	✓	N/A	✓
		0.10%	FMD001_D1	✓	✓	N/A	✓
	HEFS	10%	FHD100_D1	✓	✓	N/A	✓
		1%	FHD010_D1	✓	✓	N/A	✓
		0.10%	FHD001_D1	✓	✓	N/A	✓
Coastal	Current	50%	CCD500_D1	N/A	✓	✓	✓
		20%	CCD200_D1	N/A	✓	✓	✓
		10%	CCD100_D1	N/A	✓	✓	✓
		5%	CCD050_D1	N/A	✓	✓	✓
		2%	CCD020_D1	N/A	✓	✓	✓
		1%	CCD010_D1	N/A	✓	✓	✓
		0.50%	CCD005_D1	N/A	✓	✓	✓
		0.10%	CCD001_D1	N/A	✓	✓	✓

Source	Scenario	%AEP	Run Name	Durrus Model (I29DS)	Bantry Model (I30BY)	Castletown BearHaven (I31CN)	Kenmare Model (I32KE)
	MRFS	50%	CMD500_D1	N/A	✓	✓	✓
		20%	CMD200_D1	N/A	✓	✓	✓
		10%	CMD100_D1	N/A	✓	✓	✓
		5%	CMD050_D1	N/A	✓	✓	✓
		2%	CMD020_D1	N/A	✓	✓	✓
		1%	CMD010_D1	N/A	✓	✓	✓
		0.50%	CMD005_D1	N/A	✓	✓	✓
		0.10%	CMD001_D1	N/A	✓	✓	✓
	HEFS	10%	CHD100_D1	N/A	✓	✓	✓
		0.50%	CHD005_D1	N/A	✓	✓	✓
		0.10%	CHD001_D1	N/A	✓	✓	✓
Wave Over-Topping	Current	10%	WCD100_D1	N/A	N/A	N/A	✓
	Current	5%	WCD050_D1	N/A	N/A	N/A	✓
	MRFS	0.50%	WMD005_D1	N/A	N/A	✓	N/A
	MRFS	0.10%	WMD001_D1	N/A	N/A	✓	N/A
TOTAL Model Runs				19	38	21	40

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;

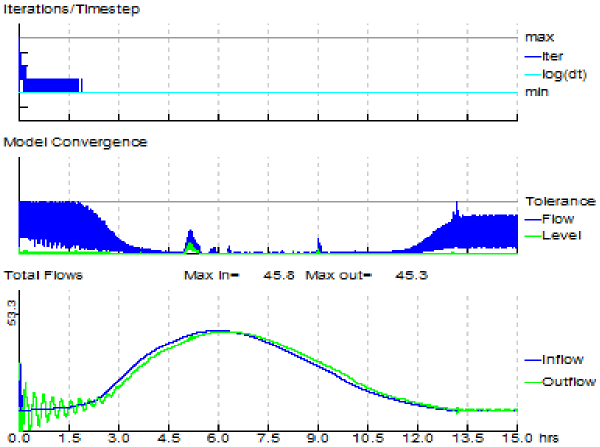
- The number of iterations per timestep taken to resolve flow and level in the model;
- The convergence of flow and water level in the model within the recommended tolerance of +/- 0.01 m or 0.01 m^3/s between consecutive timesteps;
- The total inflow and outflow from the model components.

The 1D ISIS models were convergent within the recommended tolerances for the majority of the design event in Durrus, Bantry and Kenmare. There is no 1D convergence plot for Castletown Bearhaven as there are no 1D components for this model. The following observations can be made:

- The oscillation between 0 and 3 hours in Durrus is caused by the stabilisation of flow in the estuary. However this only converts to a less than 0.01m change in water level and does not affect the results during the flood event.
- The outflow is larger than the inflow in the Bantry model as the outflow includes both the ISIS inflows and inflows from the ESTRY reaches.
- The flow hydrograph is attenuated by 1.5 hours in the Mealagh catchment in Bantry due to the attenuation of flood waters on the floodplain.
- The iterations increase in the Kenmare model as the water comes out of bank at 15 hours due to the resolution of backwater from Finnihy Bridge and the wetting of cells in the 2D domain. However it does not affect the peak.
- The flow hydrograph is attenuated by 1.25 hours in Kenmare due to attenuation of flood waters on the floodplain once the Finnihy spills out-of-bank.

The cumulative mass balance for the 2D model components is shown in Figures 6.4 to 6.7. All the design models were convergent and within the recommended tolerance of ±1% mass error at the peak flow and/or tide plus surge level. There is an initial increase in cumulative mass error for the start of the Castletown Bearhaven model caused by the wetting of the cells at Brandyhall Bridge. However, the mass error rapidly decreases to less than 0.1% within an hour and does not affect the model results at the peak tide.

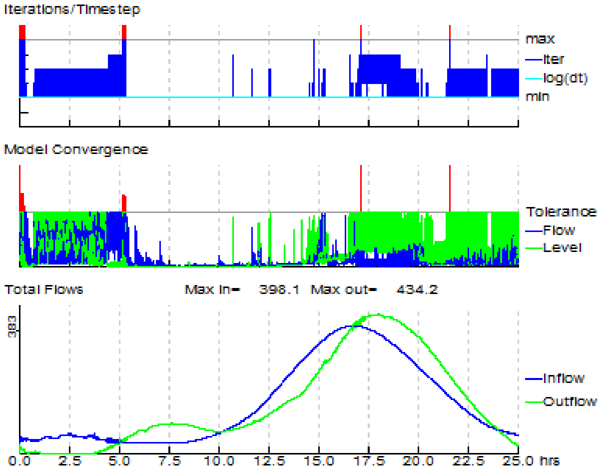
Figure 6.1: 1D Convergence Plot - Durrus



Datafile: ...\MODEL\ISIS\DAT\29DS_ISIS_001_296_01.DAT
Results: ...\RESULTS\29DS_FCD010_D1_DURRUS_296_01.zzl
Ran at 19:43:47 on 03/10/2013
Ended at 20:00:57 on 03/10/2013
Start Time: 0.000 hrs
End Time: 15.000 hrs
Timestep: 1.0 secs

Current Model Time: 15.00 hrs
Percent Complete: 100 %

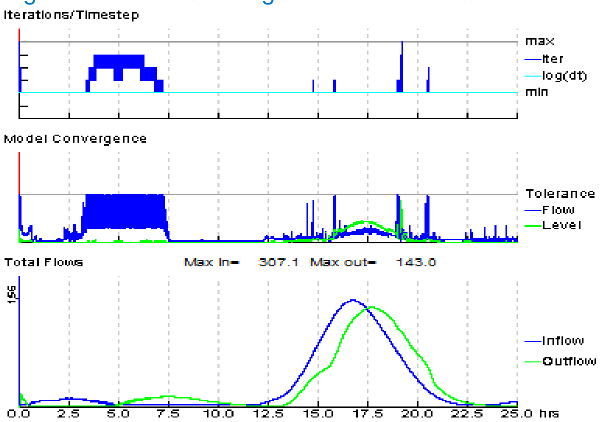
Figure 6.2: 1D Convergence Plot - Bantry



Datafile: ...\MODEL\ISIS\DAT\30BY_ISIS_BASE_001_409.DAT
Results: ...\RESULTS\30BY_FHD001_D2_BANTRY_409.zzl
Ran at 16:56:25 on 19/05/2014
Ended at 19:50:09 on 19/05/2014
Start Time: 0.000 hrs
End Time: 25.000 hrs
Timestep: 1.0 secs

Current Model Time: 25.00 hrs
Percent Complete: 100 %

Figure 6.3: 1D Convergence Plot - Kenmare



Datafile: ...\32KE_KENMARE\DESIGN\MODEL\32KE_D2.DAT
Results: ...\32KE\DESIGN\RESULTS\1D\32KE_FHD001_D2.zzl
Ran at 09:52:05 on 12/05/2014
Ended at 10:30:45 on 12/05/2014
Start Time: 0.000 hrs
End Time: 25.000 hrs
Timestep: 1.0 secs

Current Model Time: 25.00 hrs
Percent Complete: 100 %

Figure 6.4: 2D Mass Balance Plot - Durrus

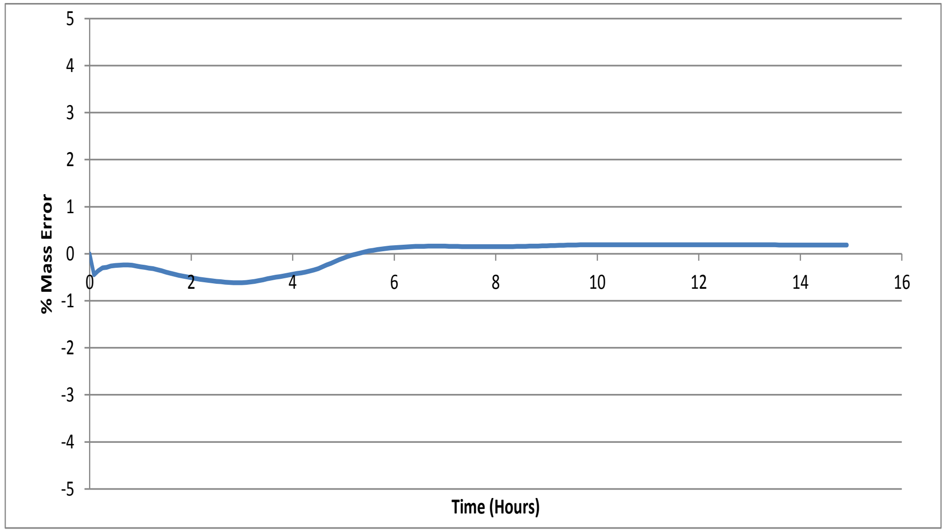


Figure 6.5: 2D Mass Balance Plot - Bantry

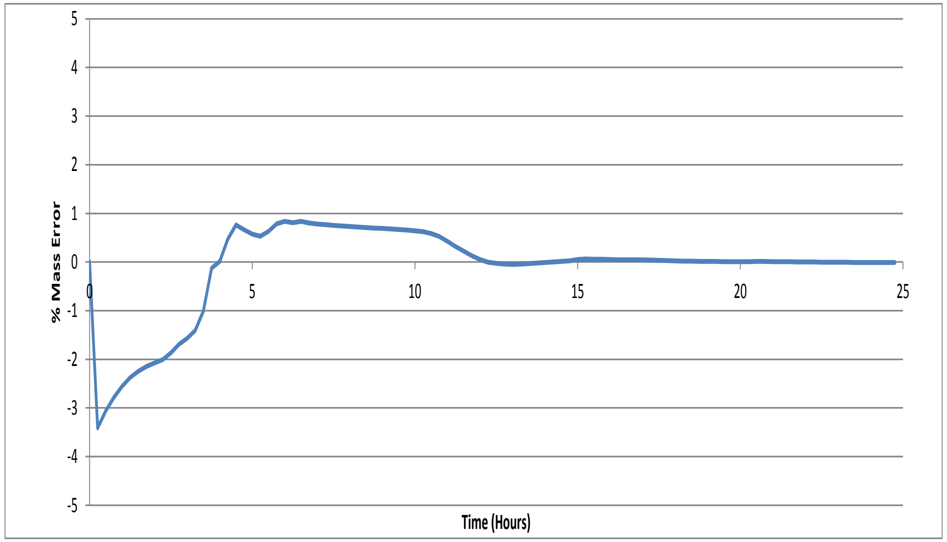


Figure 6.6: 2D Mass Balance Plot – Castletown Bearhaven

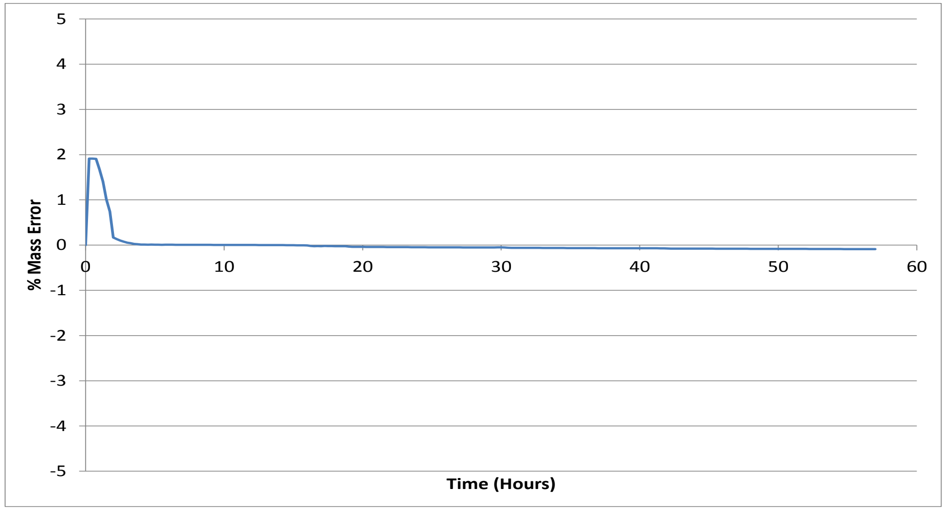
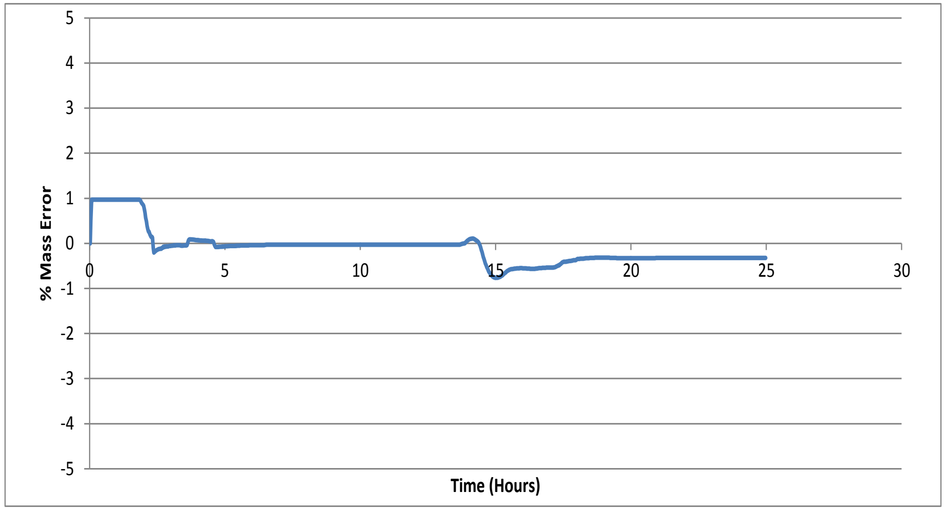


Figure 6.7: 2D Mass Balance Plot - Kenmare



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. In UoM21, the inflow hydrographs were shifted uniformly within each hydrological catchment to ensure a physically realistic single storm event in these small coastal catchments (< 30 km^2) (discussed in Table 3.4).

Tables 6.2 compares the model predicted flows with the design peak flows at the target HEPs for the 10%AEP, 1%AEP and 0.1%AEP fluvial events. 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 modelled flows are within 9% of the design flows for the majority of HEPs not affected by backwater. Greater discrepancies between modelled and design flows were found at the tidal outfalls due to backwater effects that limit fluvial discharge. The larger discrepancies can be explained as follows:

- The modelled peak flows at the upstream end of the Bantry are higher than the design peak flow because of the cross-catchment flow over the watershed from the Ardnageehy tributary.
- The fluvial discharge is limited by the tide at the outfall of each coastal catchment but tide-locking is not considered in the design hydrology which assumes free-flow conditions.

Table 6.2: Summary of Hydrological Routing Performance for the 1%AEP Fluvial Current Event

HEP ID	Location	Model Node	10%AEP Design Target Flow (m^3/s)	10%AEP Model Predicted Flow (m^3/s)	10%AEP Difference (%)	1%AEP Design Target Flow (m^3/s)	1%AEP Model Predicted Flow (m^3/s)	1%AEP Difference (%)	0.1%AEP Design Target Flow (m^3/s)	0.1%AEP Model Predicted Flow (m^3/s)	0.1%AEP Difference (%)
Durrus											
21_6225_1	Ahanegavanagh d/s Clashadoo	21NAGH00046H	8.6	8.8	2%	12.7	13.3	5%	18.8	18.9	0%
21_6225_2	Ahanegavanagh d/s extent	21NAGH00018H	8.7	8.9	2%	12.9	13.5	5%	19.1	20.2	6%
21_7736_5	Four Mile Water North at Waterfall	21FMWN00211W	20.5	20.2	-2%	30.7	30.1	-2%	46	45.0	-2%
21_8044_2	Four Mile Water North d/s (fluvial)	21FMWN00147H	24.2	22.0	-9%	36.3	32.9	-9%	54.3	49.1	-10%
Bantry											
21_7060_2	Bantry d/s Knocknavaghaea	21BANT00202H	3.4	3.6	6%	4.7	5.7	21%	6	8.4	39%
21_7249_2	Bantry d/s Sheskin	21BANT00141H	5.1	5.3	3%	7.1	8.1	14%	9.1	10.1	11%
21_7092_1	Bantry d/s Carrignagat	21BANT00118H	5.8	6.4	11%	8.1	8.7	7%	10.4	12.9	24%
21_7096_1	Bantry d/s Dromleigh	21BANT00091I	6.9	7.2	4%	9.6	9.7	1%	12.3	14.0	14%
21_7225_2	Bantry d/s	21BANT0045J	9.4	8.4	-11%	13.2	11.4	-14%	16.9	16.2	-4%
21_7668_2	Dromacoosane d/s (fluvial)	DROM00063D	3.8	4.0	5%	4.9	4.8	-2%	7.3	7.1	-3%
21_6183_1	Mealagh d/s Raheen Beg	21MEAL00195H	124.3	117.7	-5%	181.3	173.2	-4%	266.3	252.9	-5%
21_6412_1	Mealagh d/s Derryginagh	21MEAL00175H	131.6	138.0	5%	191.9	198.2	3%	282.0	287.4	2%
21_6258_3	Mealagh d/s (fluvial)	21MEAL00094H	133.3	128.0	-4%	194.5	202.6	4%	285.7	279.9	-2%
Castletown Bearhaven not assessed as there are no fluvial inflows											
Kenmare											
21_2408_1	Finnihy downstream of Gortamullen	21FINN00253H	44.5	42.5	-4%	66.3	64.3	-3%	98.8	93.6	-5%
21_2495_1	Finnihy downstream of Lissaniska	21FINN00137B	50.8	49.0	-4%	75.7	75.6	0%	112.8	113.7	1%
21_2495_4	Finnihy downstream (tidal outfall)	21FINN00001H	52.5	53.1	1%	78.1	73.7	-6%	116.4	109.9	-6%
21_6311_1	Lissaniska downstream of Kilowen	21LISS00098H	3.4	3.7	9%	5.1	5.3	4%	7.5	7.9	6%
21_6311_3	Lissaniska downstream	21LISS00003A	4.3	4.4	2%	6.4	6.2	-4%	9.5	7.8	-17%
Green denotes HEPs affected by predicted cross-watershed flow.											
Yellow denotes HEPs affected by backwater.											

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.
- Utility pipes that cross immediately upstream of or under bridges were assumed to form the soffit as a worst-case scenario for the capacity of the structure.
- In Bantry, the culverts on the Ardnageehy are assumed to outfall on the far side of Caherdaniel Road and do not enter the urban drainage network. A number of site investigations by both the surveyors and Mott MacDonald could not find the outfall location. The outlet dimensions are assumed to be the same as the inlet dimensions.
- In Bantry, the dimensions of the inlet to the downstream culvert on the Knocknavaghaea West Stream have been assumed based on the adjoining culvert from the eastern tributary combined with DTM levels as no access was gained during the river channel survey.

7.2 Limitations

There are a number of uncertainties associated with the flow estimation and hydraulic modelling methodology used in UoM21. They include:

- There is uncertainty in the derivation of design flows for small catchments in Bantry, Durrus and the upper catchment of Kenmare. 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 flows between the surface water channel and urban drainage network on the Ardnageehy and Knocknavaghaea West Stream once the flows enter the culverts.
- The absence of river flow or continuous water level data in Durrus, Kenmare, Bantry Dromcarra and Bantry Town catchments 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 UoM21 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 would be used to derive the flood depth and flood extents. However UoM21 does not include any MPW reaches and the 1D mapping process is not discussed further. 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 (in the bullet points 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.

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. This has been modified from the DEFRA FD2320 guidance:

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

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. 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 9.1) 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. The flood extents were reviewed to remove significant areas of disconnected flooding from initial water levels were removed. However, the 2D model simulates all active flow paths so wet cells are generally 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.

Flood Zone A and B have been derived from the outer extent envelope of the two undefended extents (i.e. 1%AEP (0.5%AEP for coastal) and 0.1%AEP). There are no formal or informal effective flood defences in UoM21. Therefore, the flood zone outlines are the same as the flood extents.

Combined Flood Source Mapping

Bantry and Kenmare are 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.

In the case of Kenmare and Castletown Bearhaven, the wave overtopping extents were kept separate from the tidal dominant scenario as agreed with the OPW.

8.5 Flood Risk (Assessment) Mapping

8.5.1 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.

8.5.2 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

8.5.2.1 Indicative Number of Inhabitants

For each AFA, the study area was broken into a number of grids, each 100m^2. 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. The average occupancy of residential properties varied between 2.6 and 2.8 across the South West Region based on the 2011 census data. No allowance was made for commercial properties.

8.5.2.2 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.

8.5.2.3 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 methodology to determine the unit cost of damage for different property types is to be confirmed at a later date.

Following the calculation of the estimated cost of damages for the full range of AEP events, the Annual Average Damage (AAD) for each property will be calculated. The AAD for each property within each a $100m^2$ grid was summed and represented on a map providing the economic risk density (€ AAD / $100m^2$).

9 Model and Mapping Results

9.1 Overview

Based on the model predicted results and flood maps, the greatest fluvial flood risk in UoM21 is located in Bantry and Kenmare. For the target 1%AEP event, over 100 properties were affected along the Bantry Stream and in Kenmare flooding over 150 properties along the Finnihy and Lissaniska Rivers in Kenmare. In both cases, undersized bridges and culvert structures lead to flooding upstream. The fluvial flood risk is further exacerbated in Bantry due to tidelocking of the downstream culverts.

The model predicts regular flooding of riverside areas in Durrus in the 10%AEP event. However, fluvial flood risk in Durrus was predicted to be lower than in the other UoM21 AFAs with no properties flooded in the 1%AEP event and only the gardens of approximately 10 properties affected in the 0.1%AEP current fluvial event.

The greatest coastal flood risk is predicted at Wolfetone Square, Bantry for the 10% AEP event and larger events due to gaps in the sea wall along the quayside. Coastal risk in Castletown Bearhaven is limited to less than 20 properties near the Garda Station and along Main Street from the 0.5%AEP current.

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 will be discussed in the subsequent Flood Risk Assessment.

9.2 Durrus AFA

Map 9.1 summarises the fluvial flood risk in Durrus 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 river banks upstream of School Road.
- Overtopping of School Road, bypassing the bridge in the most extreme fluvial events.
- High flows along the loop channel on the right bank of Four Mile Water North that provides a low point through which flood water passes before flowing around the houses at Sruth Mhuilean.

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

- 50%AEP fluvial event overtops the low lying areas upstream of School Road.
- 10% AEP fluvial event overtops the river banks to flood low lying areas at the Sruth Mhuilean estate.
- 0.5%AEP fluvial event overtops the right bank at Sruth Mhuilean waterfall.
- 0.1%AEP fluvial event overtops the river banks at the Sruth Mhuilean estate to flood properties but flooding is shallow.
- Less than ten buildings are affected by the 0.1%AEP fluvial event, located around the Sruth Mhuilean estate.

The risk to life at the Sruth Mhuilean estate is low because the depth of flooding is shallow and velocities are low. However, flood hazard is significant to extreme for the riverside fields upstream of School Road because the depth of flooding is greater and velocities are significant.

The critical structures in determining fluvial flood risk include:

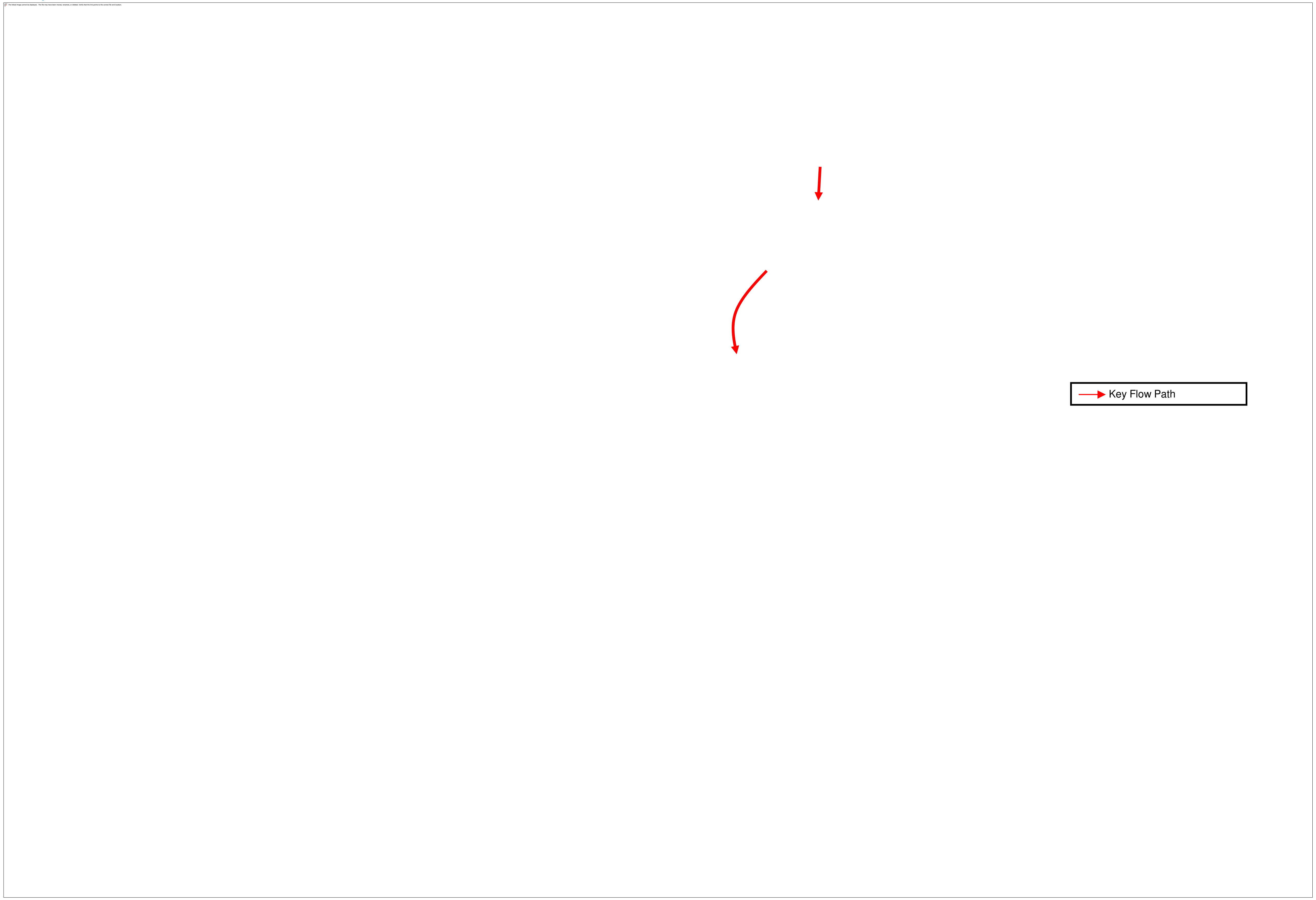
- The Sruth Mhuilean waterfall.
- The loop channel on the right bank of Four Mile Water North.

The areas flooded are consistent with the limited information available from the local authority staff during the flood risk review and the topography of the AFA. There is uncertainty in the flow estimates for this ungauged catchment. However, the flood levels and extents were not found to be sensitive to the inflows applied. Therefore there is reasonable confidence in the flood mapping in Durrus based on the information available at the time of this study.

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

- Flood storage upstream of School Road is feasible but the benefit relative to the risk should be considered.
- Localised property protection on the Sruth Mhuilean is likely to reduce flood risk.
- Flood warning is likely to be effective given the > 6 hours' time to peak for the Four Mile Water catchment.

Map 9.1: Summary of Fluvial Flood Risk - Durrus



9.3 Bantry AFA

Maps 9.2 to 9.4 summarise the fluvial flood risk in Bantry Dromacoosance, Mealagh and Bantry Town catchments respectively for the 10%, 1% and 0.1%AEP design scenarios. Map 9.5 summarises the coastal flood risk for the 10%, 0.5% and 0.1%AEP design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Backwater from the West Lodge Park weirs causes water to spill out of bank upstream and flood down the valley.
- High river flows exceed the capacity of the River Mealagh to flood areas between Dromnafishin and Lahadane.
- High river levels flood the Lahadane Business Estate from the low point in the river bank at the eastern end of the estate.
- Tidal levels inundate areas adjacent to a tidal creek near Dunnamark House at the tidal outfall of the Mealagh based on LIDAR information available.
- High flows along the Mileencoola East tributary rapidly exceed the capacity of the culverts under the Raheen Beg Road to flood the north side of the road.
- High flows exceed the culvert downstream of the Millwheel on Bantry Stream and the downstream channel of the Reenrour tributary causing flooding along Bridge Street and High Street.
- High flow exceeds the capacity of the downstream culvert on the Knocknavaghaea and Ardnageehy tributaries in the most extreme events, causing shallow but fast flowing water across the main road.

The key thresholds and areas affects by flooding in Bantry are:

- 50%AEP floods gardens of properties along Glengarriff Road.
- 1%AEP fluvial event causes flooding at Knocknavaghaea tributary but is shallow and affects < 3 properties .
- 1%AEP fluvial event causes flooding in central Bantry due to overtopping upstream of the Millwheel on Bantry Stream and overtopping of the downstream culvert on Reenrour.
- 0.1%AEP fluvial event causes flooding at Heatherfields due to the capacity of the downstream culverts.
- 5-10%AEP fluvial event overtops the right bank at the low point in the Lahadane business park embankment but does not affect properties. This matches well with the estimated frequency of flooding provided by the local authority staff during the flood risk review.
- 2%AEP fluvial event on the Mealagh results in extensive flooding of the Lahadane business park.
- 10%AEP coastal event overtops the gaps in the sea wall and floods Wolfetone Square.

The greatest risk to life is associated with deep flooding at Lahadane Business Park on the Mealagh. However, there is also significant risk to life along Bridge Street, High Street and across the Caherdaniel Road in the 1%AEP and larger magnitude events. In comparison, extreme flood hazard from coastal flooding occurs in the 10%AEP and larger events across Wolfetone Square.

The critical structures in determining flood risk include:

- Culvert downstream of the Millwheel on Bantry Stream which adjoins the Reenrour tributary.
- Downstream culvert on the Knocknavaghaea and Ardnageehy Streams for extreme events.

- Culverts on the Mileencoola East Stream.
- The quayside sea wall at Wolfetone Square.

The key areas of uncertainty in Bantry are:

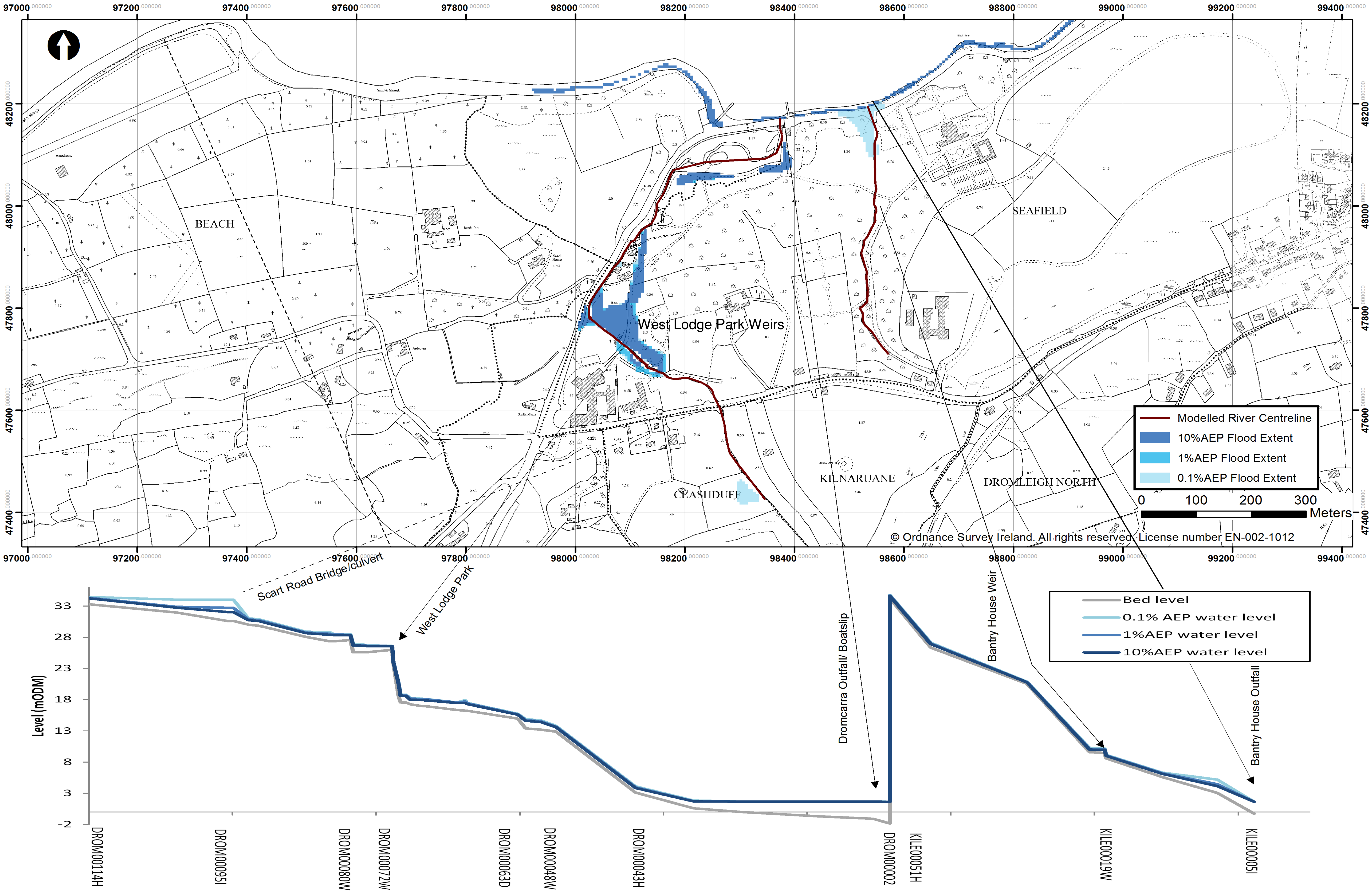
- Flow paths across the watershed between Bantry and Ardnageehy catchments because the flat bog area has multiple flow paths which cross the catchment boundary.

The uncertainty in these areas should be carefully considered when interpreting the flood maps.

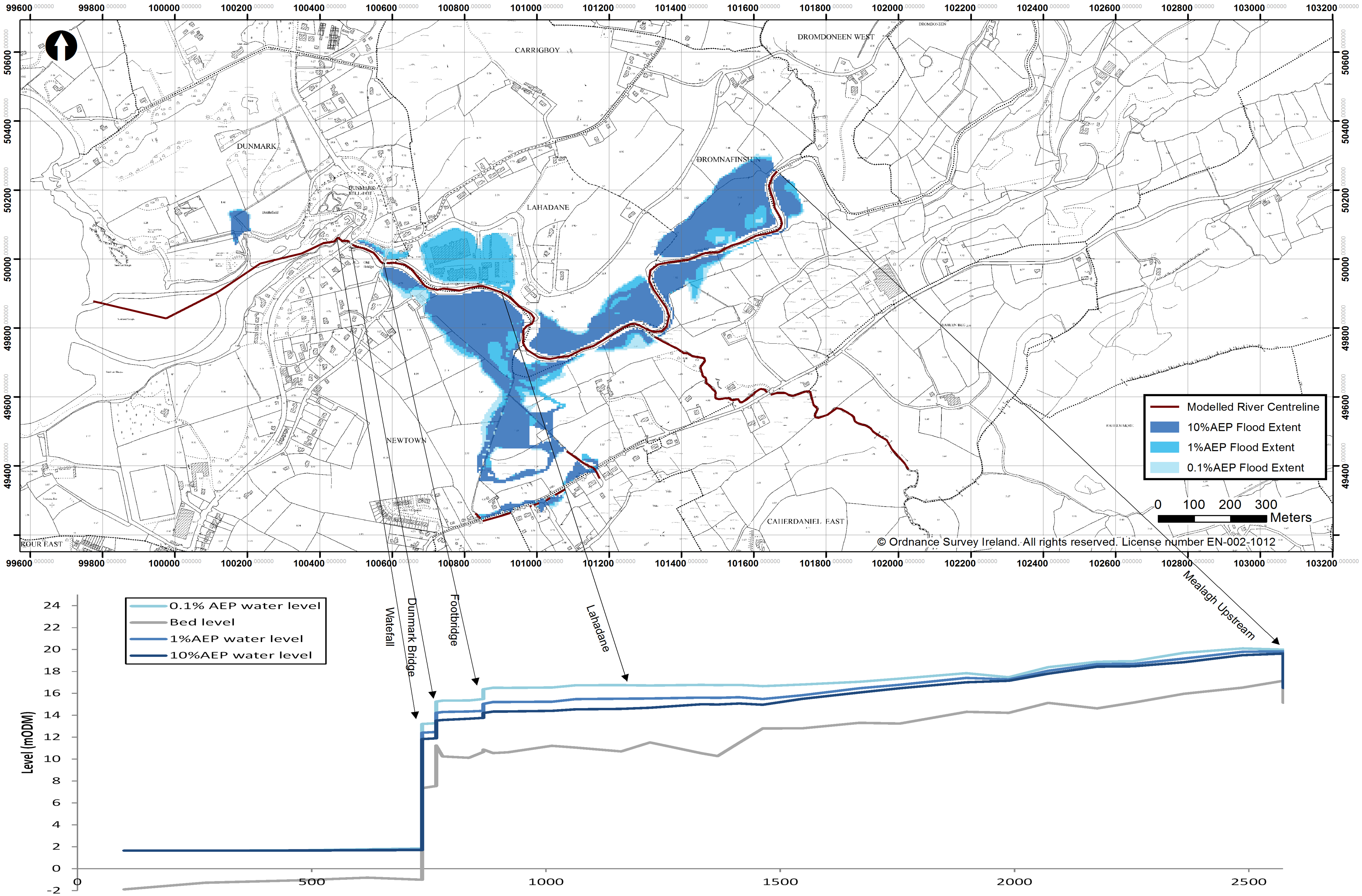
The following recommendations for flood risk management option development can be made:

- Increased conveyance at critical structures in the fluvial reaches of Bantry Stream is likely to reduce flood risk.
- Increased conveyance of the downstream culverts along the Bantry Stream is unlikely to reduce flood risk during tide-locked periods without additional pumping.
- Raising of riverside embankments at Lahadane and filling the gaps in the sea wall at Wolfetone Square is likely to reduce flood risk to the centre of Kenmare.
- Flood warning for fluvial events on the Bantry and Dromacoosane catchments is unlikely to be effective for these small steep catchments given the short time to peak.
- Flood warning on the Mealagh catchment is likely to be more effective as there would be several hours before the peak flow at the Inchiclogh Gauge, which is a good indicator of flooding downstream at Lahadane.

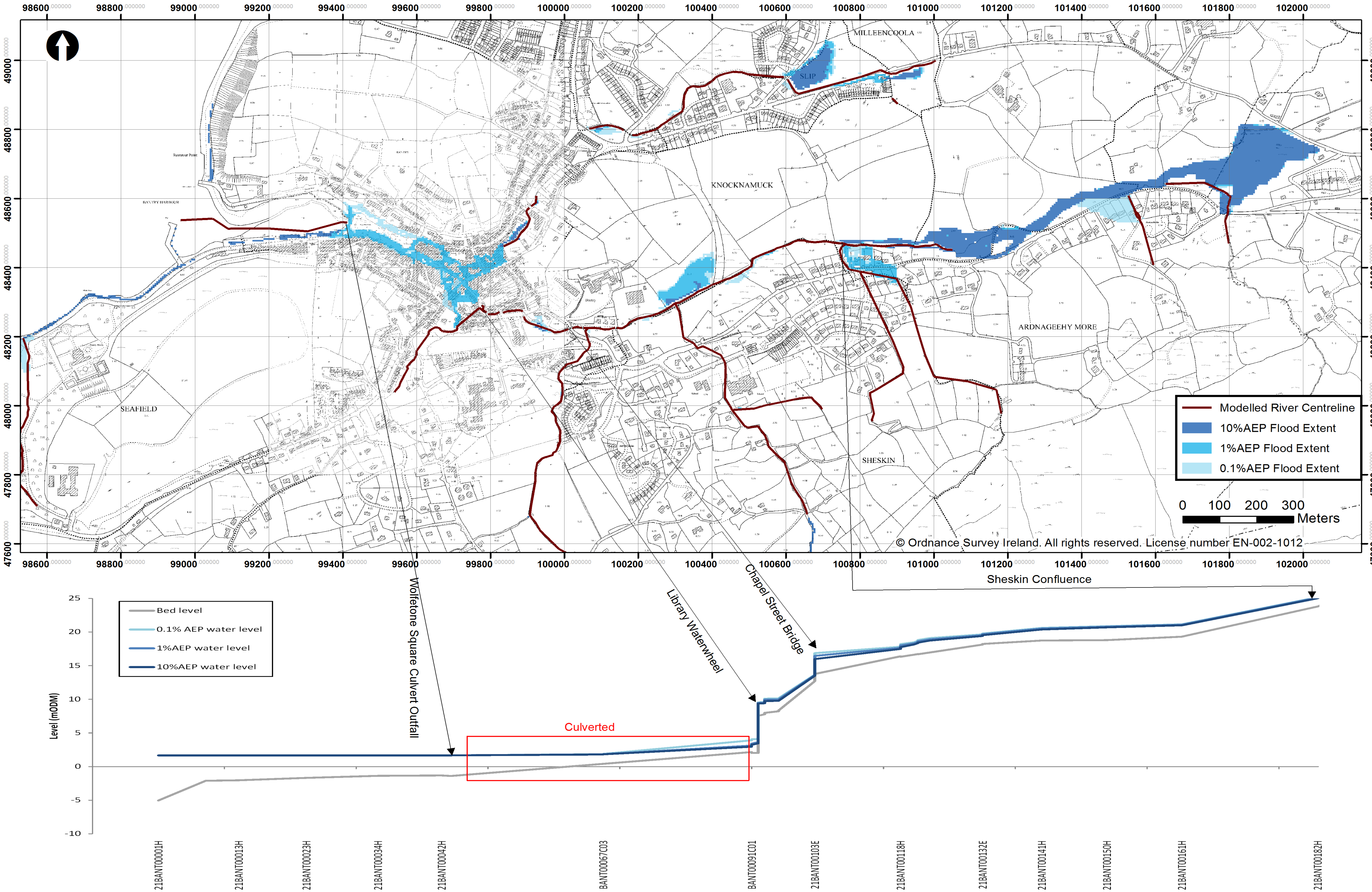
Map 9.2: Summary of Fluvial Flood Risk – Dromacoosane Catchment, Bantry



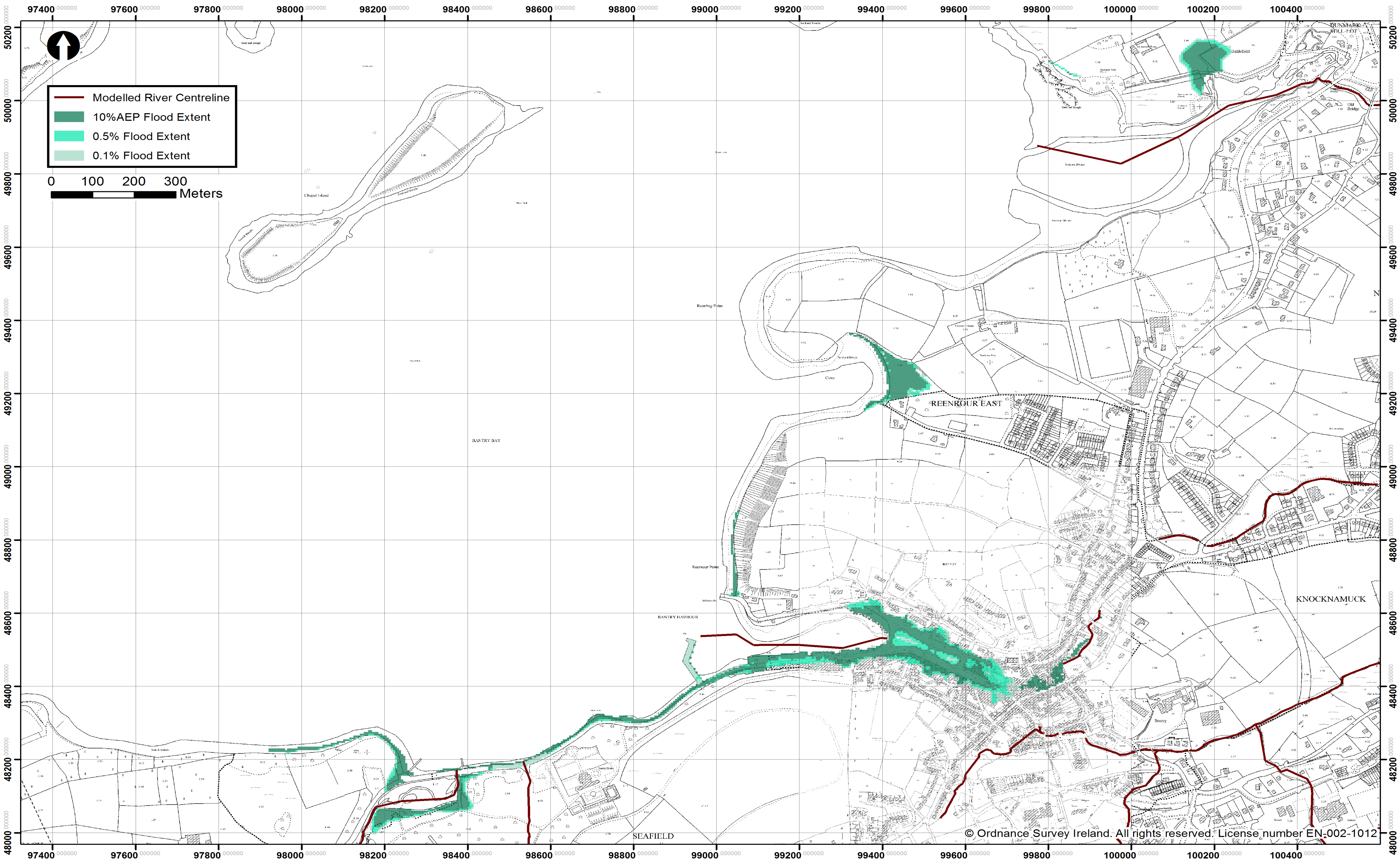
Map 9.3: Summary of Fluvial Flood Risk – Mealagh Catchment, Bantry



Map 9.4: Summary of Fluvial Flood Risk – Bantry Town Catchment, Bantry



Map 9.5: Summary of Coastal Flood Risk – Bantry



9.4 Castletown Bearhaven AFA

Map 9.6 summarises the coastal flood risk in Castletown Bearhaven for the 10%, 0.5% and 0.1%AEP design scenarios. Coastal flood risk is constrained to the areas seaward of Main Street. The key flow routes flooding mechanisms predicted by the model are as follows:

- Overtopping behind the Garda Station and West End Cottages.
- Overtopping of the quayside at the slipway along Main Street.
- Tidal ingress along the Brandyhall River up to Adhakista Bridge.

The key thresholds and areas affects by flooding in Castletown Bearhaven are:

- 20%AEP coastal event overtops the low lying areas near the Garda station.
- 2%AEP coastal event overtops the slipway at the quayside, but Main Street is only flooded in the 1% AEP and larger magnitude events.
- Less than five buildings are affected by the 10%AEP coastal event, located around the Garda Station, but flooding is shallow.
- Up to 30 buildings are affected by the 0.5% AEP coastal event along Main Street and near the Garda Station

The greatest risk to life is associated with highest velocities along Main Street, behind the Garda Station and at Brandyhall Bridge. However, flood hazard at properties is not classed as significant or extreme until the 0.5%AEP coastal flood event.

The critical reaches of sea wall in determining coastal flood risk include:

- The car park wall between Barrack Point and Blackrock Terrace which has several gaps for access forming the low points.
- The sea wall behind the Garda Station.

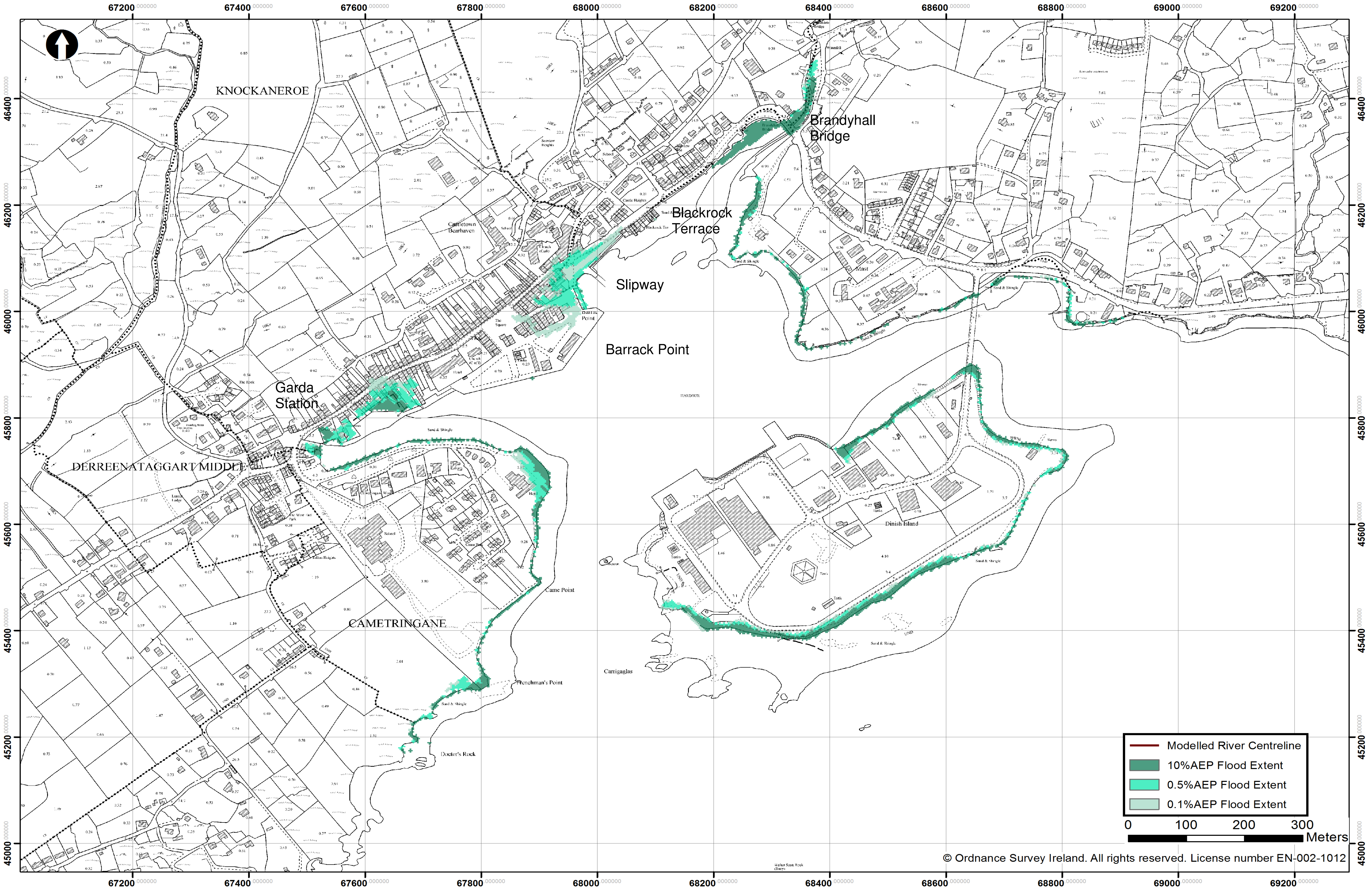
There is some uncertainty with the level at which areas behind the Garda Station and West end Cottages flood, as the LiDAR DTM was not able to accurately identify the garden fences and walls that may form a barrier to extreme coastal events. However, the local county engineers confirmed there was regular flooding in this location.

The grid resolution of 5m provides a relatively coarse estimate of flood extent. However, the grid resolution was sufficient to pick up key flow pathways along roads which were at least 5m wide and this matches with the experience of local engineers.

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

- Localised property protection and/or infilling of the gaps in the sea wall is likely to reduce flood risk.
- Flood warning is likely to be effective given that the highest astronomical tide can be predicted and > 6 hours warning can be given of storm surges from offshore buoys.

Map 9.6: Summary of Coastal Flood Risk – Castletown Bearhaven



9.5 Kenmare AFA

Map 9.7 summarises the fluvial flood risk in Kenmare for the 10%, 1% and 0.1%AEP design scenarios and Map 9.8 summarises the coastal flood risk for the 10%, 0.5% and 0.1%AEP design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Backing up from the stepping stones weir at the Finnihy Banks Estate causing water to spill over the banks and flood properties and the low-lying floodplain on the right bank.
- Backing up from the Finnihy Bridge, the bend at the Creamery Bridge and East Park Lane Bridge. This raises water levels upstream and causes water to spill out into the square and Creamery car park.
- High river levels spilling out-of-bank near East Park Lane Bridge where there are small openings in the right bank wall to the properties.

The key thresholds and areas affects by flooding in Kenmare are:

- 50%AEP fluvial event causes flooding downstream of the Finnihy Banks Estate and Convent grounds.
- 5%AEP fluvial event causes flooding at the main square and the Creamery car park.
- 50%AEP coastal event causes regular flooding of the Reenagross Park and neighbouring areas.
- Pier Road and the low lying areas at Kenmare Cooperage are at flood risk from wave overtopping in the 10%AEP and 5%AEP event respectively.
- 1%AEP coastal flood risk inundates buildings along Pier Road and Kenmare Cooperage.
- Less than five buildings are affected by the 10%AEP fluvial event, but this increases to over 250 buildings in the 1%AEP fluvial event.
- Less than 20 properties are affected by the 0.5%AEP coastal event.

The greatest risk to life is associated with highest velocities at Cromwell's bridge, near the Convent and by the stepping stones. However, flooding at properties is not classed as significant or extreme until the 2%AEP fluvial flood event. In comparison, extreme flood hazard from coastal flooding only occurs in the HEFS 10%AEP and larger events.

The critical structures in determining fluvial flood risk include:

- Finnihy Bridge, Creamery Bridge and Heritage Trail Footbridge on the Finnihy River.
- Scarteen Park and East Park Lane Bridge on the Lissaniska Stream.

The key areas of uncertainty in Kenmare are:

- Flooding in the Main Square/Market Street due to the Finnihy Bridge and East Park Lane Bridge.

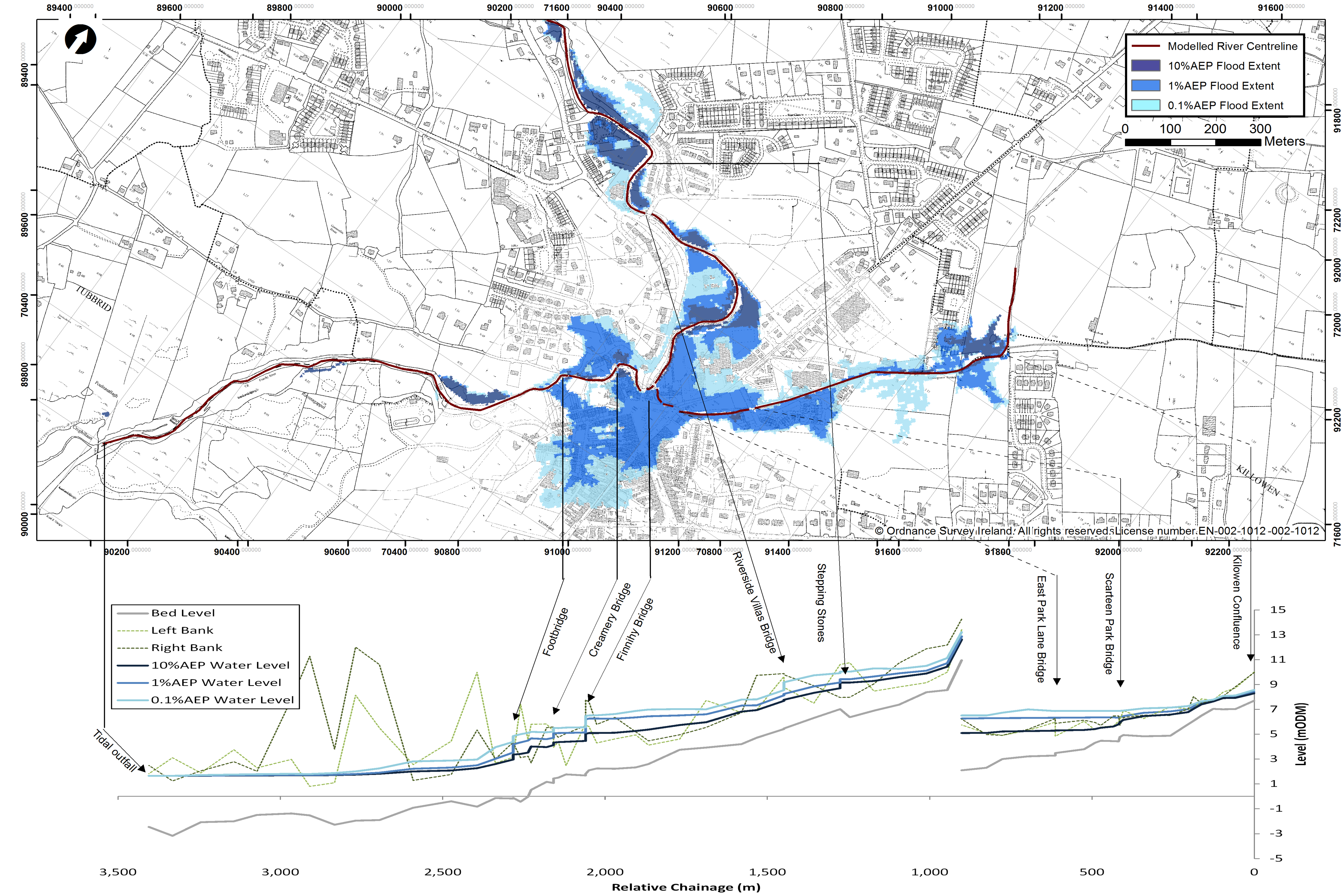
The uncertainty in this area should be carefully considered when interpreting the flood maps.

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

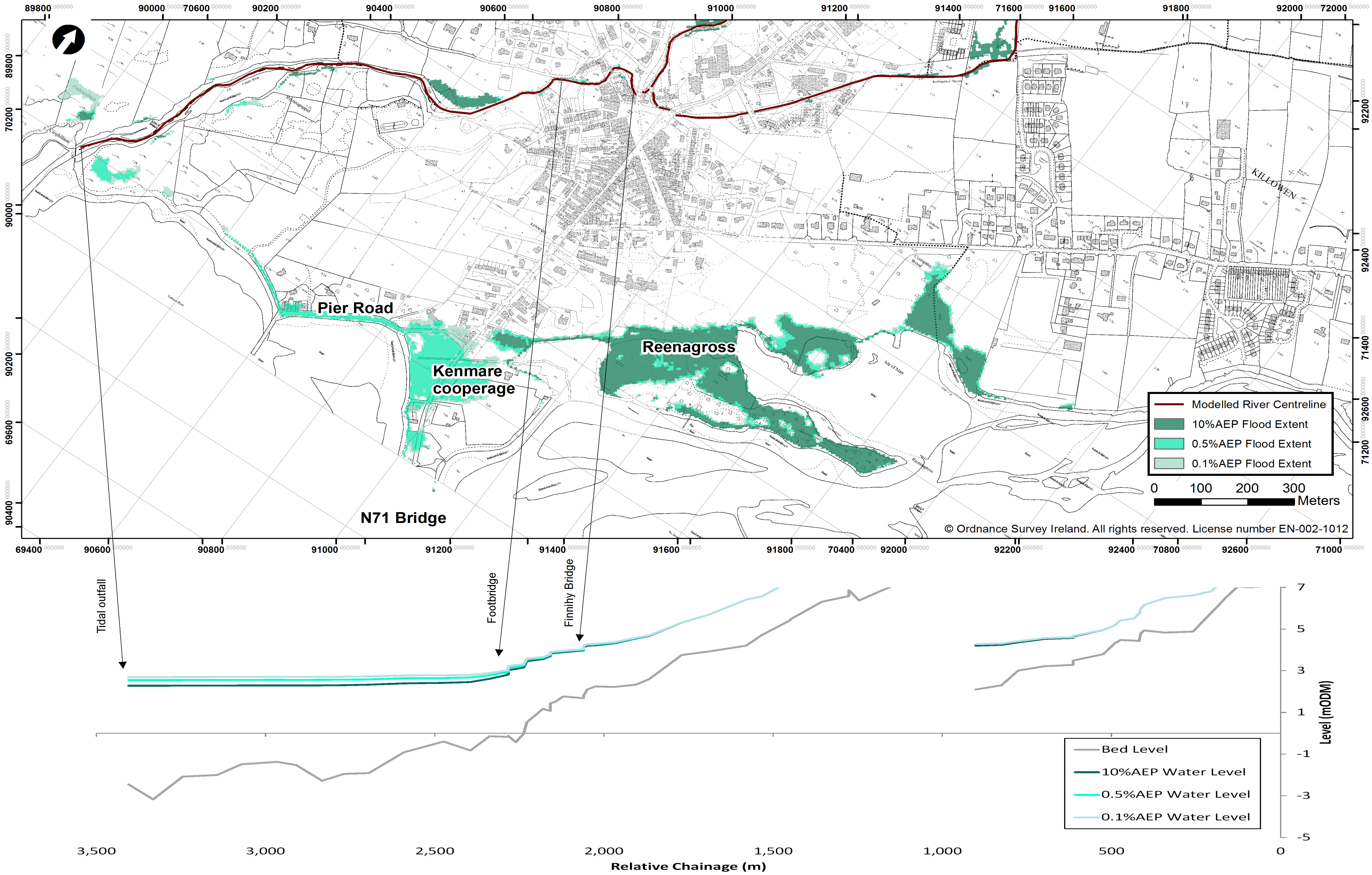
- Increased conveyance at critical structures is likely to reduce flood risk to the centre of Kenmare.
- Raising of riverside walls at key locations is likely to reduce flood risk to the centre of Kenmare.

Flood warning is unlikely to be effective given the short time to peak.

Map 9.7: Summary of Fluvial Flood Risk in Kenmare



Map 9.8: Summary of Coastal Flood Risk in Kenmare



10 Summary and Recommendations

10.1 Key Findings

The hydraulic analysis undertaken for UoM21 has developed four 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 have then been processed to map flood extent, flood depth, flood velocity and flood hazard in the four AFAs.

Historic flood events

- Recorded flood levels and extents for historic events were found to be more reliable from more recent events as data collection and verification procedures improve.
- The Kenmare model matched well with records in and around the centre of Kenmare town, but under predicted the flood level at Riverside Villas Bridge. However, there is 0.8m uncertainty in the recorded level at this location due movement of the wrack mark in the field after the event.
- Flood levels around the Finnihy Bridge in Kenmare should be treated with caution, however the flood extent is likely to be reliable as the floodplain is relatively constrained.
- The Bantry model matched well with the coastal flooding recorded on 17th October 2012, although it overestimated depths along New Street and Glengarriff Road. This discrepancy may be caused by the model not considering the impact of pumped urban drainage in the town and uncertainty in the recorded levels which are reliant on the accuracy of DTM which has a RMSE of +/-0.2m.

Sensitivity test results

- Bantry Town catchment and Kenmare AFAs are sensitive to assumptions and uncertainties in peak flow.
- Bantry Town catchment, Castletown Bearhaven and Kenmare AFAs are sensitive to the assumptions and uncertainties in downstream water level.
- Seasonal changes in vegetation or changes in roughness due to maintenance do not significantly alter flood extent and risk in any of the AFAs in UoM21. However, the roughness of the channel may improve channel capacity and/or conveyance for events which are closer to the threshold of flooding.
- Flood risk in central Kenmare AFA is sensitive to the assumptions taken for the obstruction caused by the pipe at Finnihy Bridge. The effective capacity of Finnihy Bridge should be carefully considered when interpreting flood maps, deriving flood risk management options and assessing any future flood events.

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 the high end future scenario. The key findings are summarised below.

- Durrus:
  - The fields and grounds of the Sruth Mhuilean estate in Durrus were found to be at flood risk from the 10%AEP fluvial event.
  - However, properties were only found to be at low to moderate risk in the 0.5%AEP fluvial event and larger events at this location.
- Bantry:
  - Central Bantry is at moderate risk from the 1%AEP fluvial event once the culvert capacity is exceeded.
  - Over 60 properties were found to be at significant flood hazard from coastal flooding in the 10%AEP and larger extreme tide plus surge events.
  - The Lahadane Business Park bank is overtopped by the 5-10%AEP fluvial events, but the properties are only at risk from the 2%AEP fluvial event.
  - The critical structures in determining flood risk include the culvert downstream of the Millwheel; the downstream culvert on the Sheskin and Ardnageeh Stream; the culverts on the Mileencoola East Stream and the quayside sea wall at Wolfetone Square.
- Castletown Bearhaven:
  - The low lying areas behind the Garda station were found to be at significant risk from the 20%AEP coastal event.
  - The 2%AEP coastal event overtops the slipway at the quayside, but Main Street is only flooded in the 1% AEP and larger magnitude events.
  - The quayside was also found to be at risk from wave overtopping in the 0.5%AEP Mid Range Future Scenario. The quayside was overtopped by the high tide plus surge levels in the High End Future Scenario, making the wave overtopping negligible.
  - The sea wall along Main Street has several gaps for access which form the low points through which the high tide flows.
- Kenmare:
  - The 50%AEP fluvial event causes flooding downstream of the Finnihy Banks Estate and Convent grounds.
  - The 5%AEP fluvial event causes flooding at the main square, the Creamery car park and Scarteen Park.
  - Pier Road and the low lying areas at Kenmare Cooperage are at flood risk from wave overtopping in the 10%AEP and 5%AEP event respectively.
  - Flood risk in central Kenmare was found to be sensitive to the assumptions made for the utility pipe upstream of Finnihy Bridge.
  - The critical structures were found to be Finnihy Bridge, East Park Lane Bridge, the Creamery Bridge and the Heritage Trail Footbridge.

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 based on the storage of flood waters in Kenmare and Bantry.
- 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 Kenmare, Castletown Bearhaven and Bantry.
- Reducing the roughness of the channel may increase channel capacity and reduce water levels for events which are closer to the threshold of flooding i.e. more frequent events than 1%AEP event.
- The capacity of Finnihy Bridge and neighbouring bridges in Kenmare should be carefully considered for increased conveyance options to reduce flood risk upstream, as these have been shown to be critical during the calibration and sensitivity tests.
- The capacity of the culverts in the Bantry Town catchment should be carefully considered for increased conveyance options to reduce flood risk upstream.
- Infilling works (temporary or permanent) of the access gaps in the sea wall at Bantry and Castletown Bearhaven should be considered to block flow routes before the wall itself is overtopped.

The following recommendations can be drawn from the hydraulic analysis for future analysis in the UoM21:

- 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 Bantry, given the history of pluvial flooding around Wolfetone Square.

Glossary

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
BFISOILS	Baseflow index from Irish Geological Soils dataset. Often used as a permeability indicator.
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.
DAD	Defence Asset Database
DAS	Defence Asset Survey
EU	European Union
EPA	Environmental Protection Agency
FARL	Index of flood attenuation due to reservoirs and lakes
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 Detection And Ranging. A remote based system used to determine surface elevations.
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.
$QMED_{amax}$	QMED derived from the annual maximum series at a gauged location
$QMED_{rural}$	QMED derived from physical catchment descriptors according to the Flood Studies Update methodology.
$QMED_{adj}$	QMED adjusted by the ratio of $QMED_{amax}$:$QMED_{rural}$ at a hydrologically similar Pivotal site.
$QMED_{urban}$	QMED adjusted to account for the impacts of urban areas according to the Flood Studies Update methodology.
S1085	Typical slope of the river reach between 10%ile and 85%ile along its length.
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.
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.