



South Western CFRAM Study

Draft Final Hydraulics and Flood Mapping Report,
Unit of Management 22
June 2016

The Office of Public Works



South Western CFRAM Study

Draft Final Hydraulics and Flood Mapping Report,
Unit of Management 22

June 2016

The Office of Public Works

Johnathan Swift Street,
Trim,
Co. Meath

USER NOTICE

Please read carefully the following statements and conditions of use of the data, contained in this report. Accessing the information and data denotes agreement to, and unconditional acceptance of, all of the statements and conditions.

I have read in full, understand and accept all of the above notes and warnings concerning the source, reliability and use of the data available in this report.

I agree that the Commissioners of Public Works in Ireland have the absolute right to reprocess, revise, add to, or remove any data made available in this report as they deem necessary, and that I will in no way hold the Commissioners of Public Works in Ireland liable for any damage or cost incurred as a result of such acts.

I will use any such data made available in an appropriate and responsible manner and in accordance with the above notes, warnings and conditions.

I understand that the Commissioners of Public Works in Ireland do not guarantee the accuracy of any data made available, or any site to which these pages connect and it is my responsibility to independently verify and quality control any of the data used and ensure that it is fit for use.

I further understand that the Commissioners of Public Works in Ireland shall have no liability to me for any loss or damage arising as a result of my use of or reliance on this data.

I will not pass on any data used to any third party without ensuring that said party is fully aware of the notes, warnings and conditions of use.

I accept all responsibility for the use of any data made available that is downloaded, read or interpreted or used in any way by myself, or that is passed to a third party by myself, and will in no way hold the Commissioners of Public Works in Ireland liable for any damage or loss howsoever arising out of the use or interpretation of this data.

Issue and revision record

Revision	Date	Originator	Checker	Approver	Description	Standard
A	June 2014	M Piggott	R Gamble	R Gamble	Draft	
B	September 2014	M Piggott	R Gamble	R Gamble	Draft Final	
C	January 2015	M Piggott	R Gamble	R Gamble	Minor changes	
D	June 2016	M Piggott	C Hetmank	C Hetmank	Update for River Maine	

This document is issued for the party which commissioned it and for specific purposes connected with the above-captioned project only. It should not be relied upon by any other party or used for any other purpose.

We accept no responsibility for the consequences of this document being relied upon by any other party, or being used for any other purpose, or containing any error or omission which is due to an error or omission in data supplied to us by other parties.

This document contains confidential information and proprietary intellectual property. It should not be shown to other parties without consent from us and from the party which commissioned it.

Contents

Chapter	Title	Page
	Executive Summary	i
1	Introduction	1
1.1	The CFRAM Process	1
1.2	Report Structure	3
1.3	Flood Probabilities	4
2	Data Collection, Survey and Review	5
2.1	Data Collection and Review	5
2.2	Geometric Survey Data	5
2.3	Digital Terrain Model Data	6
2.4	Land Cover Data	6
3	Hydrological Approach	10
3.1	Summary of Design Hydrology	10
3.2	Summary of Design Coastal Conditions	11
3.3	Lough Leane Analysis	12
3.4	Joint Probability	13
3.5	Integration of Hydrology and Hydraulic Modelling	16
3.6	Critical Storm Duration	18
4	Hydraulic Modelling Approach	19
4.1	Schematisation	19
4.2	River Channels	25
4.3	Structures	26
4.4	Floodplain	28
4.5	Model Run Parameters	31
5	Calibration and Sensitivity Analysis	32
5.1	Calibration	32
5.1.1	2 nd November 1980 Killarney	34
5.1.2	4 th October 2008 Castleisland	36
5.1.3	19 th November 2009, Killarney	40
5.1.4	In Bank Calibration	42
5.1.5	Validation to Historic Flood Information	44
5.1.6	Summary	49
5.2	Sensitivity Analysis	50
5.2.1	Flow	50
5.2.2	Level	57
5.2.3	Roughness	60
5.2.4	Culvert Coefficients in Milltown	64
5.2.5	Summary	66

6	Design Event Runs and Model Performance	67
6.1	Design Scenarios and Event Runs	67
6.2	Model Run Performance	70
7	Assumptions and Limitations	78
7.1	Assumptions	78
7.2	Limitations	78
8	Flood Mapping Approach	80
8.1	Approach	80
8.2	Flood Depth and Velocity Mapping	80
8.3	Flood Hazard Mapping	81
8.4	Flood Extent and Zone Mapping	81
8.5	Combined Flood Source Mapping	83
8.6	Flood Risk (Assessment) Mapping	83
8.6.1	General Flood Risk Maps	83
8.6.2	Specific Flood Risk Maps	83
8.6.2.1	Indicative Number of Inhabitants	83
8.6.2.2	Types of Economic Activity	83
8.6.2.3	Economic Risk Density	84
9	Model and Mapping Results	85
9.1	Overview	85
9.2	Castleisland AFA	85
9.3	Milltown AFA	89
9.4	Glenflesk AFA	92
9.5	Killarney AFA	95
9.6	Dingle AFA	97
9.7	Portmagee AFA	101
10	Summary and Recommendations	103
10.1	Key Findings	103
10.2	Recommendations	104
	Glossary	105

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

A total of eight hydraulic models have been developed for the six Areas for Further Assessment (AFAs) and Medium Priority Watercourse downstream (MPW) to assess fluvial and coastal flood risk for various flood probabilities.

The majority of models used a 1D/2D ISIS/TUFLOW hydrodynamic ally linked approach such that water can flow between the river and floodplain during the event to simulate the observed flood mechanisms within AFAs. 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. However, Portmagee was developed with a 2D TUFLOW only approach to assess coastal flood risk as it was not deemed to be at risk from fluvial flooding.

The Castleisland and Killarney models were calibrated to flood events of 4th October 2008, 2nd November 1980 and 19th November 2009 where sufficient data enabled full calibration of the hydraulic parameters. The Maine model was also calibrated for high flow in-bank events on the 4th October 2008 and 12th January 2010 events. The Milltown, Glenflesk and Dingle models were validated against reports of recurring flooding to ensure representation for historic flooding. Sensitivity tests were undertaken on flow, downstream level and Manning's 'n' for all models. An additional sensitivity test was undertaken on the culvert coefficient at Milltown following comments from the local area engineers.

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 UoM22 and the extent of the hydraulic models which are the subject of this report.

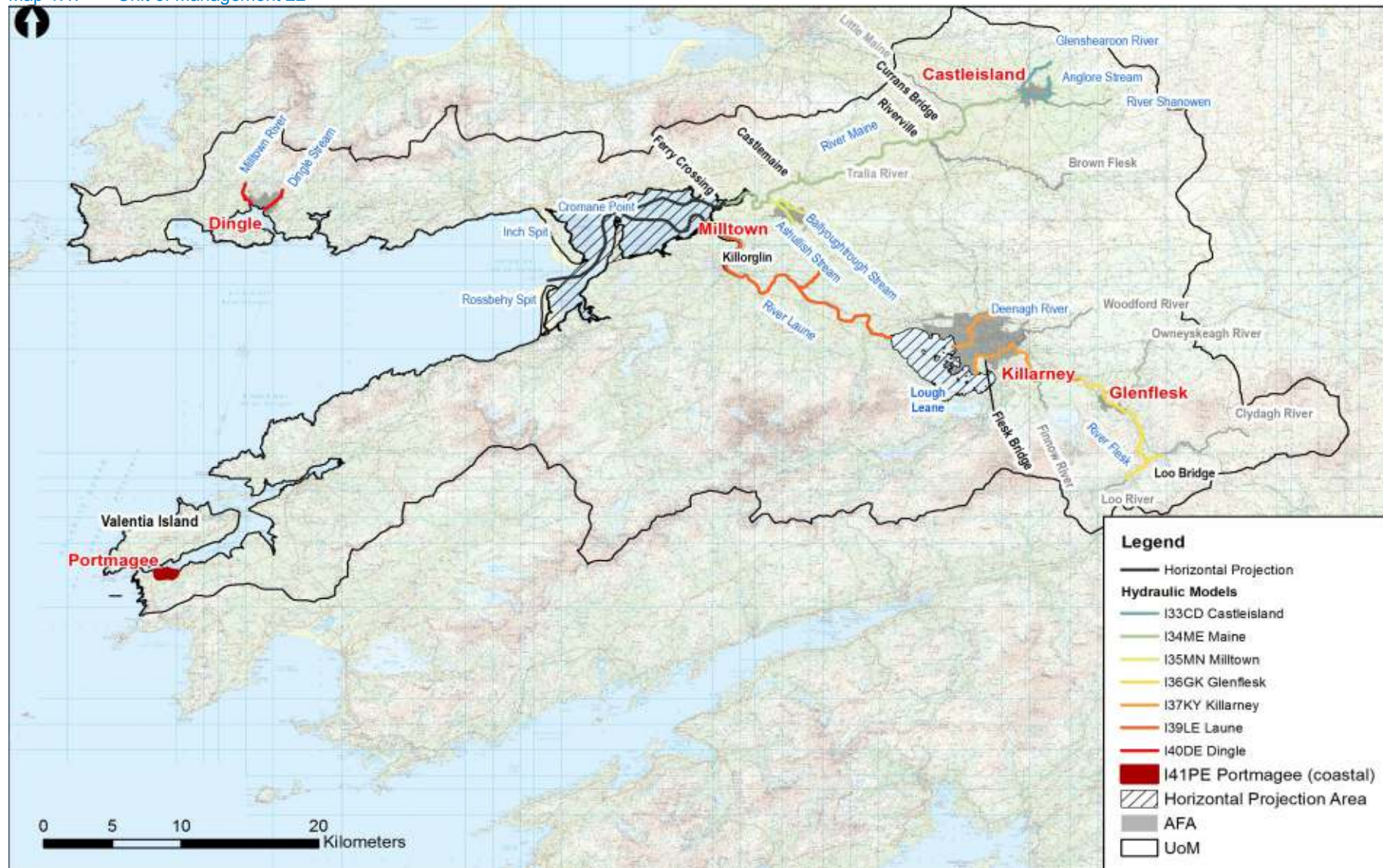
The overarching aims of the SW CFRAM Study are as follows:

- 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 of people, economy and environment;
- Development and assessment of flood risk mitigation options; and,
- Development of the Flood Risk Management Plans (FRMPs).

Map 1.1: Unit of Management 22



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 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	<ul style="list-style-type: none"> ■ The SW CFRAM process ■ Report structure ■ Flood probabilities
2. Data Collection, Survey and Review	<ul style="list-style-type: none"> ■ Summary of data sources ■ Review of all topographical and land cover data used
3. Hydrological Approach	<ul style="list-style-type: none"> ■ Summary of design inflows and downstream conditions ■ Summary of joint probability ■ Integration of design hydrology into the hydraulic model
4. Hydraulic Modelling Approach	<ul style="list-style-type: none"> ■ 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	<ul style="list-style-type: none"> ■ Discussion of calibration events ■ Discussion of sensitivity tests on key parameters
6. Design Runs and Model Performance	<ul style="list-style-type: none"> ■ List of design runs ■ Discussion of model convergence and performance
7. Assumptions and Limitations	<ul style="list-style-type: none"> ■ The key limitations and assumptions of the models and associated data
8. Flood Mapping Approach	<ul style="list-style-type: none"> ■ 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	<ul style="list-style-type: none"> ■ 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	<ul style="list-style-type: none"> ■ Conclusions and key findings from the hydraulic analysis ■ Summary of flood hazard in the Unit of Management ■ Recommendations for flood mitigation option development and the FRMP ■ Recommendations for future improvements in the hydraulic modelling

1.3 Flood Probabilities

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

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

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

Table 1.2: Flood Probabilities

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

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

2 Data Collection, Survey and Review

2.1 Data Collection and Review

A range of different data sources have been used to undertake the hydraulic analysis for the SW CFRAM Study. Table 2.1 lists the data used in Unit of Management 22 and the confidence in each dataset based on the review in the following sections.

Table 2.1: Summary of Data Used

Type	Details	Owner	Date Captured	Confidence Level
Geometric Survey Data	River channel and structure survey and photographs of all HPWs and MPWs in UoM22	OPW	As part of this study 2012-2013	+/- 0.1
Detailed Digital Terrain Models	Filtered LiDAR data for AFAs	OPW	2012	+/- 0.1
National Height Model	IFSAR coarse elevation data with national coverage	OPW	2010	+/- 0.5 to 1.0
OSI Mapping	Building footprints and vector data of land cover	OSI	2010	No elevation data included. Not Applicable

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

2.2 Geometric Survey Data

As part of this study, extensive river channel survey was undertaken of all the High Priority Watercourses (HPWs) and Medium Priority Watercourses (MPWs) in UoM22 between November 2012 and July 2013 by Murphy Surveys Ltd (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. The detailed South West CFRAM Contract 5 Survey is available in a separate survey report (August 2013).

All of the geometric survey data captured by the surveyor was reviewed with checks carried out on 10% of the cross sections. Using GPS survey equipment spot levels checks were carried out on structures and cross sections captured by the surveyor. The levels were reviewed and differences compared at bank crest. The average difference between the levels of the survey and the spot checks was found to be 113 mm. This is considered to be a good correlation when considering that the comparison points were; mostly on rough ground; the exact location of the bank crest could vary from the original survey due to access and vegetation; and, the crest could be subject to footpath erosion where the river bank is unsupported earth.

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

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

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

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

Modifications made to individual structures and river channel sections have been justified in the model build proforma for each AFA which can be found in the Appendices.

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 September 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 deemed to be appropriate for use without further adjustment. The vertical accuracy was found to be 0.05m on average within urban areas, such as Castleisland, increasing to 0.2m in more rural areas such as upstream of Glenflesk and Killorglin.

LiDAR was not available on the Upper Laune, Flesk and Maine downstream of Castleisland to Currans Bridge. Therefore, IFSAR data from OSI's national height model has been used to create the DTM for hydraulic modelling and flood mapping in these reaches. IFSAR has a lower vertical accuracy than LiDAR of ± 0.5 m on average. When the IFSAR data was compared with river channel survey on the floodplain and discrepancies between -0.2 and +1.30m were found in some locations on the Laune. Therefore, the IFSAR data was adjusted to meet the river channel survey points and then joined with the LiDAR data to create a complete DTM. Every effort has been made to ensure a consistent transition from LiDAR to IFSAR but some uncertainty remains in the areas which use IFSAR due to the poorer data quality.

2.4 Land Cover Data

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

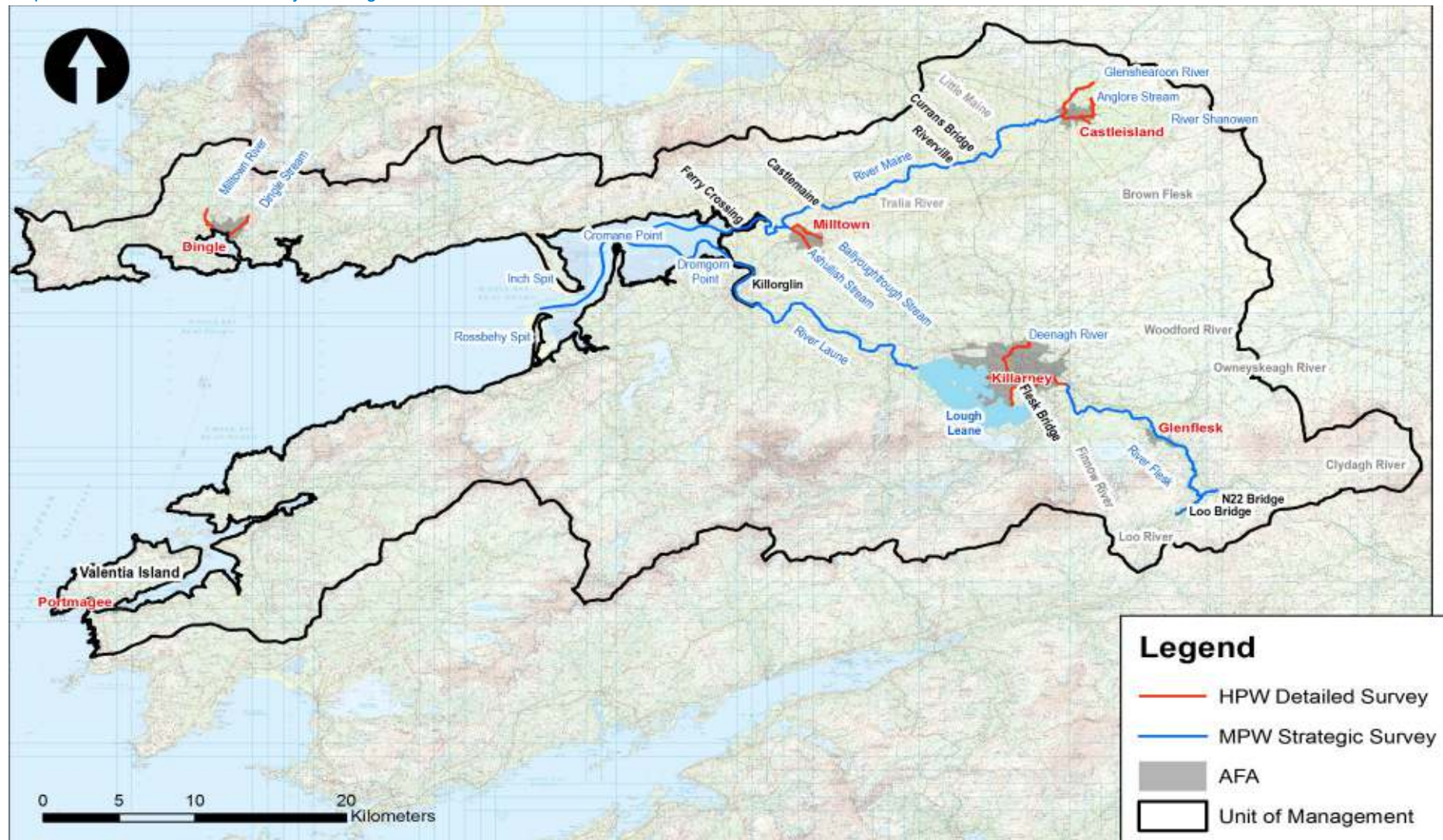
- Building footprints derived from OSI mapping
- 1:1000, 1:2500 and 1:5000 vector OSI Mapping

- Surface cover detailed in the geometric survey and survey photographs
- Site visits

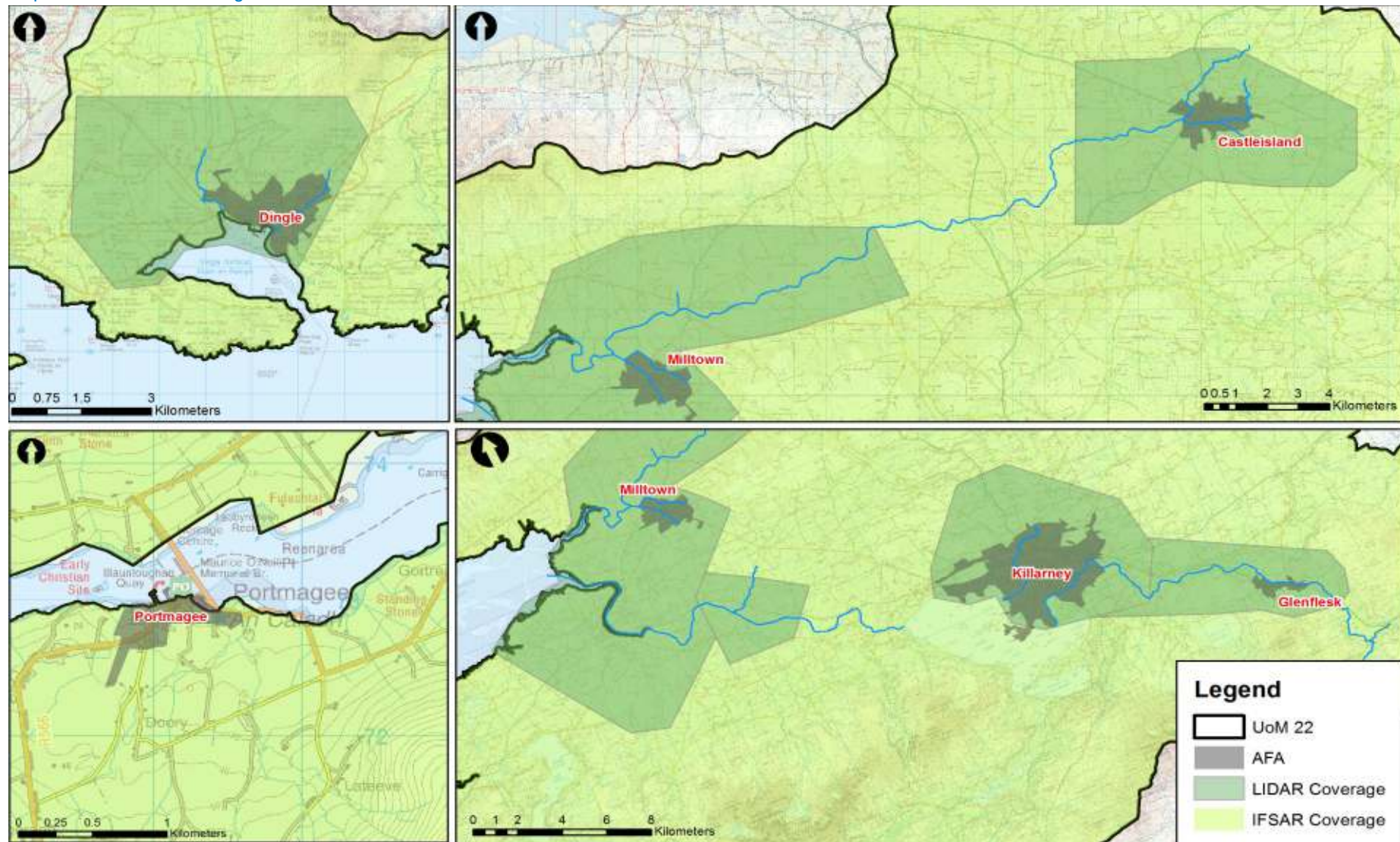
The mapping datasets were used in the first instance to classify land cover within each AFA into broad surface types of river bed and standing water; river banks; dense vegetation; pasture, parkland and arable; buildings; and, hard-standing urban areas. The land cover was subsequently refined during 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 UoM22



Map 2.2: LiDAR Coverage in UoM22



3 Hydrological Approach

3.1 Summary of Design Hydrology

As part of the previous UoM22 Hydrology Report, design peak flows and hydrographs were derived at hydrological estimation points for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP fluvial flood events.

The hydrological estimation points were located in every AFA and along the MPWs downstream. The HEP 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 gauge within the AFA where available or the hydrological similar pivotal sites of 22022, 22003, 22006, 22014, 22035 and 22039 as well as 36021, 19014 and 25044. The White Gauge of 22009 was also used to derive QMED along the River Deenagh. However the gauge was not deemed suitable to estimate extreme flows above QMED. Table 3.1 summarises the design peak flows for each catchment in the AFAs for ease of reference.

Table 3.1: UoM22 Design Peak Flood Flows at Key Locations

HEP	Gauge	Flow (m³/s)							
		50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.1%AEP
River Laune catchment									
22_3712_1	22039 (Clydagh Bridge Gauge)	57.9	72.0	82.1	92.8	108.8	122.6	138.2	182.9
22_3372_6	22006 (Flesk Bridge Gauge)	172.6	205.2	224.4	251.9	295.1	334.8	379.3	494.6
22_510_2	22035 (Laune Bridge Gauge)	114.2	132.1	148.9	168.4	201.8	225.5	252.4	329.4
22_4001_4+	Laune downstream	186.0	215.2	242.4	274.2	328.6	367.3	411.0	536.4
22_4003_14	22009 (White Bridge Gauge)	12.0	13.6	15.4	17.4	21.1	24.3	28.0	36.9
River Maine Catchment									
22_1587_3	22014 (Castleisland Gauge)	29.7	38.0	43.6	48.9	55.5	61.9	69.6	91.9
22_3101_1	22003 (Riverville Gauge)	144.0	181.8	210.5	242.6	292.7	338.2	392.0	558.5
22_3958_1+	Maine Downstream	203.6	257.1	297.8	343.2	414.0	478.4	554.4	790.0
Milltown Catchment									

HEP	Gauge	Flow (m ³ /s)							
		50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.1%AEP
22_3116_4	Ashullish – Ballyoughtragh U/s Confluence	3.8	4.8	5.4	6.1	7.2	8.1	9.1	12.0
22_3958_2	Ashullish Stream Downstream	6.7	8.3	9.5	10.7	12.6	14.1	15.9	21.1
22_3425_9	Ballyoughtragh Stream Downstream	7.3	9.0	10.3	11.6	13.6	15.3	17.2	22.8
Dingle Catchment									
22_1712_2	Milltown Gauge (22022)	23.0	32.0	40.9	49.0	61.0	70.8	81.4	109.9
22_3437_1	Dingle Stream Downstream	5.1	6.5	7.4	8.4	9.9	11.2	12.7	16.9

The annual maximum flood hydrographs were standardised and compared to derive the width exceedance for specific percentage flows at gauges on the River Maine, Flesk, Deenagh, Laune and Milltown (Dingle) Rivers. The design median flood hydrograph was derived from the width exceedance analysis. The FSU WP 3.1 UPO-ERR-gamma curve was fitted to the design median flood hydrographs and the parameters applied to derive the design hydrograph shape for the ungauged HEPs.

The 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 UoM22 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. The outfall of the Laune and Maine is at the upstream end of Castlemaine Harbour which is over 13km from the nearest prediction point. Furthermore, complex estuarine features such as Cromane Point modify the normal tidal levels and progression up the estuary. Therefore, the design total tide plus surge levels and tidal hydrographs were transformed from the open coast up the estuary based on the ICPSS analysis, Admiralty prediction points and observed water level at Castlemaine gauge. The resultant design levels are provided in Table 3.2.

Table 3.2: UoM22 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
Portmagee Harbour	ICPSS point SW16	2.15	2.25	2.32	2.38	2.46	2.52	2.59	2.73
Dingle Harbour	ICPSS point SW22	2.20	2.30	2.38	2.45	2.54	2.61	2.68	2.85
Dingle Bay at Inch Point	ICPSS point SW20	2.37	2.48	2.56	2.63	2.73	2.81	2.88	3.06
Cromane Point Castlemaine Harbour	Transformed	2.70	2.80	2.88	2.95	3.04	3.11	3.18	3.35
River Maine	Transformed to meet frequency at Castlemaine Gauge	3.00	3.11	3.19	3.26	3.35	3.42	3.49	3.66

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 3.4.

3.3 Lough Leane Analysis

The assessment for Lough Leane uses extreme value flood frequency analysis to determine design lake levels for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP events rather than hydraulic modelling of the lake which has a flat water level profile. Statistical flood frequency analysis was undertaken on Tomies Pier (22071) and BVM Park (22082) level gauges located on the Lough and water level profiles for extreme events assessed. The water level estimates at Tomies Pier were used as the design lough levels for the corresponding design %AEP event in Killarney (Table 3.3).

Table 3.3: UoM22 Design Lough Leane Levels

Location	Source	Design Water Level (mODM)							
		50%AEP	20%AEP	10%AEP	5%AEP	2%AEP	1%AEP	0.5%AEP	0.1%AEP
Lough Leane	Tomies Pier (22071)	19.23	19.52	19.71	19.88	20.11	20.28	20.46	20.85

3.4 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 various combinations to achieve a design %AEP event which can be described by the joint probability. 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.

Fluvial Dominant Events

The fluvial-fluvial dependence was guided by the methodology set out in Flood Studies Update Work Package 3.4. In UoM22, the joint probability of tributaries tended to be the more frequent smaller events in order to achieve the design flow on the main watercourse. In order to simplify the modelling process, the closest design AEP to the joint probability estimate was selected. The flow was interpolated where the joint probability was half way between two design AEPs (Table 3.4).

Table 3.4: Summary of Joint Probabilities Applied for Fluvial Dominant Events

Applicable Models	Overall %AEP (Fluvially dominated event)	Design Flood Event Occurs on	Main River Inflow %AEP	Typical Tributary River Inflows %AEP	Coastal %AEP (where the downstream limit is the open coast)
Castleisland Maine Glenflesk Killarney Laune	50%AEP	Main River	50%	50%	MHWS
		Tributary River	50%	50%	MHWS
	20%AEP	Main River	20%	50%	MHWS
		Tributary River	50%	20%	MHWS
	10%AEP	Main River	10%	20%	MHWS
		Tributary River	20%	10%	MHWS
	5%AEP	Main River	5%	20%	MHWS
		Tributary River	20%	5%	MHWS
	2%AEP	Main River	2%	10%	MHWS
		Tributary River	10%	2%	MHWS
	1%AEP	Main River	1%	5%	MHWS
		Tributary River	5%	1%	MHWS
	0.5%AEP	Main River	0.50%	2%	MHWS
		Tributary River	2%	0.50%	MHWS
	0.1%AEP	Main River	0.10%	1%	MHWS
		Tributary River	1%	0.10%	MHWS

Milltown Dingle	50%	Main River and Tributary Rivers (as tributary provides equal contribution to flow downstream)	50%	50%	MHWS
	20%		20%	20%	MHWS
	10%		10%	10%	MHWS
	5%		5%	5%	MHWS
	2%		2%	2%	MHWS
	1.00%		1.00%	1.00%	MHWS
	0.50%		0.50%	0.50%	MHWS
	0.10%		0.10%	0.10%	MHWS
Portmagee		Not at fluvial risk. Therefore not assessed.			

Coastal Dominant Events

The extreme fluvial flow estimates at the outfall of the Maine; outfall of the Laune; and, Milltown River were assessed with the ICPSS total tide plus surge levels to derive the joint probability combinations between fluvial and coastal events based on the DEFRA FD2308_TR1 desk-based assessment tool in accordance with GN20¹. 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. This resulted in ten different combinations of fluvial flows and tide plus surge levels for each design %AEP and the two critical scenarios for flood risk selected.

Previous studies (Lee CFRAM Study, River Clyde Flood Management Strategy, River Thames T2100 studies) have undertaken extensive sensitivity testing on a range of different combinations of fluvial flows and tidal levels to generate the 0.5%AEP design event, and found the following two scenarios to be critical to the flood extent at the target 0.5%AEP event:

- 1%AEP fluvial flow combined with the MHWS tide; and
- 50%AEP fluvial flow combined with 0.5%AEP tide plus surge level.

Therefore, the SW CFRAM Study has taken a similarly pragmatic approach and limited the joint probability analysis to one fluvial dominate scenario and one tidally dominant scenario for models affected by both fluvial and coastal flooding:

- Design %AEP fluvial flow combined with MHWS tide
- Design %AEP tide plus surge combined with 50% to 70%AEP fluvial flow

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 that the highest still water level combined with smallest wave height was the critical scenario for wave overtopping at Dingle.

In order to simplify the modelling process, the closest design AEP to the joint probability estimate was selected. The flow was interpolated where the joint probability was half way between two design AEPs. The resultant joint probabilities are provided in Table 3.4 and 3.5.

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

Table 3.5: Summary of Joint Probabilities Applied for Coastal Dominant Events

Table 6.10: Summary of Joint Probability Applied for Coastal Dominant Events					
Applicable Models	Overall %AEP (Coastal dominated event)	Design Flood Event Occurs on	Main River Inflow %AEP	Tributary River Inflows %AEP	Coastal %AEP (where applicable)
Maine	50%	Coast	50%	50%	50%
Laune	20%		50%	50%	20%
Dingle	10%		50%	50%	10%
	5%		50%	50%	5%
	2%		50%	50%	2%
	1.00%		50%	50%	1.00%
	0.50%		50%	50%	0.50%
	0.10%		50%	50%	0.10%
Portmagee	50%		Coast	No fluvial inflows	No fluvial inflows
	20%	20%			
	10%	10%			
	5%	5%			
	2%	2%			
	1.00%	1.00%			
	0.50%	0.50%			
	0.10%	0.10%			
Milltown		Coastal risk from the Maine assessed as part of the Maine Model. AFA itself not at coastal risk.			
Castleisland		Not at coastal risk. Therefore not assessed,			
Glenflesk					
Killarney					

3.5 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 disproportionately 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 ends of the models for the Dingle, Milltown, Deenagh (Killarney) and Woodford (Killarney) catchments because the intermediate catchments were 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 compared to the design peak flows derived at the target HEPs to assess performance. The hydrological inflows were iteratively phased such that the modelled flows were within 10% of the design peak flows along the reach as part of the hydraulic modelling process. However, it should be noted that the design fluvial flows do not consider the following hydraulic processes:

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

Therefore, it was not appropriate to compare modelled flows upstream of confluences to the design HEPs where there are significant out-of-bank flows. Table 3.6 details the timing adjustments made to the inflow hydrographs to achieve the design peak flows at the target HEPs for each reach.

Section 6.2 discussed the performance of the modelled flows against the design flows.

Table 3.6: Phasing of Inflows

Model	Sub-catchment	Time Shift Applied to the Tributary Inflows to Achieve the Design Peak Flows at the target HEPS (Hours)
Castleisland AFA	Maine	0.00
	Glenshearoon	0.00
Maine MPW	Tributaries to Brown Flesk	2.00
	Tributaries from Inchiveema to Groin	6.00
Milltown AFA	Ashullish	0.00
	Ballyoughtragh	0.00
Glenflesk AFA	Flesk, Loo and tributaries to Oweneskagh	0.00
Killarney AFA	Finnow	-7.00
	Flesk, Woodford and Deenagh	0.00
Laune MPW	Loe and upper tributaries	30
	Gweestin	17.5
Dingle AFA	Dingle Stream	0.00
	Milltown Stream	0.00
Portmagee AFA		Not Applicable

The tributaries to the Laune were delayed because Lough Leane significantly attenuates flows resulting in a much later peak. The Finn timer was phased 7 hours before the Flesk because the Dromickbane gauge on the Finn timer statistically peaks earlier than the River Flesk due to the storm track movement from northwest to southeast.

The design tide plus surge hydrographs discussed in Section 3.2 were used to form the downstream boundary conditions for the hydraulic models. An iterative approach was used to phase the design tide plus surge hydrographs so that the peak tide coincides with the peak flow in the AFA. This phasing is a conservative assumption of combined flood risk in line with the joint-probability analysis discussed in Section 3.3 above. Table 3.7 outlines the downstream conditions applied and time by which the tidal hydrograph was adjusted in order to meet the peak river flow.

Table 3.7: Downstream Boundary Conditions

Model	Downstream Condition	Time Adjustment to Coincide Peak Tide with Peak Flow (Hours)
Castleisland AFA	Fluvial downstream boundary set by Flow-Stage boundary	Not Applicable
Maine MPW	Full tidal boundary at the outfall in Castlemaine Harbour.	3.0
Milltown AFA	Tidal boundary set by the Maine model.	Extracted from the Maine model.
Glenflesk AFA	Fluvial downstream boundary set by Flow-Stage boundary	Not Applicable
Killarney AFA	Fluvial downstream boundary set by Lough Leane	Not Applicable

Model	Downstream Condition	Time Adjustment to Coincide Peak Tide with Peak Flow (Hours)
Laune MPW	Full tidal boundary at the outfall in Castlemaine Harbour.	22.5
Dingle AFA	Full tidal boundary along the coast.	2.5
Portmagee AFA	Full tidal boundary along the coast.	Not applicable as there is no fluvial inflows

3.6 Critical Storm Duration

In UoM22, the median width hydrographs have been derived at the gauged locations to establish the design storm duration at target HEPs across each catchment. The duration of the tributary inflows were based on the gauged duration. The intermediate inflows account for the difference in duration between the target HEPs within the same hydrological catchment. Table 3.8 outlines design storm durations for UoM22.

Table 3.8: Critical Storm Durations

Gauge ID	Name	Watercourse	Applicable Reach/AFA	Design Duration (Hours)
22014	Castleisland	Maine	Castleisland	13
22003	Riverville	Maine	Maine Milltown	24
22022	Milltown	Milltown (Dingle)	Dingle	11
22039	Clydagh Bridge	Clydagh	Flesk (upstream of Oweneskagh) Glenflesk	15
22006	Flesk Bridge	Flesk	Flesk (downstream of Oweneskagh) Killarney (tributaries to the Laune)	29
22009	White Bridge	Deenagh	Killarney	23
22035	Laune Bridge	Laune	Laune	237*

*Significantly affected by attenuation of Lough Leane. Therefore the duration at Flesk Bridge (29hrs) used to inform tributary inflows

4 Hydraulic Modelling Approach

4.1 Schematisation

Table 4.1 outlines the approach for each of the eight models which cover the six AFAs and MPW reaches downstream. Maps 4.1 to 4.3 present the areas and reaches modelled.

Table 4.1: UoM22 Model Approach

Model ID	AFA/MPW	Modelled Rivers	Approach	Length Modelled (km)	Upstream Limit(s) (Irish NGR)	Downstream Limit(s) (Irish NGR)
I33CD	Castleisland AFA	River Maine River Shanowen Glenshearoon River Anglore Stream	1D/2D ISIS/TUFLOW	10.1	101333,109073 101418,111041 101320,111975	098629,109333
I34ME	Maine MPW	River Maine Annagh Stream	1D ISIS to Currans Bridge 1D/2D ISIS/TUFLOW Currans Bridge to Castlemaine Harbour	29.5	098629,109333	077744,101293
I35MN	Milltown AFA	Ballyoughtrough Stream Ashullish Stream	1D/2D ISIS/TUFLOW	4.3	083165,100740 082695,100032	081259,101423
I36GK	Glenflesk AFA	River Flesk Clydagh River Loo River Owneyskeagh River	1D ISIS leading to 1D/2D ISIS/TUFLOW	16.7	109709,081837 106976,080040 106980,086634	103586,087657
I37KY	Killarney AFA	River Flesk River Deenagh Woodford River	1D ISIS from Old Flesk Bridge to White Bridge Killarney 1D/2D ISIS/TUFLOW from White Bridge Killarney to Lough Leane	18.1	103586,087657 099400,090607 097240,092782	096084,088014 094466,090214
I39LE	Laune MPW	River Flesk Gweestin River	1D ISIS	22.2	089869,090909	077220,099220
	Lough Leane MPW	N/A	Horizontal projection Combined with the mapping for I39LE	N/A	096084,088014 094466,090214	089869,090909
	Castlemaine Harbour MPW	N/A	Horizontal projection Combined with the mapping for I39LE	N/A	077220,099220 077744,101293	065210,095750
I40DE	Dingle AFA	Dingle Stream Milltown Stream	1D/2D ISIS/TUFLOW	4.3	045666,102210 042976,102772	044455,100745 043415,101309
I41PE	Portmagee AFA	None (Coastal)	2D TUFLOW	N/A	N/A	037275,073046

Modelling of AFAs

A hydrodynamically linked one-dimensional (1D) and two-dimensional (2D) approach has been taken for Castleisland, Milltown, Glenflesk, Killarney and Dingle. The HPWs have been modelled in ISIS 1D 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. TUFLOW 2D modelling software version 2013-AC has been hydrodynamically linked to the ISIS model and used to simulate out-of-bank and river-floodplain interactions.

TUFLOW two-dimensional modelling software (version 2013-AC) has been used to model the floodplains in all the AFAs in order to simulate complex flow paths and variable velocities across the urban floodplains. The 2D approach is also the most appropriate to simulate coastal flooding, such as found in Portmagee, as it is able to simulate the multi-directional flow paths as the sea overtops the quayside, coastal roads and sea walls.

Modelling of MPWs

The MPW reaches have typically been modelled using ISIS to simulate both in-bank and out-of-bank flows by extending the river sections across the floodplain. In order to improve hydrological routing and simplify modelling, the Flesk MPW upstream of Glenflesk has been modelled with Glenflesk AFA (I36GK), and the Flesk MPW downstream of Glenflesk has been modelled with Killarney AFA (I37KY).

However, extended sections are inappropriate for the Maine MPW downstream of Currans Bridge because this approach would overestimate flow across the floodplain which is disconnected from the river by the raised embankment. Therefore, the lower Maine MPW model takes a 1D/2D approach to more accurately simulate the raised channel above the floodplain, flow over the raised embankments and complex flows across the low-lying floodplain.

The assessment for Lough Leane is a special case where horizontal projection of the design lough levels have been mapped rather than full hydraulic modelling. This approach is appropriate for Lough Leane as the water level determines the extent of flooding rather than volume overtopping the banks. This approach has two key benefits over detailed hydraulic modelling:

- It provides an accurate estimate of design water levels without the need for extensive bathymetric survey of the Lough itself.
- It is based on observed gauge data rather than taking assumptions that simplify lake storage and complex lake currents.

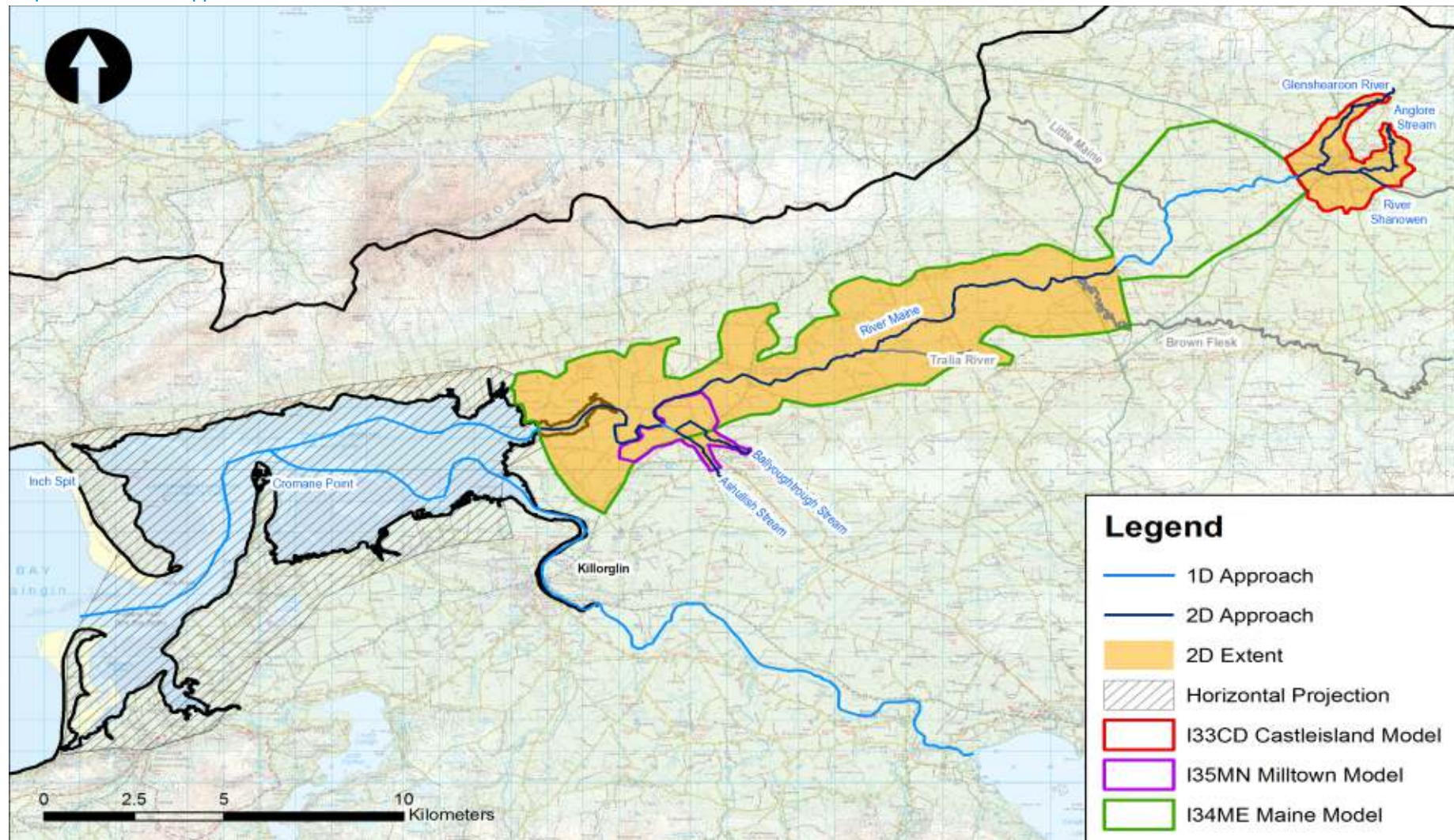
Castlemaine Harbour downstream of the Laune and Maine outfalls is tidally-dominated. Therefore, the design water level profile for the extreme coastal events has been horizontally projected across the estuary. This approach is sufficient to generate reliable flood extents and depths required for this MPW reach.

Therefore, there is no hydraulic model for Lough Leane and Castlemaine Harbour. The flood maps for these MPW reaches are included as part of the Laune flood maps (I39LE).

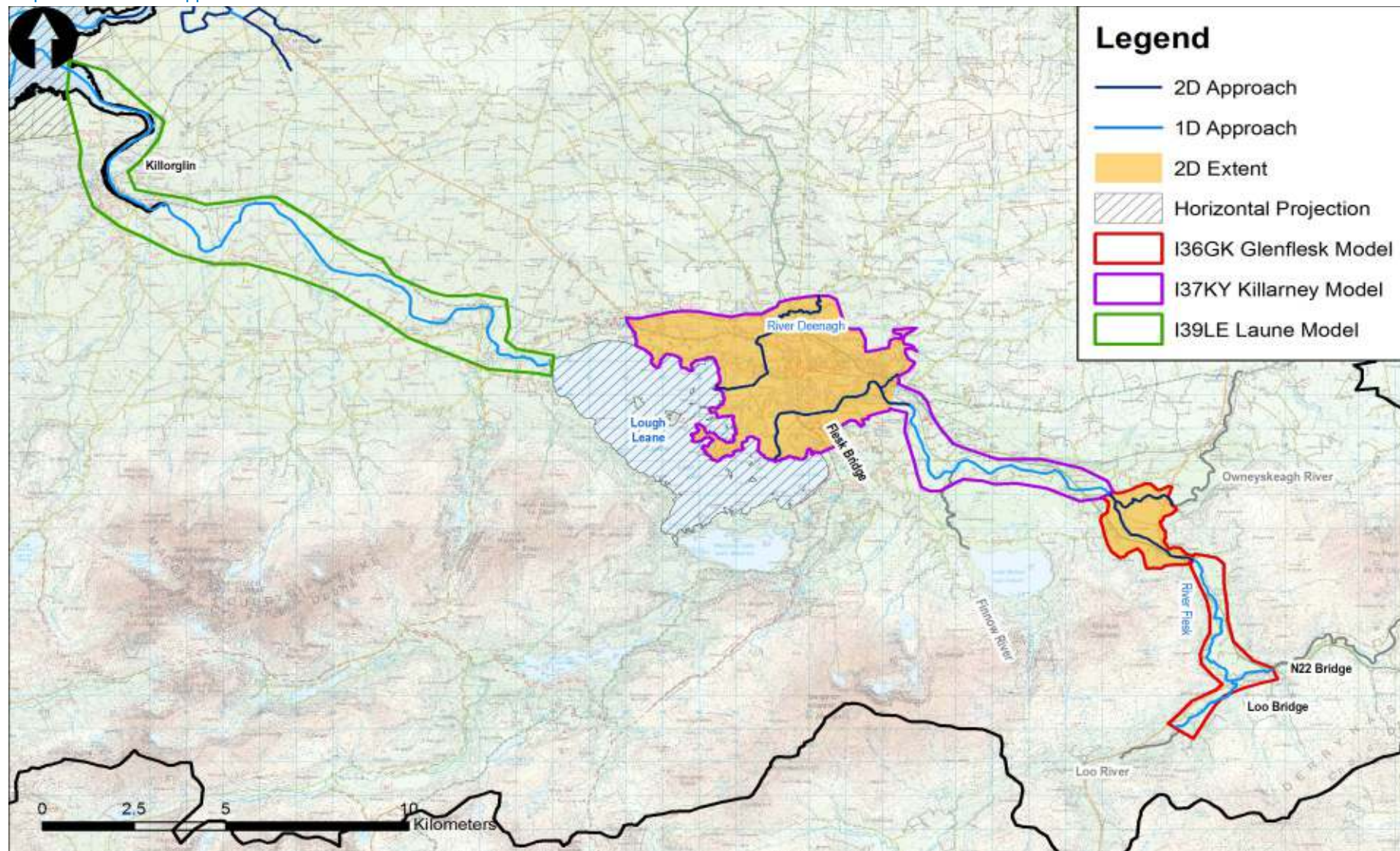
The River Flesk and River Maine catchments were split into several model reaches to more accurately model the upstream AFAs. The model reaches tended to be split at weirs or steep slope sections which form a hydraulic control and defined by the modelled stage-discharge relationship. This ensures that the water levels from the upstream model reach are consistent with the downstream model results

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.

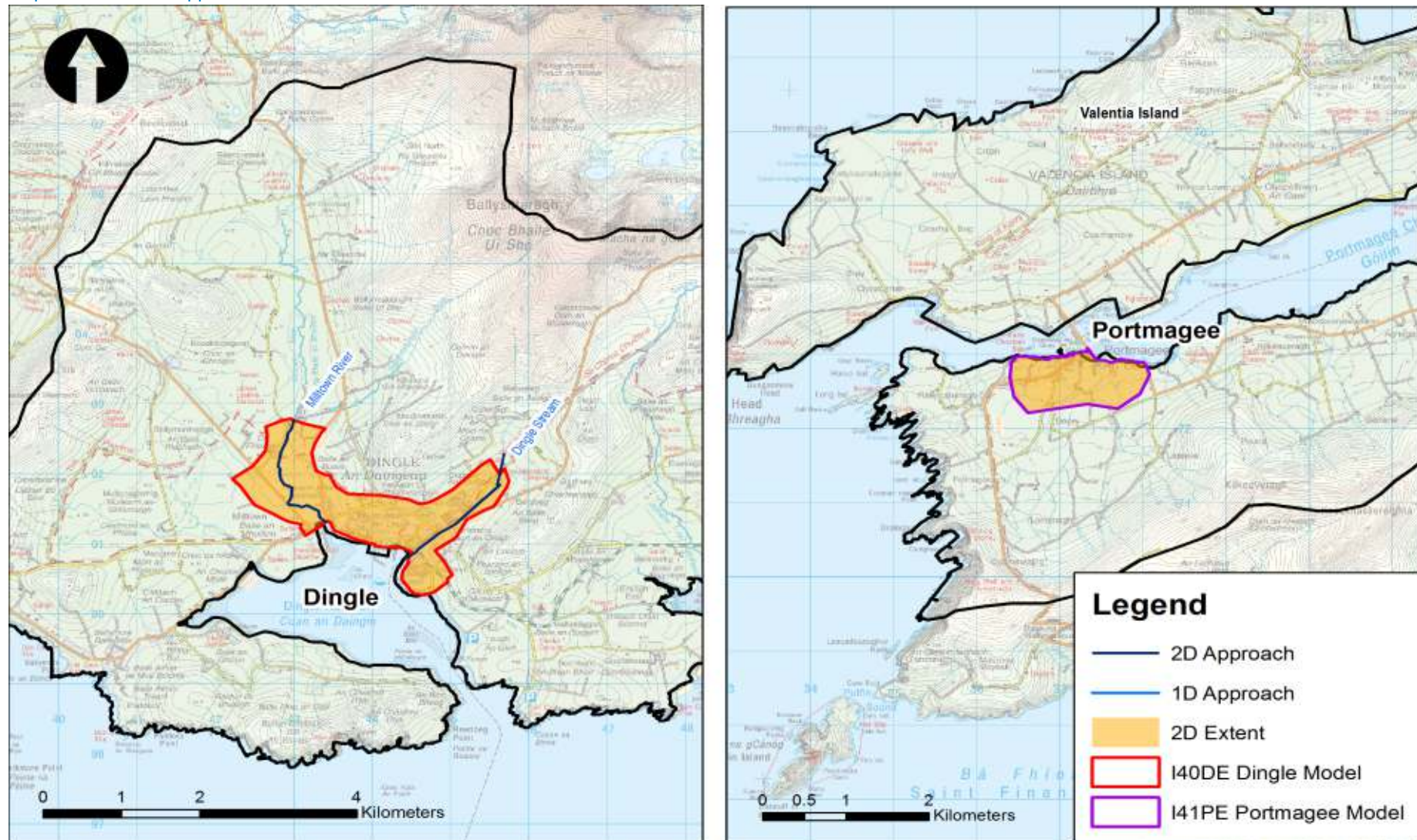
Map 4.1: Model Approach in the Maine Catchment



Map 4.2: Model Approach in the Laune Catchment



Map 4.3: Model Approach for Remote AFAs in UoM22



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 UoM22 with the exceptions of the following:

- Correction for skew angle surveyed at Herbert's Bridge on the River Maine in Castleisland.
- Correction for skew angle surveyed at the N22 (Brewsterfield) bridge on the River Flesk.
- Open channel sections were interpolated at the rapid section upstream of the N22 road bridge on the River Flesk to maintain stability.
- Interpolate sections were added and bank levels modified along Milltown River in Dingle to stabilise the exchange of flows between the channel and floodplain round the meander bend on the Commons.

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 broader reach.

The exception are the bridges in Dingle 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 some of the bridge structures.

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

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

Material Type	Selected Manning's 'n'	Applicable Reaches
Active river bed with gravel to boulders	0.045 to 0.050	River Flesk Anglore Stream
Active river bed with silts	0.040 to 0.045	River Maine
Light brush and/or grass during winter	0.060 to 0.075	Shanowen River Glenshearoon River Ashullish and Ballyoughtrough Streams Milltown and Dingle Streams White River River Flesk and Oweneskagh River Laune
Dense vegetation year round	0.080	Anglore Stream River Maine in some reaches

Source: Chow 1959

4.3 Structures

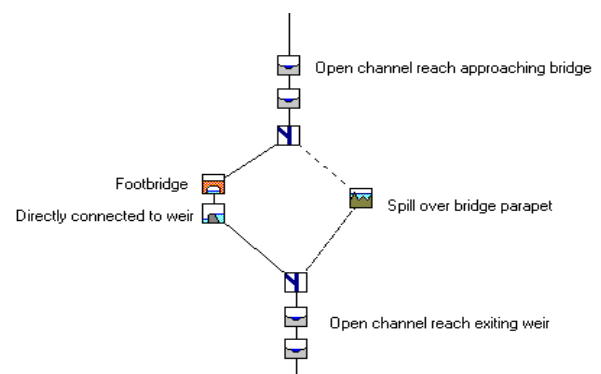
The surveyed structure dimensions were used to conceptualise bridges, culverts and weirs to simulate the hydraulic controls and flow paths that modify flood risk in the AFA. The conceptualisation sought to reduce complex structures to the simplest schematisation that accurately represented the hydraulic 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 instability at high flows as the reach is so much shorter than the other reaches in the 1D model and connection to the 2D model may cause recirculation of water. Therefore, the model is simplified to the configuration in Figure 4.1b which maintains the weir as the level control at low flows but avoids instabilities at high flow.

Figure 4.1: Simplification of Kanturk Footbridge and Weir



A: Kanturk Footbridge with Weir 2m downstream



B: Simplified Model Configuration

The simplification of structures in UoM22 is discussed in the following sections. There were no operable structures within the UoM22 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 UoM22:

- Using the USBPR approach where the bridge was a flat soffit highways bridge and the afflux was largely controlled by the flow around the piers and a spill over the deck to consider high flow routes.
- Using the HR Wallingford arched bridge approach where the bridge was arched and the afflux was largely controlled by the flow under the arch above springing point and a spill over the deck to consider high flow routes.
- Using a Bernoulli head loss unit based on the calculated head loss with the effects of piers, skew, eccentricity and other hydraulic losses. The loss coefficients (K values) were derived using the industry standard Hydraulics of Bridge Waterways².

The first two approaches were applied most widely in UoM22. However, the Bernoulli Loss approach was applied to several bridges through Dingle to improve stability when transitioning between open channel flow, bridge flow and orifice flow in this steep catchment. Orifice units were used to represent other bridges in Dingle where the opening was relatively small compared with the channel area and the bridge was in orifice flow in the 50%AEP event.

Photo 4.1: Barrack Lane Bridge, Castleisland



Captured: 14 Sept 2012

In UoM22, there are 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 where the pipe was deemed to be a significant obstruction and bridge coefficient adjusted to 1.5, assuming inefficient turbulent flow above this level. This is a conservative estimate of head loss for flood mapping purposes.

Culverts

Culverts were modelled in ISIS using; i) a culvert inlet to simulate losses associated with the constriction of flow at the entrance ii) an appropriate sized and shaped conduit unit and iii) a culvert outlet to simulate losses associated with the expansion of flow at the exit, or a weir unit to simulate the bed drop for culverts out-falling above the downstream river water level.

² US ACE (1978) Hydraulics of Bridge Waterways

Losses associated with trash screens have been considered as part of the inlet coefficients for both ISIS and ESTRY models. The trash screens have been assumed to be clear in accordance with the design scenario defined by OPW. However there are no trash screens in the modelled reaches for UoM22. Blockage of such structures will be considered separately as part of the option development process.

Weirs

Formal weir structures, such as those found at Flesk Bridge in Killarney at Castleisland gauge, have been modelled using formal round-nosed weir equations. Other informal weirs/natural bed drops over steep gradients, such as the rapid sections on the Flesk and Dingle Stream, have been modelled using online spill approaches. In both cases, the river sections have been extracted 20m upstream and downstream of the weir structure based on the surveyed weir long profile to adjust the bed levels and better represent the upstream and downstream open channel reaches. The surveyed weir crest was then used to inform the width and elevation in the formal round-nose weir structures and the spill elevations for informal structures. This approach ensures the weir or spill crest forms the hydraulic control and the localised scour pool effects are removed. Where the defined weir crest is narrower than the river channel width, online spills have been used to represent flow over the banks with calibrated coefficients to simulate the effects of bank vegetation.

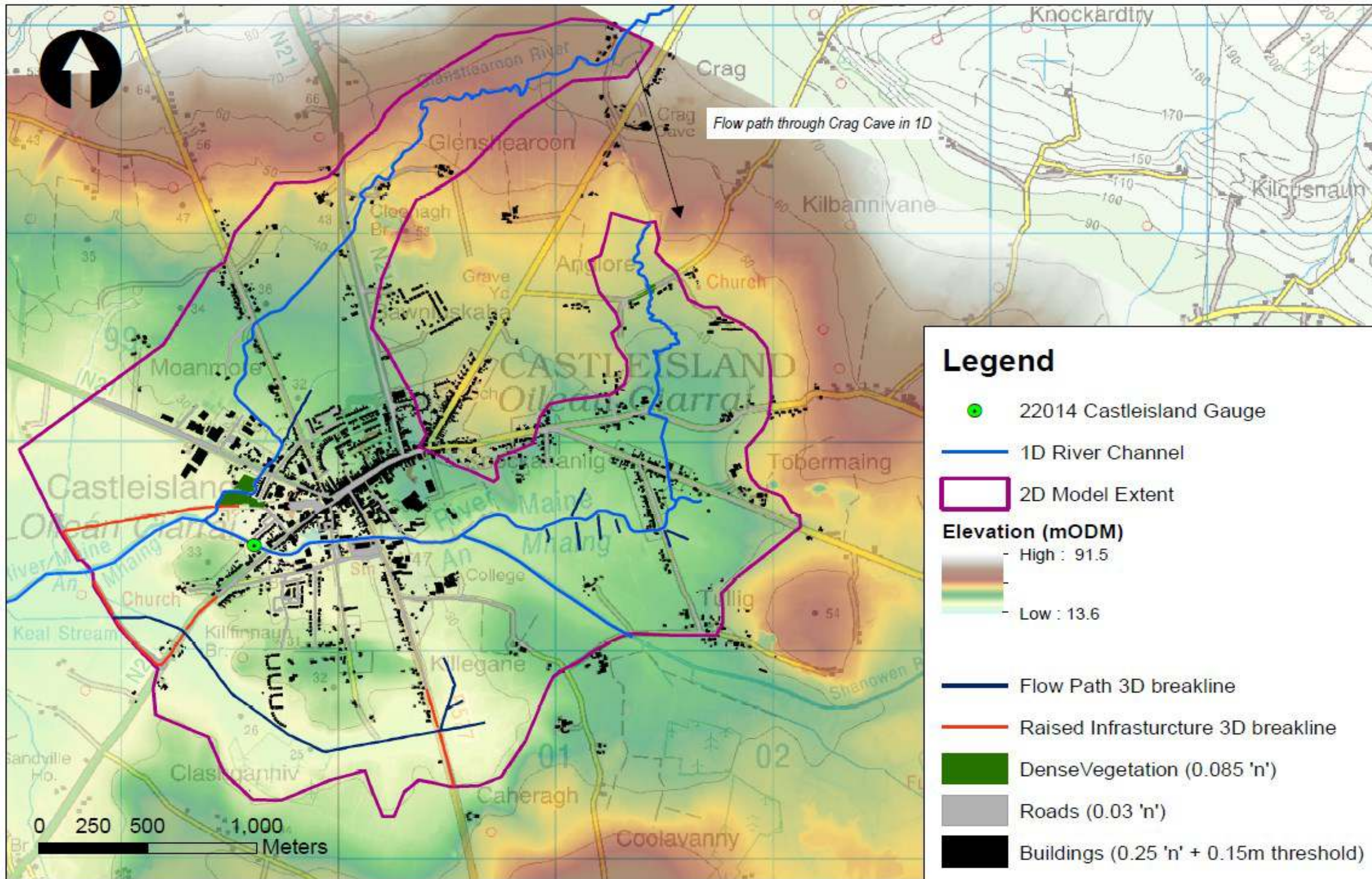
4.4 Floodplain

The floodplain in all the AFAs was represented by a regular 5m grid orientated to be perpendicular to the dominant flow path. A 5m grid cell size was selected to optimise the run time whilst adequately representing the complex urban nature of these AFAs. Map 4.4 presents an example for Castleisland.

Floodplain Topography

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

Map 4.4: Example Geoschematic of the Castleisland 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. A value of 0.2 has been selected as the upper limit of industry standard values for floodplains.

Syme (2008)³ 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 UoM22 are typically 6 to 16 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. Where the road is raised above the floodplain such as the N71 at Castleisland and N22 at Glenflesk, the road crest has been enforced in the 2D model domain based on LIDAR elevations and a lower Manning's 'n' applied as above.

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

³ 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

incorporated into the selection of the upper range of recommended floodplain Manning's 'n'. Table 4.3 summarises the design values selected. Sensitivity tests on Manning's 'n' values are discussed in Section 5.2.3.

Table 4.3: Floodplain Roughness Values

Surface	Manning's 'n' Roughness Value
Standing water	0.040
River Banks - Dense Vegetation	0.080 to 0.100
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 in both 1D Laune model, the 1D/2D Maine and Dingle models and the 2D only Portmagee model to dry conditions on the floodplain and stability of the models.

A 1D timestep interval of one second was applied to all the UoM22 models to ensure stability along the steep tributaries and to be divisible into the 2D timestep. A 2D timestep of two seconds was applied to all models to be divisible by the 1D timestep and within the recommended a half to a quarter of the 2D cell size.

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

5 Calibration and Sensitivity Analysis

5.1 Calibration

Table 5.1 outlines the historic flood events selected for the calibration of the hydraulic models during the hydrological analysis. The selection of historic events was based on scoring the flow estimates, observed data and reliable flood history as set out in Guidance Note 23⁴.

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
02/11/1980	Killarney	Fluvial	3	3	2	0	1	9	Largest flood on record at Flesk Bridge. Calibrate main channel to large event data. Smaller tributaries such as Woodford River should take note of uncertainties due to blockage.
04/10/2008	Castle-island	Fluvial	3	3	2	0	2	10	Calibrate main channel to large event data. Flows along Anglore stream should take note of uncertainties with % of Glanshearoon flow through the Crag Cave complex.
19/11/2009	Killarney	Fluvial	3	3	2	0	2	10	Calibrate main channel to large event data. Smaller tributaries such as Woodford River 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

In the absence of detailed historic flood evidence, there are a number of in-bank events which can be calibrated along the Maine and Laune catchments based on the gauged data only. These include:

- 12th January 2010 – River Maine Catchment
- 1st February 2002 – River Maine Catchment
- 4th January 2008 – River Maine Catchment
- 26th October 2008 – River Laune Catchment

⁴ Jacobs, (January 2013) Guidance Note 23 Model Calibration. Version 1.

Reports of recurring flooding and information from local engineers were also used to verify the modelled outlines such that there is “reasonable” representation of the historical flood frequency in Milltown, Glenflesk, Laune and Dingle models.

It was not practical to calibrate the Laune model as there was no flood extent or level for specific events beyond the gauge at the upstream limit to calibrate the hydraulic parameters. Additionally, there were no reports of flooding in Portmagee or gauge data to enable model calibration for this AFA.

Sensitivity analysis has been used to further assess hydraulic parameters where there was insufficient data to fully calibrate the hydraulic model, discussed in Section 5.2.

5.1.1 2nd November 1980 Killarney

The November 1980 event was the largest flow on record at Flesk Bridge. The River Flesk levels spilled out-of-bank to flood Killarney National Park as well as fields and recreational grounds adjacent to the river in Killarney. The River Deenagh was also reported as flooding but no precise locations were provided.

The quality of the historic flood data has been reviewed based on the local engineers' comment for the 3rd November 1980 event:

- Photographs
 - Photographs were available however it was difficult to reconcile the recorded location with the view shown in the photograph due to development since 1980 along Kenmare Road.
 - Therefore the description from the local engineer has been used to validate the flood extent.
- Levels and flows
 - Peak water levels were recorded at the Flesk Bridge gauge and are deemed to be reliable. However the floodplain flows may be underestimated based as the rating is based on in-bank gaugings.
 - Gauge records at White Bridge were not available from this period.
- Extent
 - No flood outline was produced as part of the report.
 - Areas flooded are based on the description contained within the local engineer's report.

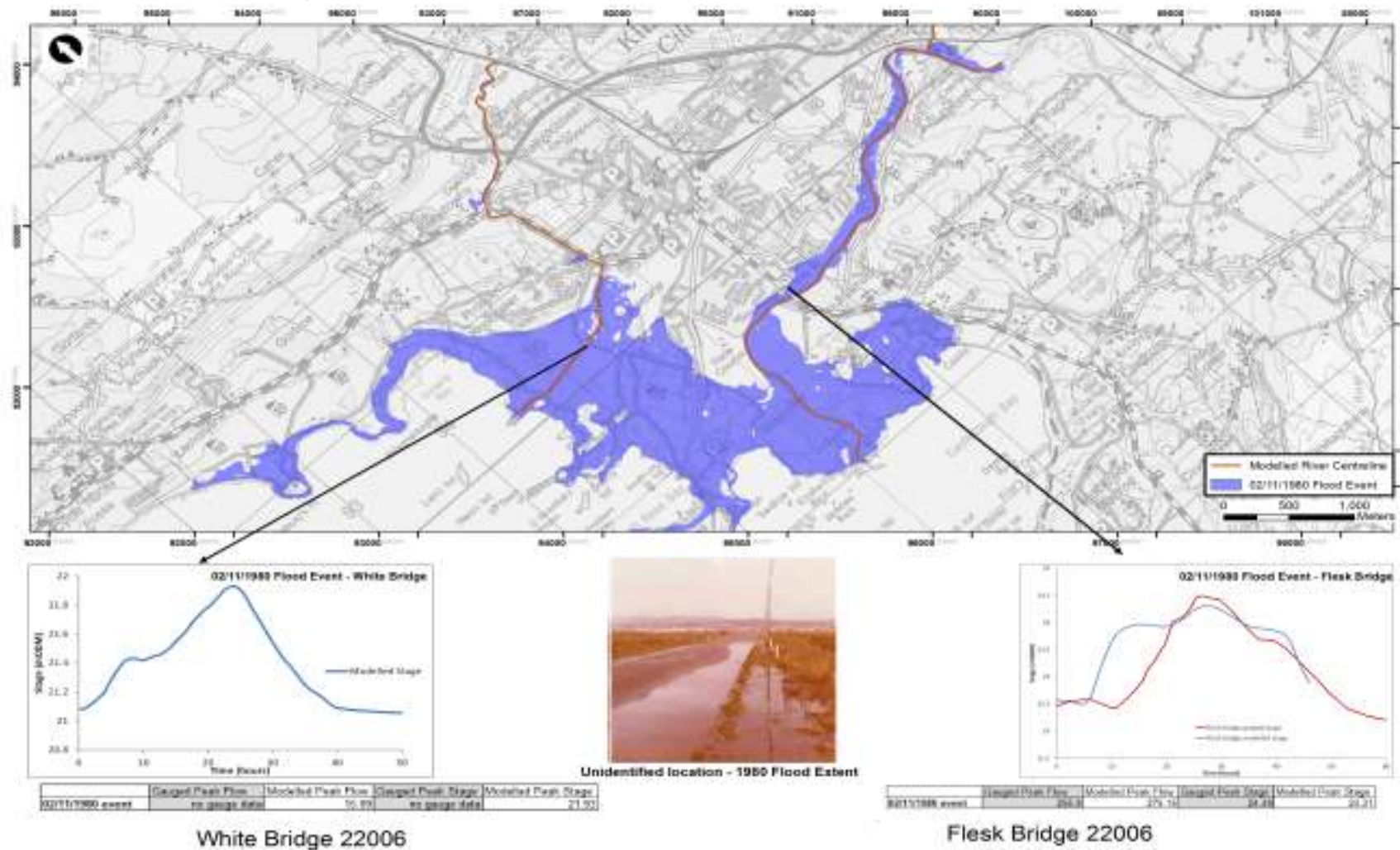
The design hydraulic model was modified as follows to represent the hydrological and hydraulic conditions of this event:

- The rainfall profile was transferred from Valentia Observatory, and hydrographs produced using the FSSR16 rainfall-runoff approach with percentage runoff increased to over 90% to meet the observed flows at Flesk Bridge.
- This corresponds with saturated conditions in the Met Eireann records.
- The downstream water level in Lough Leane was set based on the design gradient, because level records on the Lough were not available for this event.
- No other topographic conditions were changed to reflect the 1980 event.

The Manning's 'n' values were adjusted from 0.040 to 0.045 in order to best match the flood levels and extents in Killarney. The weir coefficients, spill coefficients and Bernoulli Loss at White Bridge were also iteratively adjusted to the values quoted in the appendices in order to replicate the mechanisms of flooding reported.

Map 5.1 compares the resultant model flood extent and levels with the recorded information at the gauges. White Bridge was not operational in 1980, therefore the Deenagh was not calibrated for this event. The FSSR hydrograph was calibrated by adjusting the percentage runoff and phasing as part of the previous hydrology report. This achieved a flow hydrograph within +7% of the peak flow at Flesk Bridge gauge. The flood level is 0.17m lower than observed principally because the rating curve at Flesk Bridge is based on in-bank gaugings. Extrapolation of this rating curve can lead to underestimation of floodplain flows and therefore the discrepancy in level. However, the flood level is still within the required accuracy of 0.2m for HPWs.

Map 5.1: Calibration of Killarney Model to 2nd November 1980 Event



5.1.2 4th October 2008 Castleisland

Initially, river levels in the Glenshearoon River overtopped the left bank on 4th October 2008 whereupon the flood water entered the Crag Cave complex to flood areas downstream on the Anglore Stream. Several properties were flooded along Anglore Stream at Cordal Road. An additional commercial property was also affected by the flooding.

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

- Photographs
 - The photographs were taken of the Glenshearoon spilling out of bank, but were not available at flooded property locations.
- Flood Levels
 - Water levels were recorded at the Castleisland gauge on the River Maine in the AFA and are deemed to be reliable.
 - Depth of flooding at the affected properties was not recorded at the time, so levels on the floodplain could not be calibrated.
- Extent
 - No flood outline was produced as part of the report.
 - Areas flooded are based on the annotated points on the flood report map and description contained within.

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 60% to meet the observed flows at Castleisland gauge. These calibrated rainfall-runoff parameters were then transferred to the model inflows.
- The design downstream QH relationship was retained for this event.
- No other hydraulic modifications were made.

Map 5.2 compares the resultant model extent and levels with the recorded information. The model was calibrated by adjusting Manning's 'n' to 0.048 and the spill coefficient of the spill at the swallow hole to 1.0 in order to reproduce the recorded flow route through Crag Cave and extent of flooding on the Anglore Stream. The resultant flood extent matches well with the reported flooding at Glebe House Road and the property flooding at Tullig as the excess flows cause the Anglore to exceed the capacity of Glebe House Bridge and the Tullig culvert.

The modelled water level at the gauge was 0.05m higher than recorded however it is within the CFRAM framework calibration tolerance of +/- 0.1m and reproduces the duration reasonably, given that the hydrological calibration underestimated duration on the falling limb.

⁵ Kerry County Council (2008) Flooding at Tullig, Castleisland Co. Kerry on 04th October 2008.

Map 5.2: Calibration of the Castleisland Model for 04 October 2008 Event

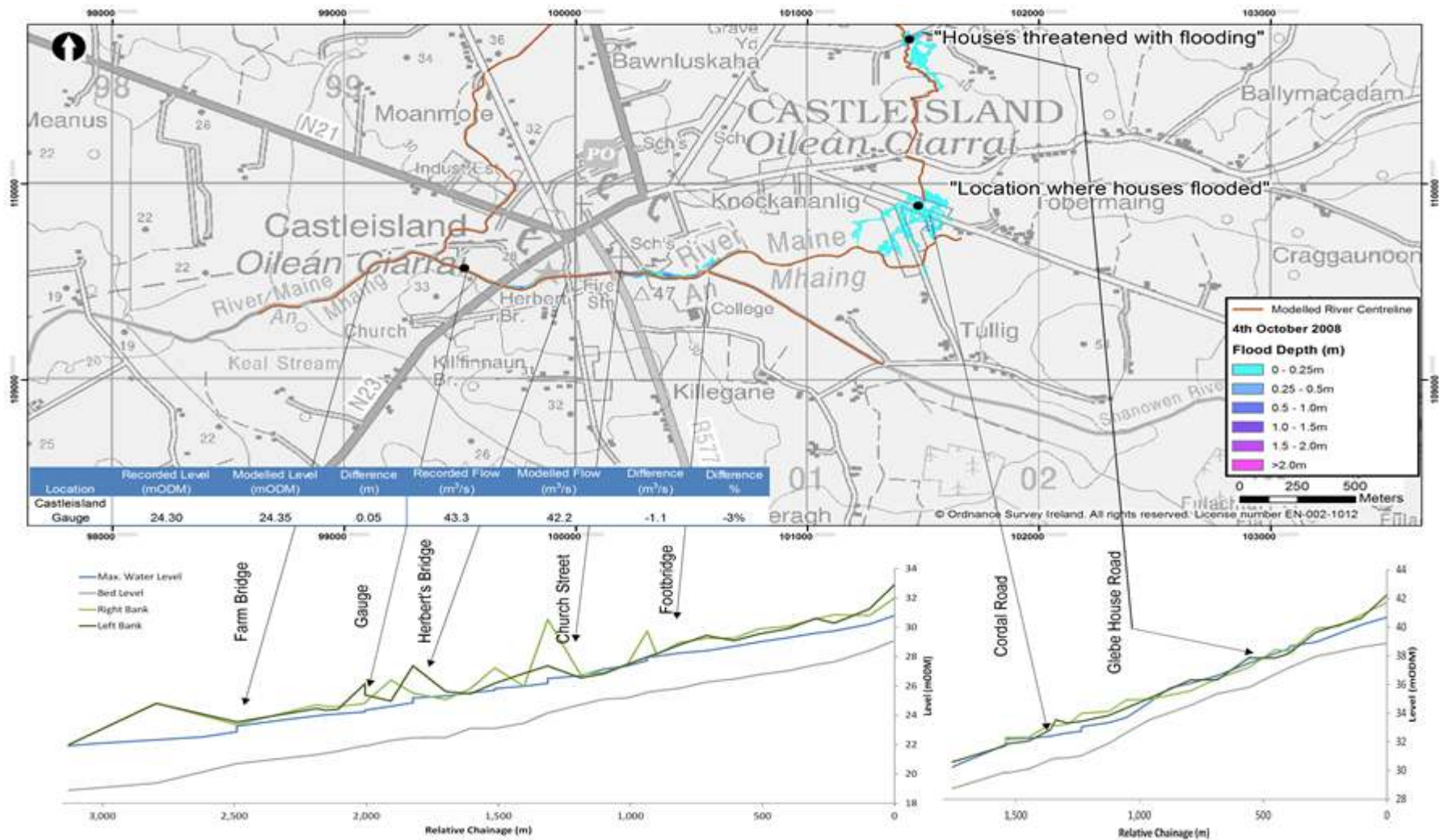


Figure 5.1: Performance of Flow at Castleisland Gauge

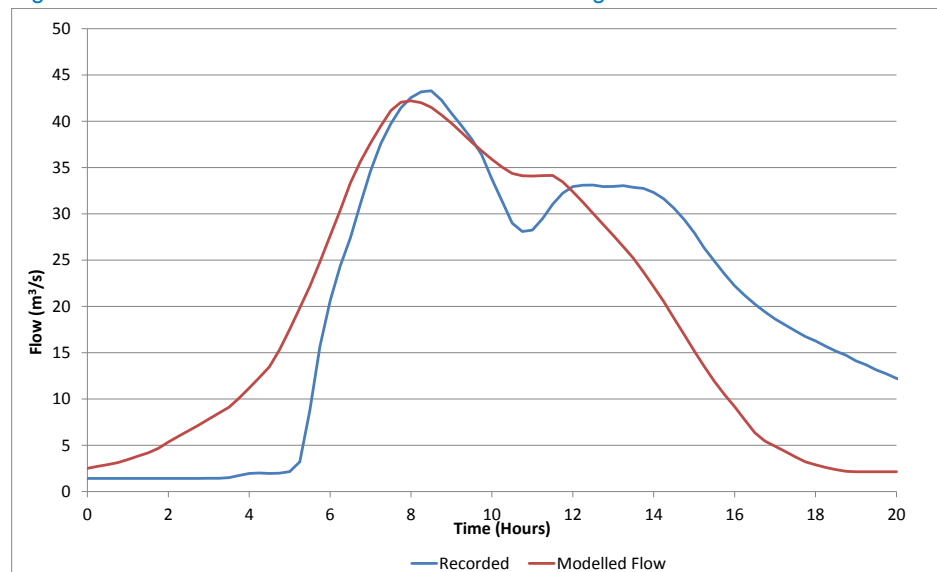
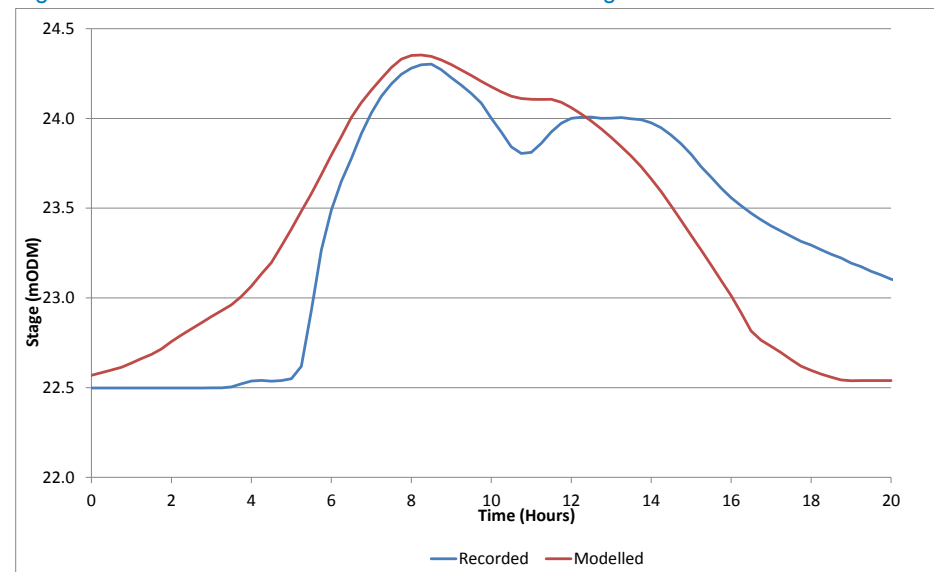


Figure 5.2: Performance of Level at Castleisland Gauge



5.1.3 19th November 2009, Killarney

Lough Leane levels were already high prior to the event due to prolonged rainfall over the preceding month and saturated catchment conditions. The intense rainfall on the 19th November 2011 further raised levels which caused significant flooding to the Killarney National Park area, parts of the N70 and the local road network. The tourist area around Muckross and the Lake Hotel were extensively flooded. This was the first recorded flooding of the Lake Hotel in the past 190 years.

The quality of the historic flood data has been reviewed from Kerry County Council and correspondence with the Lake Hotel staff:

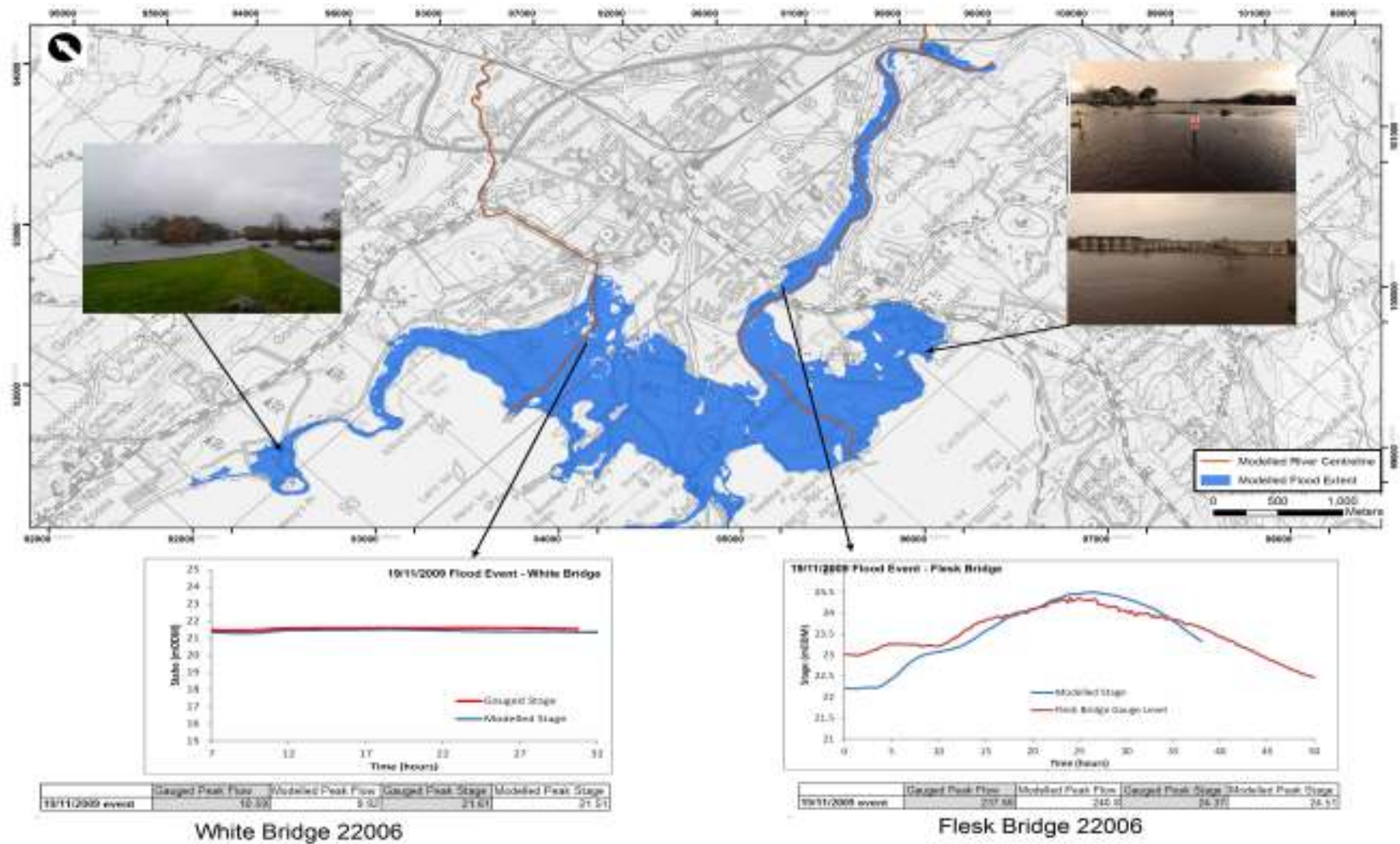
- Photographs
 - The photographs have been collected from Kerry County Council and the Lake Hotel staff.
 - The photographs are taken on 19th November 2009 during the flood, but the time is not known.
 - Therefore these photographs are deemed representative of locations flooded but do not necessarily represent the peak.
- Flood Levels and Depths
 - Flood level was recorded at Flesk Bridge and White Bridge gauges.
 - Depth of flooding was not recorded, however anecdotal reports during the flood risk review suggest flooding at the Lake Hotel was over 0.5m deep.
- Extent
 - No recorded extent was available,
 - The areas flooded have been verified by the photos and correspondence with the Lake Hotel (annotations provided on the following map).

The design hydraulic model for Killarney 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 58% to meet the observed flows at Flesk Bridge.
- The downstream water level in Lough Leane was set based on level gauge records at Tomies Pier.
- No other topographic conditions were changed to reflect the 2009 event.

The hydraulic parameters were adjusted to best match the flood levels and extents in Killarney including Manning's 'n' values as discussed in Section 5.1.1. Map 5.2 compares the resultant model extent and levels with the recorded information. The calibrated model results match well with the recorded flooding at the lakeside hotels and extensive flooding of Killarney National Park. The gauge flow was within 1% of the peak flow and 0.1m of the peak level at Flesk Bridge. The peak flow at White Bridge was 7% greater because the rating does not account for bypass flow once out of bank. However the level was within 0.1m. Therefore the model is deemed to provide a reasonable representation of the November 2009 event.

Map 5.3: Calibration of the Killarney Model for 19th November 2009



5.1.4 In Bank Calibration

In-bank calibration was undertaken on the River Maine for two additional fluvial events where there was data available at multiple gauges:

- 4th January 2008: Data available at Castleisland gauge and Riverville gauge
- 12th January 2010: Data available at Riverville gauge and Castlemaine (tidal) gauge

For the 4th January 2008 event the Castleisland gauged hydrograph formed the inflow to the Maine and the hydraulic model calibrated to achieve the gauged hydrograph at the Riverville gauge.

For the 12th January 2010 event, the observed rainfall at Valentia was transferred to Riverville gauge to form the input to the rainfall-runoff hydrograph. The rainfall-runoff hydrograph was calibrated to the gauged flow by adjusting percentage runoff to 30%. The observed rainfall at Valentia was then transferred to the various HEPs based on the daily rainfall gauge totals and the flood hydrograph derived using the FSSR rainfall runoff approach with the calibrated 30% runoff.

The astronomic predicted tide was derived for Castlemaine Harbour and applied directly to the downstream of the Maine model for all scenarios. The surge residual at Castletown Bearhaven was less than 0.1m in all events therefore surge was not considered.

The model was calibrated by adjusting the Manning's 'n' in-bank and the weir coefficients at Riverville gauge to reproduce the water level and flow at Riverville gauge and the level at the tidally influenced Castlemaine gauge. Figures 5.3 and 5.4 summarise the performance at the Riverville gauge for this event. The hydrological routing of flow was within 1% and the hydraulic model was within 0.02m of the peak at Riverville. The 15 minute level record for Castlemaine was not readily available for this event.

In-bank calibration was also undertaken on the River Maine catchment for the 4th October 2008 where there was concurrent gauge information available at Castleisland, Riverville and Castlemaine. The astronomic predicted tide was derived for Castlemaine Harbour and applied directly to the downstream of the Laune model. The surge residual at the nearby Castletown Bearhaven tidal gauge was less than 0.1m in all events therefore surge was not considered.

The model was calibrated by adjusting the Manning's 'n' in-bank and the weir coefficients at the rapid sections to reproduce the water level gauge and the level at the tidally influence Castlemaine gauge. Figures 5.5 to 5.7 summarise the performance at the gauge locations for the 4th October 2008 event. The hydrological routing of flow was within 1% at Riverville and the hydraulic model was within 0.01m of the peak level at Riverville and 0.03m of peak level at Castlemaine.

Therefore, the in-bank model of the Maine is deemed representative of the gauge data for fluvial flood events.

Figure 5.3: Calibration of Flow at Riverville – 4th October 2008

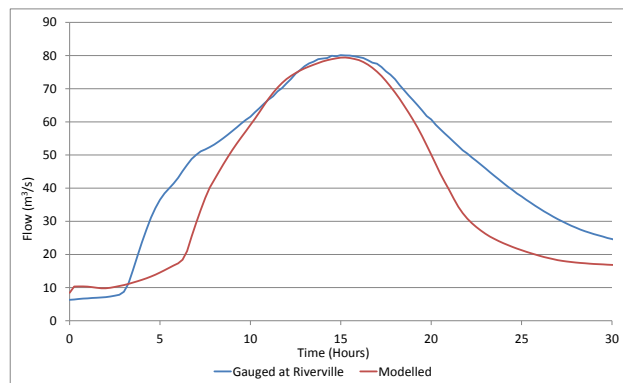


Figure 5.4: Calibration of Level at Riverville – 4th October 2008

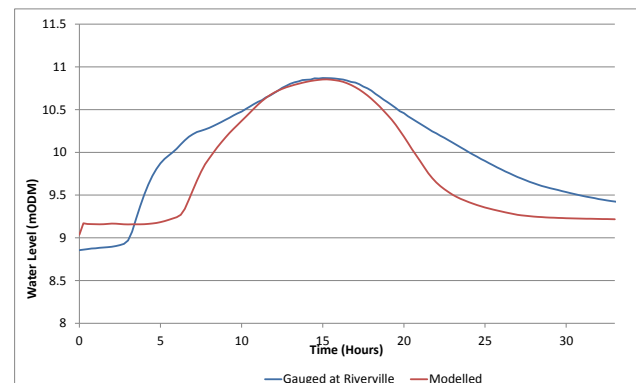


Figure 5.5: Calibration of Flow at Riverville – 12th January 2010

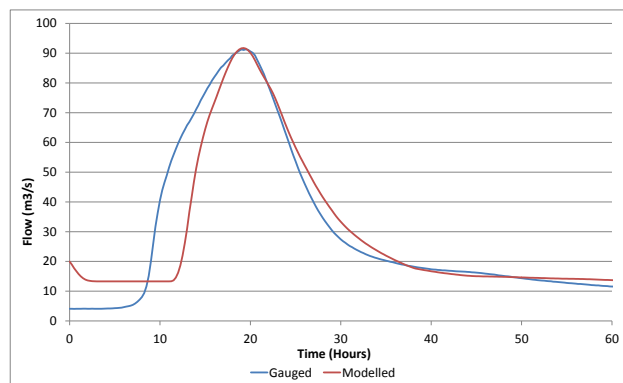


Figure 5.6: Calibration of Level at Riverville – 12th January 2010

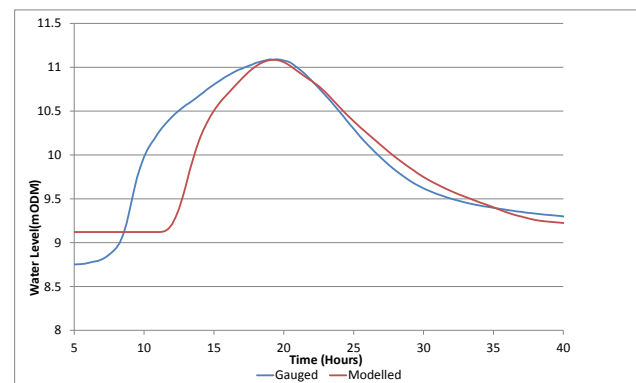
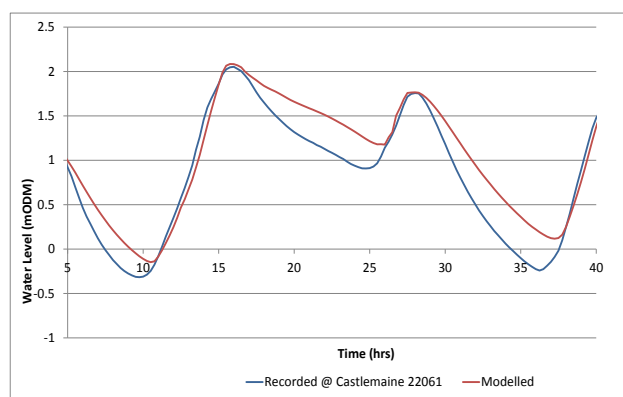


Figure 5.7: Calibration of Level at Castlemaine – 12th January 2010



5.1.5 Validation to Historic Flood Information

There was insufficient historic flood evidence and/or gauge data to fully calibrate flood levels and extents in Milltown, Glenflesk and Dingle AFA. Therefore, reports of recurring flooding and information from local engineers were compared with the modelled outlines to ensure that there is “reasonable” representation of the historical flood frequency.

In Milltown, the overtopping along Old Station Road in the 10%AEP and larger events matched well with previous reports of flooding in 2008 and recurring flooding from local residents during the Flood Risk Review (Map 5.4). In Glenflesk, the 50%AEP reaches the road level of the N22 upstream of Glenflesk which corresponds with the photographs and reports of annual flooding provided by Kerry County Council (Map 5.5). In Dingle, the recurring flooding on the road near The Woods corresponds well with the 20%AEP and larger coastal events (Map 5.6). The 10%AEP fluvial flood extent and larger magnitude events also correspond with recurring flooding along the Mall and overtopping at Herbert’s Bridge.

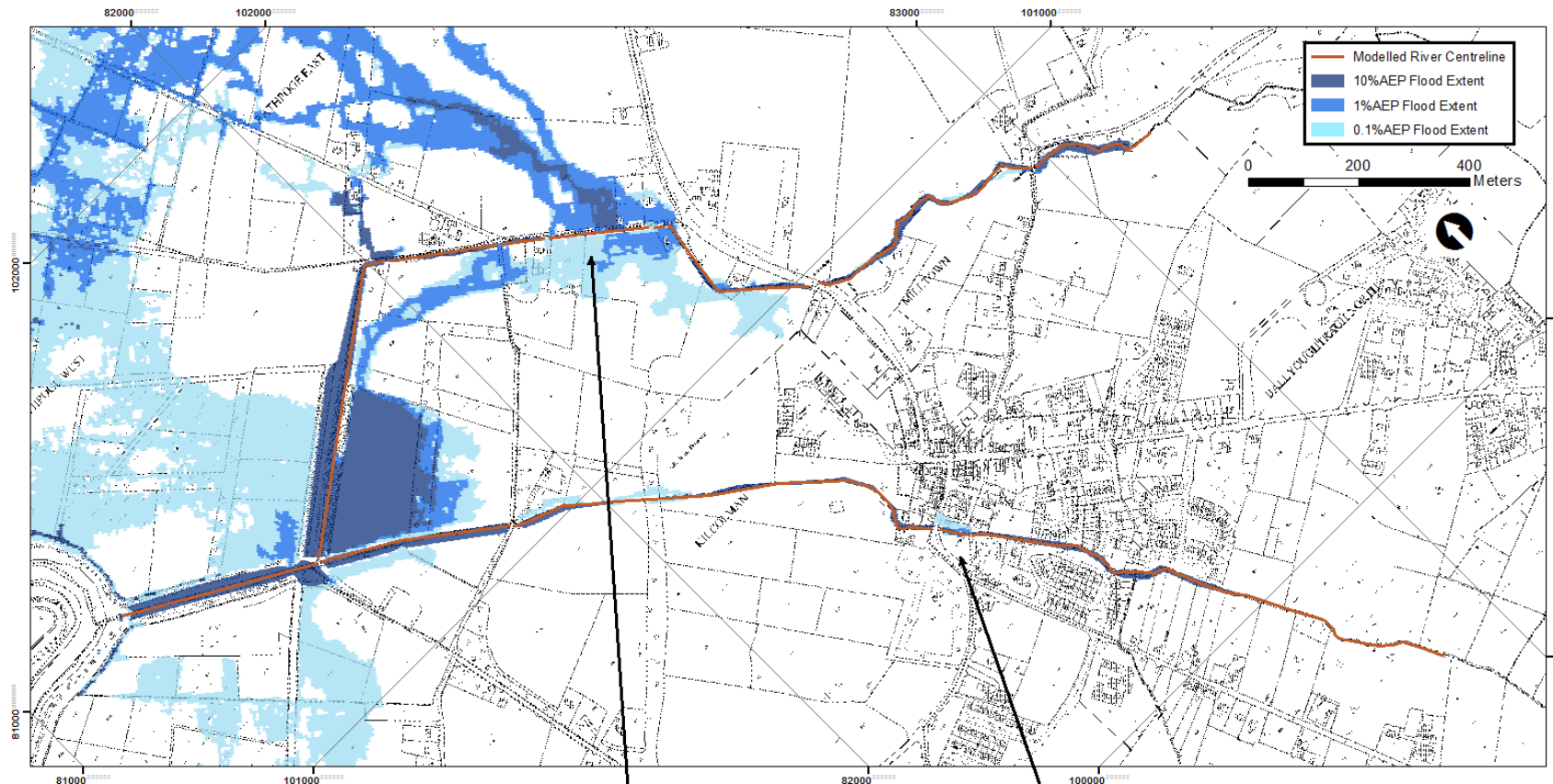
Flooding has been observed in Castleisland on 24th January 2014, since the completion of the hydrological analysis and agreement of the calibration events. Photographs have been used to validate the areas that are vulnerable to flooding as a common sense check (Map 5.7). The 5%AEP to 2%AEP modelled flood extent correspond well with photographs of overtopping along the Glenshearoon into the Crag Cave, flooding at Glebe House Bridge, Tullig and around Church Street.

The prolonged period of rainfall also resulted in surface water flooding/ponding of agricultural lands/public roads in the townlands of Meanus/Camp, situated to the northwest of the town centre. However, the CFRAMS model does not consider surface water flooding therefore the modelled outlines do not indicate flooding at this location.

Unfortunately the gauge data was not available in the AFA at the Castleisland gauge due to a malfunction. Local engineers observed a similar flooding at Glenshearoon 5 years ago in the October 2008 event. However the flooding of Church Street and Tuillig Road (off Cordal Road) was observed to be the first flooding in 20 years by residents. Therefore the modelled flood frequency is deemed to be broadly representative of the historical flood frequency at Tullig and Church Street.

There were no reports of flooding in Portmagee or gauge data to enable validation to historic flooding for this AFA.

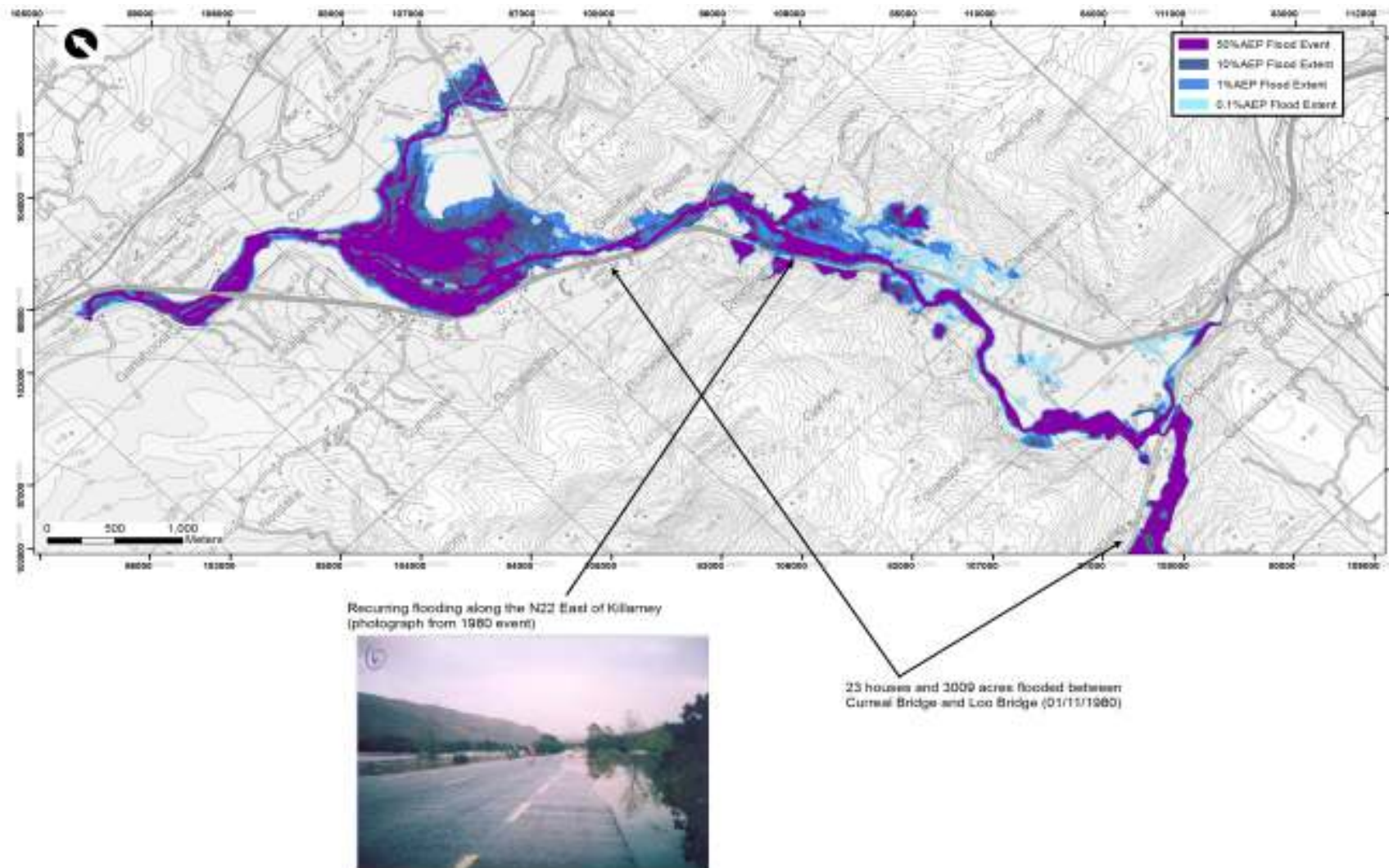
Map 5.4: Validation of Modelled Outlines to Historic Flood Evidence in Milltown AFA



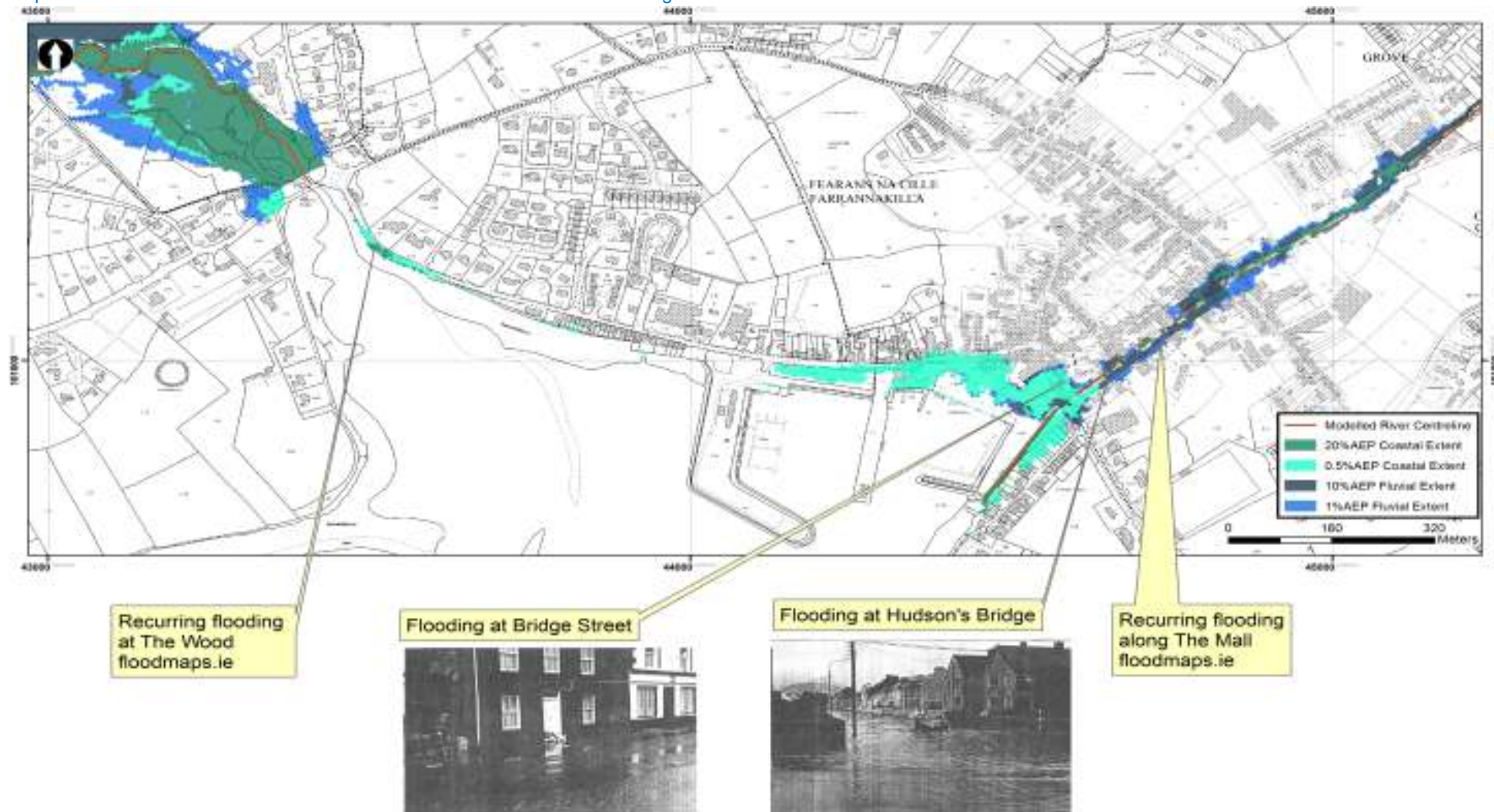
Flooding reported at these properties between 1998-2000

Flooding reported both in JBA report (2011) and anecdotally along this road in January 2008.

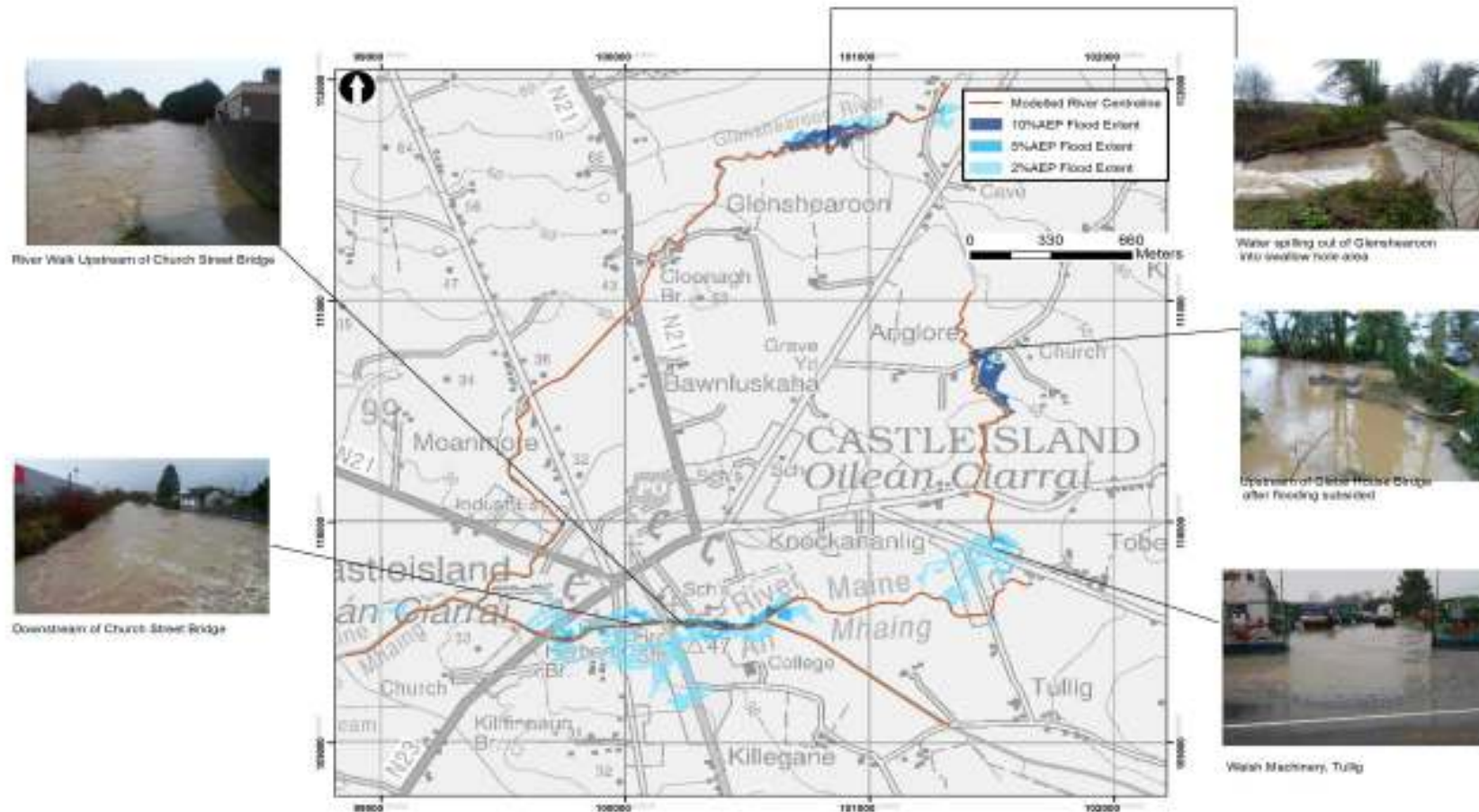
Map 5.5: Validation of Modelled Outlines to Historic Flood Evidence in Glenflesk AFA



Map 5.6: Validation of Modelled Outlines to Historic Flood Evidence in Dingle AFA



Map 5.7: Validation of Modelled Outlines to January 2014 Flood Evidence in Castleisland AFA



5.1.6 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 $\pm 0.1\text{m}$ 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
02/11/1980	$\pm 0.1\text{m}$ (Gauged)	Killarney	0.17	0.17	0.17
04/10/2008	$\pm 0.1\text{m}$ (Gauged)	Castleisland	0.05	0.05	0.05
19/11/2009	$\pm 0.1\text{m}$ (Gauged)	Killarney	0.10	0.10	0.10
In- calibration:					
04/10/2008	$\pm 0.1\text{m}$ (Gauged)	Maine MPW	0.02	0.02	0.02
12/01/2010	$\pm 0.1\text{m}$ (Gauged)	Maine MPW	0.03	0.02	0.02

The Castleisland model matched well with the gauged and flood report information for the 4th October 2008 event. The design model outlines were also validated with locations which are known to flood during the recent January 2014 event.

The Killarney model tended to overestimate water level by 0.1 to 0.2m following calibration of the weir coefficient downstream. However the flood extent matched well with recorded flooding in both events.

The in-bank calibration on the Maine indicated good performance of the model at Riverville and Castlemaine. However, concurrent gauge information for flood events was limited to calibrate the model further.

The Milltown, Glenflesk and Dingle models all represent the recorded historical flood frequency based on recurring flood reports and local engineer's comments.

The calibrated hydraulic parameters have been used to simulate the design scenarios discussed in Chapter 6. The calibrated hydrological parameters were not applied to the design scenarios as a rainfall-runoff approach was not used to generate the design inflow hydrographs.

5.2 Sensitivity Analysis

5.2.1 Flow

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

Flood level and extent in Castleisland, Killarney and the Ashullish Stream in Milltown AFA were sensitive to assumptions in peak flow. The largest increase in flood extent due to the uncertainty in flows was in Milltown whereby large areas of the flat Abbeylands become flooded. However, the increase in flood extent did not significantly increase flood risk to properties within the AFA.

In Castleisland and Killarney, the increased flows exceeded the capacity at key bridges, thereby increasing flood risk to properties nearby. The lower Maine (downstream of Traillia River) was also sensitive to assumptions in peak flow because there is limited capacity between the raised embankments. However, the increase in flood extent did not affect properties.

For the Flesk MPW upstream of Glenflesk, the increase in flows raised flood levels significantly but did not significantly increase flood extent because the narrow floodplain was already inundated in the 1%AEP design. However, the increased flows resulting in more extensive overtopping of the N22. Therefore, the Flesk upstream of Glenflesk is considered sensitive to flow.

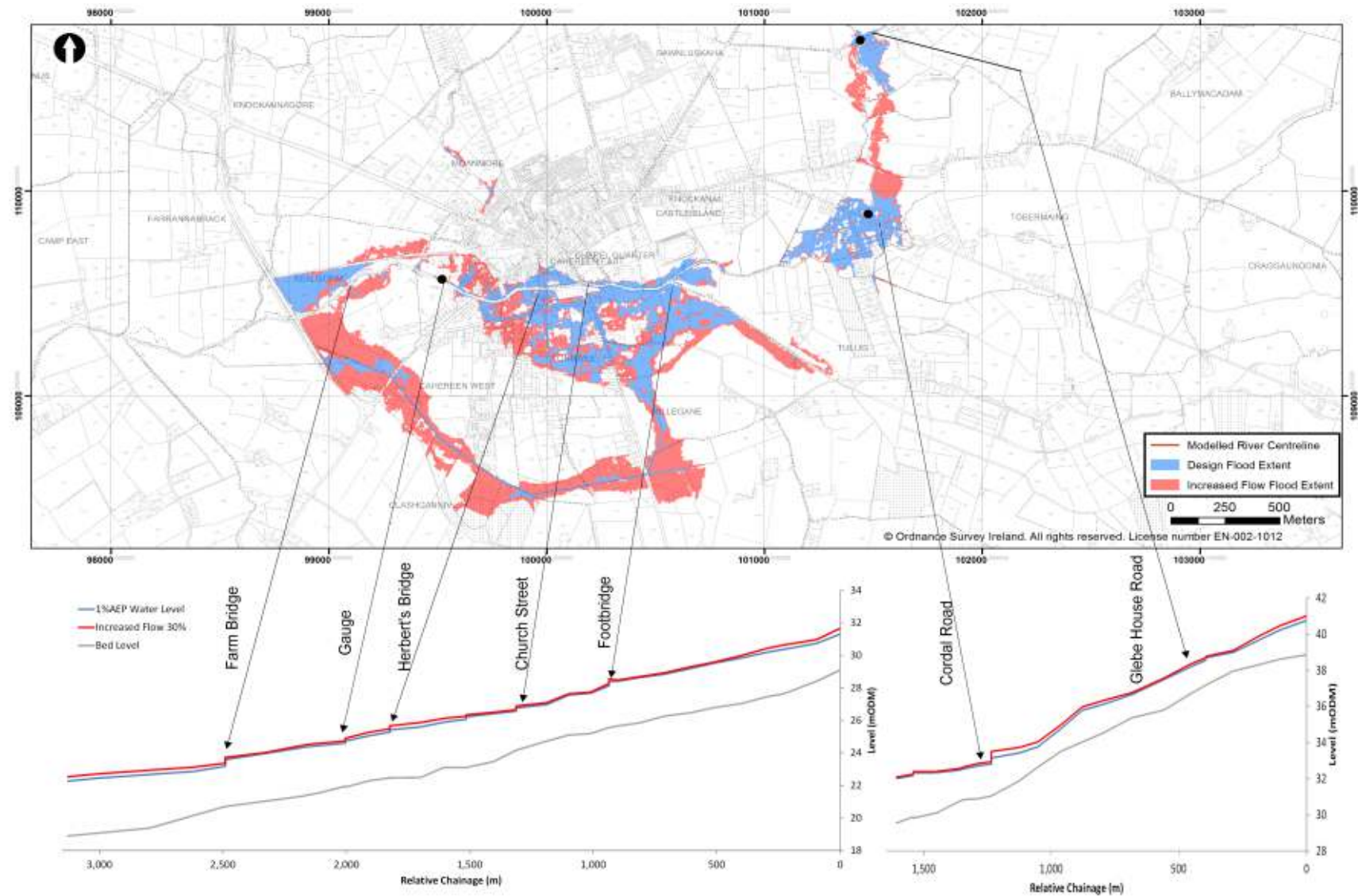
The Killarney and Dingle AFAs and the Flesk and Laune MPWs were less sensitive to the assumptions in peak flow, as their narrow floodplains are already inundated in the design 1%AEP fluvial scenario. Therefore, the increase in flow does not significantly increase areas at flood risk, although depth of flooding and risk to life increases slightly with the more extreme conditions.

While specific sensitivity tests were not carried out in respect of storm duration the impact of the increased volume has been investigated through the analysis of increased peak flow which simulates a similar increase in flood volume. The impacts of increased storm duration would be similar to the increase in peak flow.

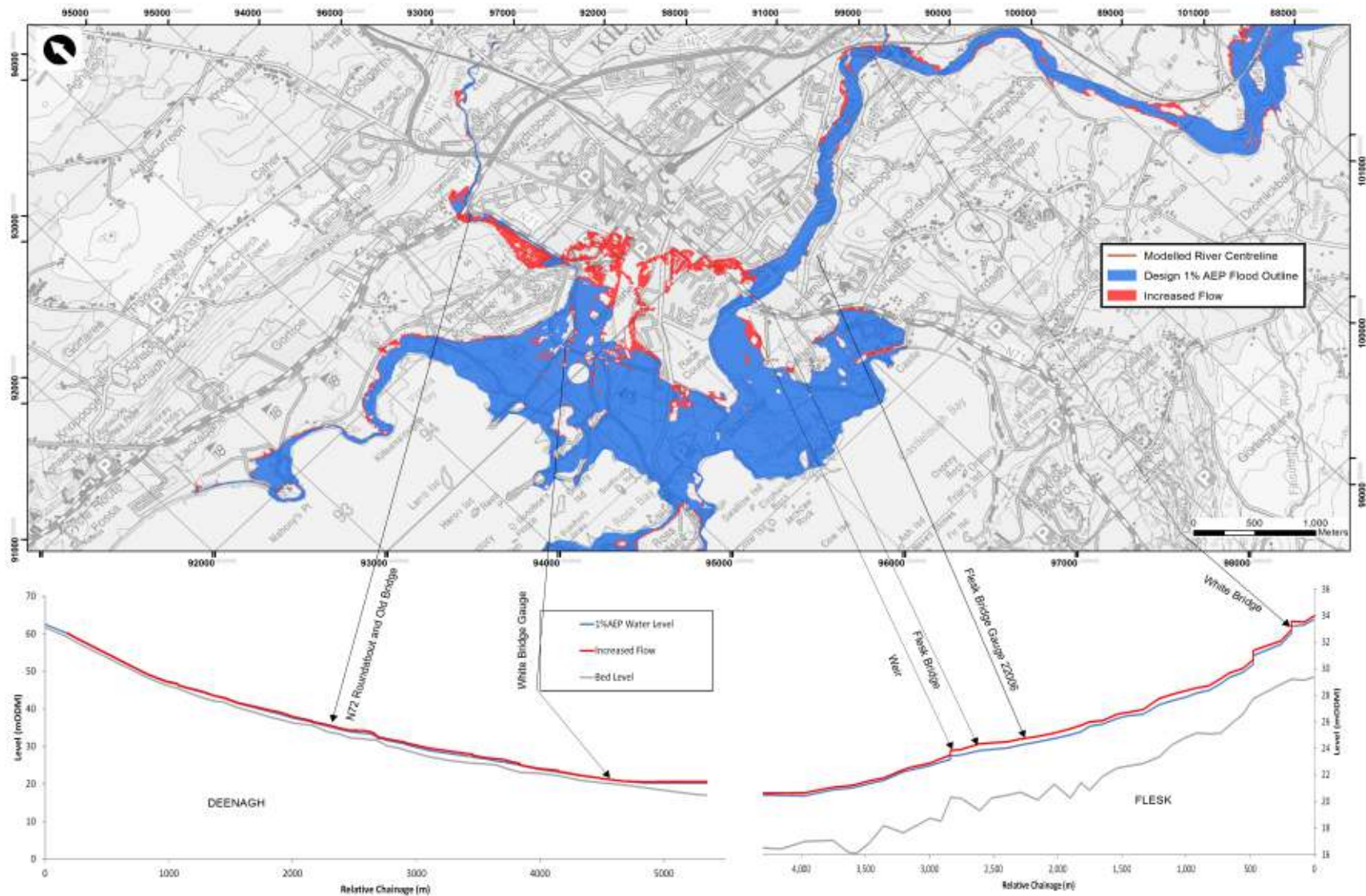
Sensitivity to flow was not assessed at Portmagee as the AFA is only deemed to be at coastal risk based on the Flood Risk Review.

Maps 5.8 to 5.10 show the sensitivity plots for most sensitive reaches. The plots for all flow sensitivity tests can be found in the model performance proformas in the relevant Appendices

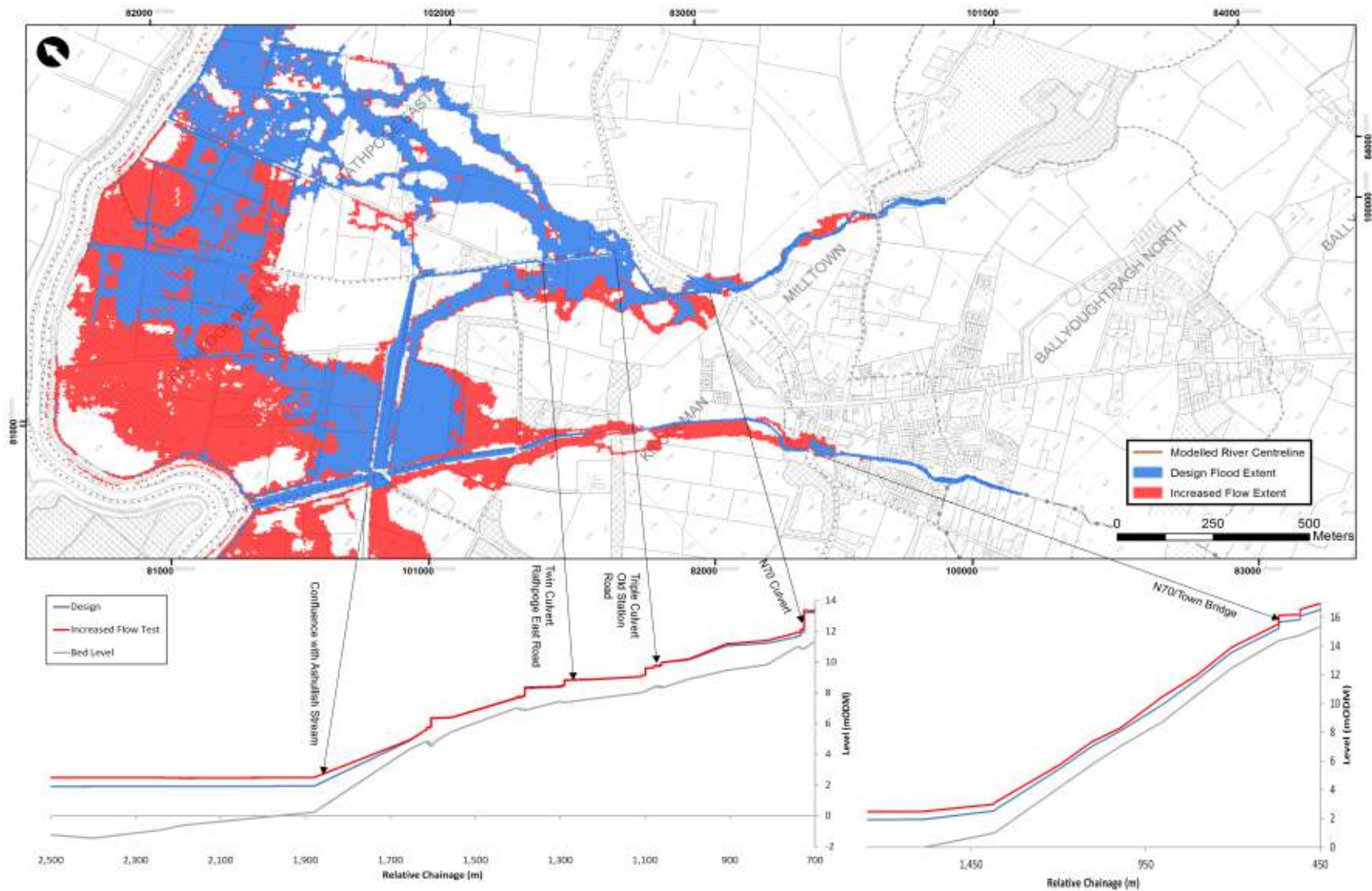
Map 5.8: Sensitivity to Peak Flow – Castleisland



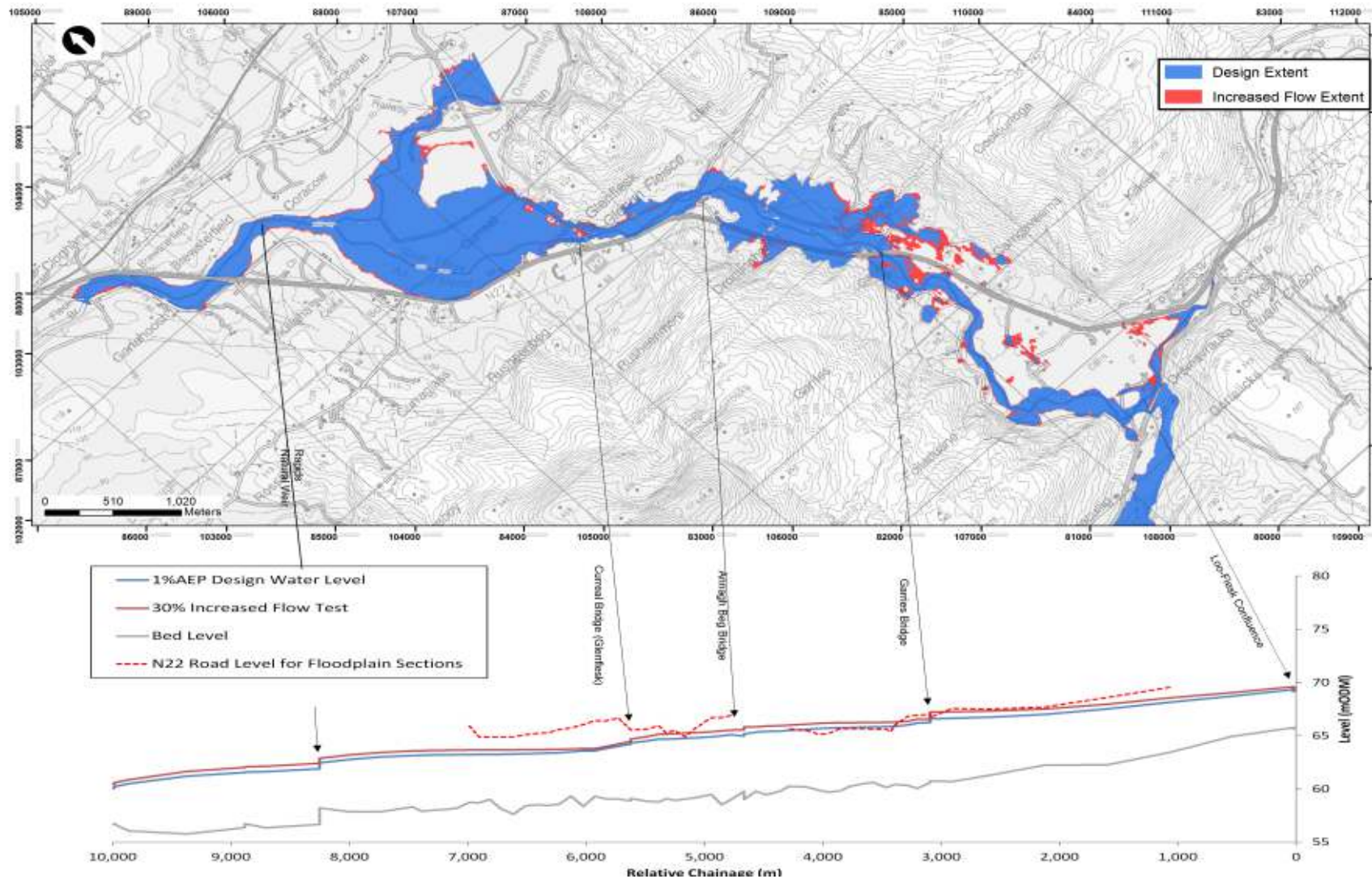
Map 5.9: Sensitivity to Peak Flow-Killarney



Map 5.10: Sensitivity to Peak Flows - Milltown



Map 5.11: Sensitivity to Peak Flow - Glenflesk



5.2.2 Level

A sensitivity test was undertaken on downstream water level for tidally-affected reaches in UoM22 (i.e. Maine MPW, Laune MPW, Dingle AFA and Portmagee AFA). 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 UoM22, flood level and extent was sensitive to the downstream coastal level in Dingle AFA and Maine MPW. The increase in water level results in more extensive coastal flooding along Dingle Quay affecting properties. On the lower Maine, the flooded area increased significantly as the increased level exceeds more of the raised embankment, but it did not significantly increase the number of properties affected.

Portmagee AFA and the Laune MPW were less sensitive to downstream coastal level. In Portmagee the land rises inland, therefore the increase in water level does not significantly increase flood risk within the AFA. Along the Laune, the narrow floodplain was already inundated in the design 1%AEP tidal scenario. Therefore, the increase in downstream 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.

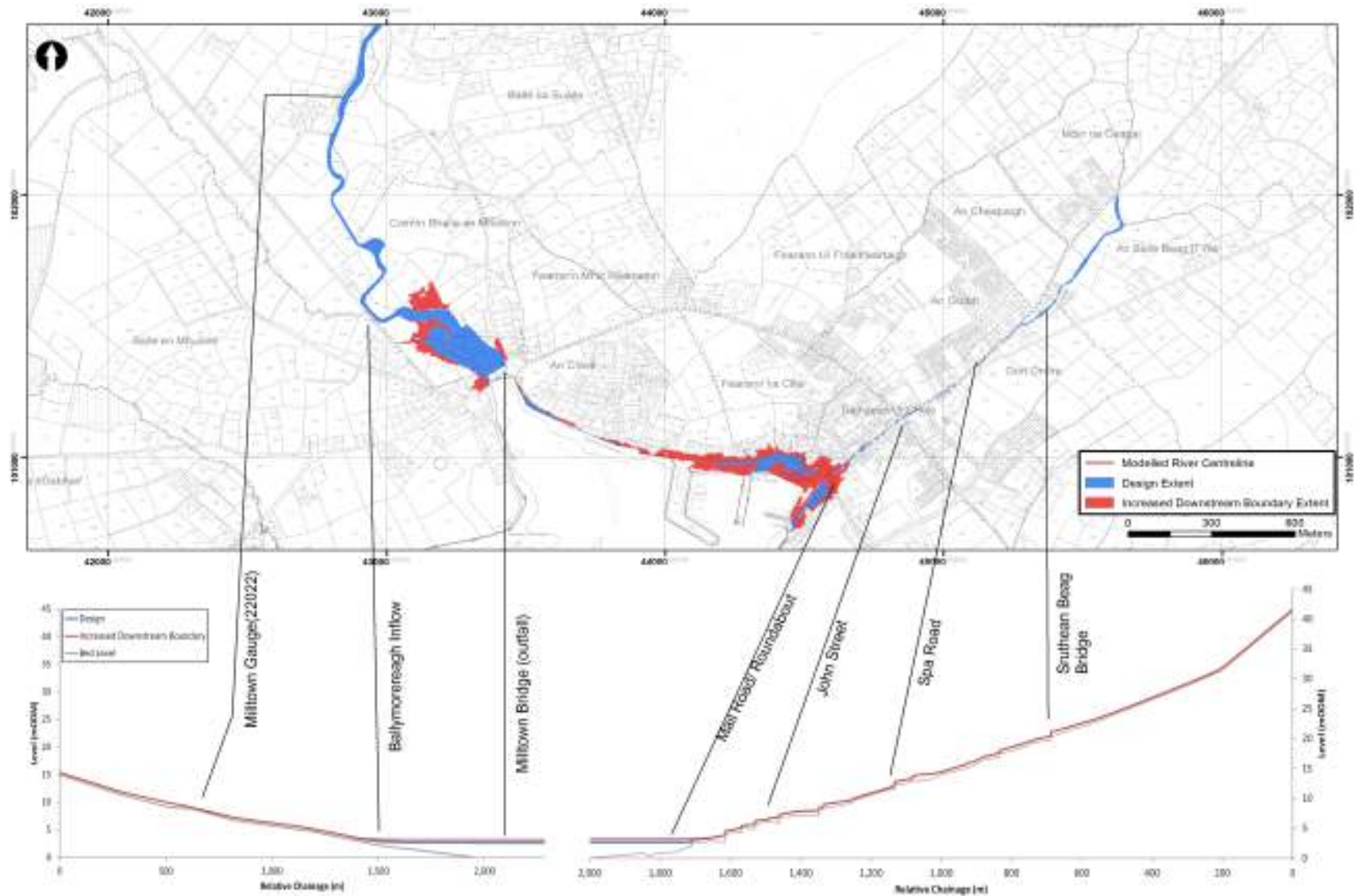
Maps 5.12 and 5.13 show the sensitivity plots for the most sensitive reaches. The plots for all level sensitivity tests can be found in the model performance proformas in the relevant Appendices.

Large catchments such as the Maine and Laune been split up into several separate models to ensure accuracy within the upstream AFAs of Glenflesk, Killarney and Castleisland. Therefore, the downstream boundaries of these fluvial models in the upper catchment are defined by a QH relationship representing the fluvial MPW reach downstream. The gradient in the QH boundary was reduced based on the flattest estimate of the floodplain gradient in the MPW reach downstream to investigate the impact of increased backwater upstream and interaction with the downstream MPW.

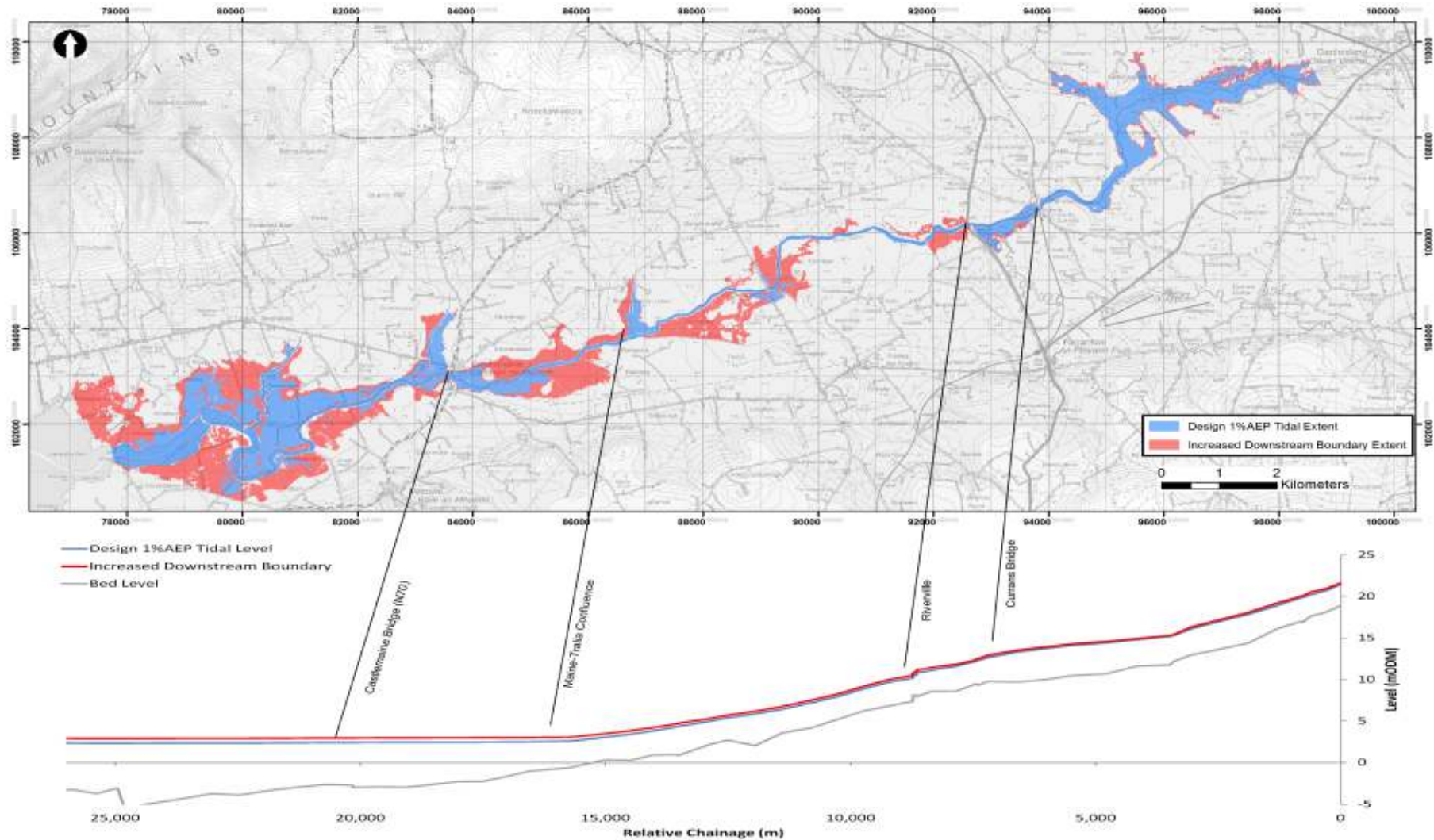
Flood risk in Killarney, Castleisland and Glenflesk was found to be less sensitive to the backwater assumptions taken at the downstream boundary, because the AFA was significantly above the downstream boundary and there were weir type structures that reduced the progression of backwater upstream.

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

Map 5.12: Sensitivity to Downstream Level – Dingle Model



Map 5.13: Sensitivity to Downstream Level – Maine MPW 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.

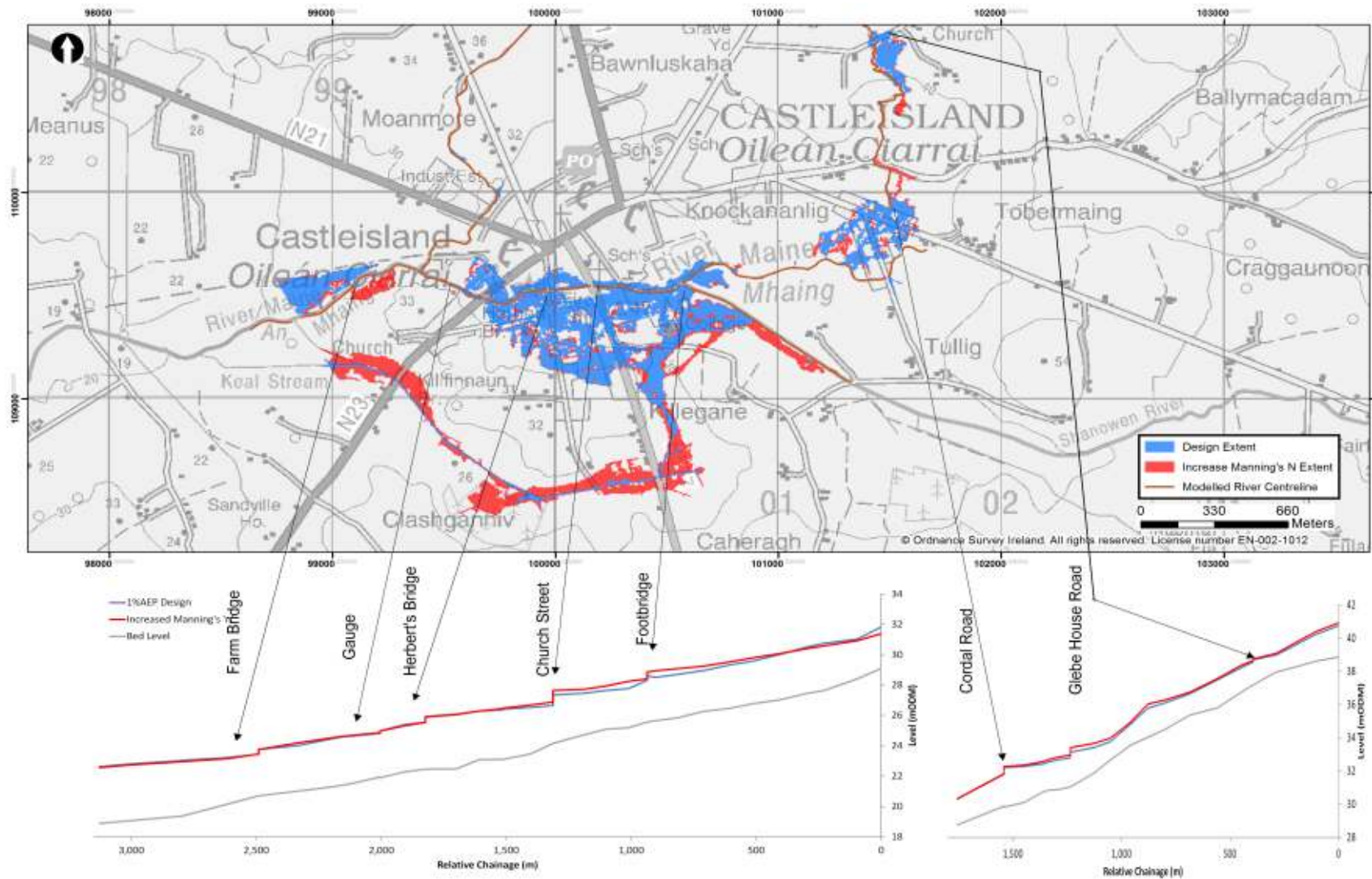
Table 5.3: Sensitivity Manning's 'n' Values

Channel or Surface	Design Manning's 'n'	Sensitivity Manning's 'n'
Active River Channel in Anglore and Flesk	0.050	0.055
Active River Channels elsewhere	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

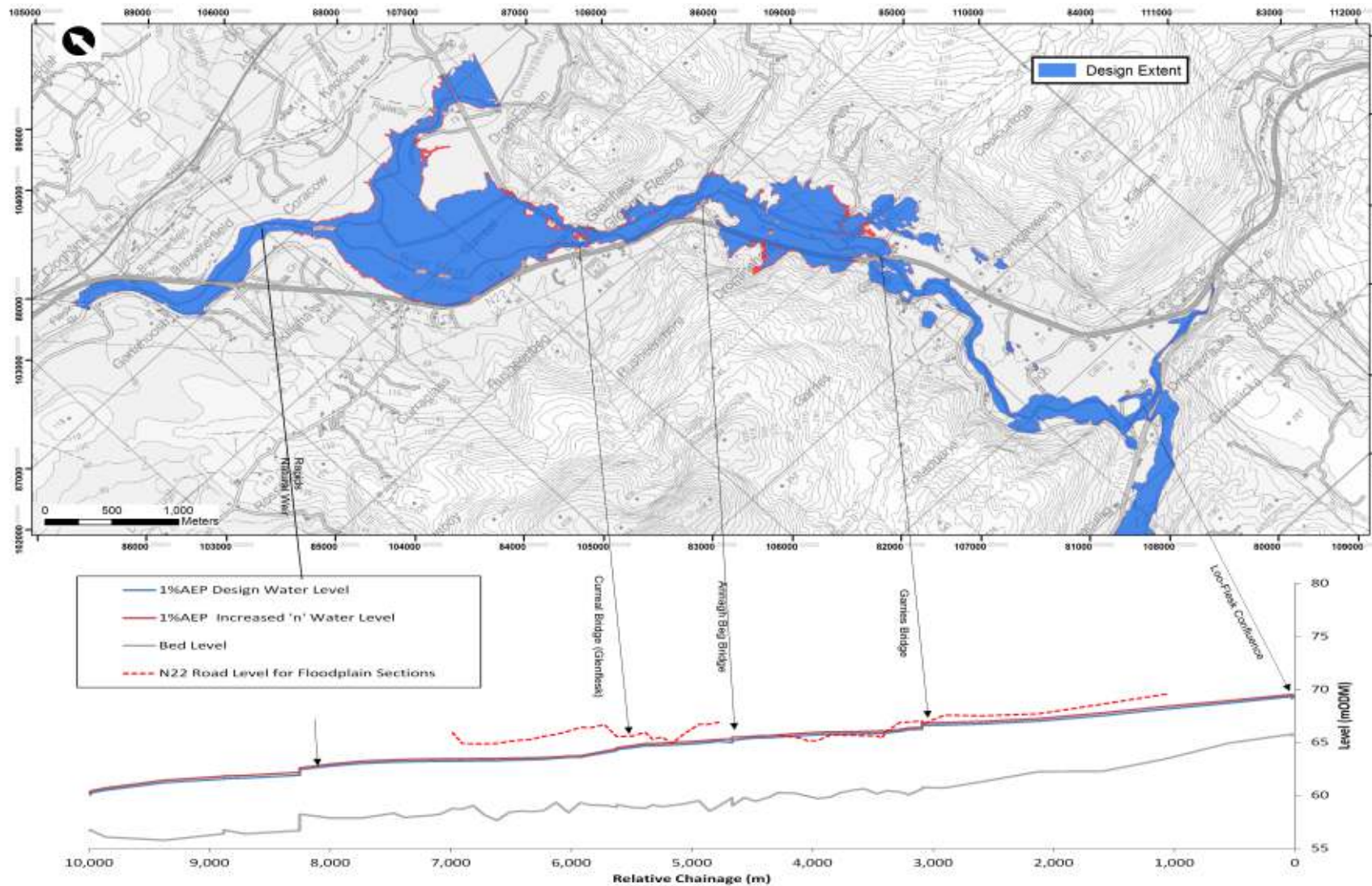
The greatest increase in flood risk attributed to Manning's 'n' was predicted in Castleisland, resulting in a larger flood extent along the R277 flow path (Map 5.14). Flooding across the N22 also increased due to the increased Manning's 'n' values, although the flood extent did not significantly increase elsewhere (Map 5.15).

The plots for all Manning's 'n' sensitivity tests can be found in the model performance proformas in the relevant Appendices. A summary of the impacts on levels are shown in Table 5.5.

Map 5.14: Sensitivity to Manning's 'n' - Castleisland



Map 5.15: Sensitivity to Manning's 'n' - Glenflesk



5.2.4 Culvert Coefficients in Milltown

The culverts along Old Station Road in Milltown AFA are reported to cause flooding in this area. The CIRIA Culvert Design and Operation Guide (2010) was used to derive the best estimate of inlet and outlet coefficients from a recommended range. A sensitivity test was undertaken on the limit of the recommended range for inlet and outlet coefficients assumed, to establish the impact of the conveyance of these structures on flood risk. All culvert coefficients were changed to the upper limit of the range. Table 5.4 outlines the changes to the circular culvert coefficients as an example.

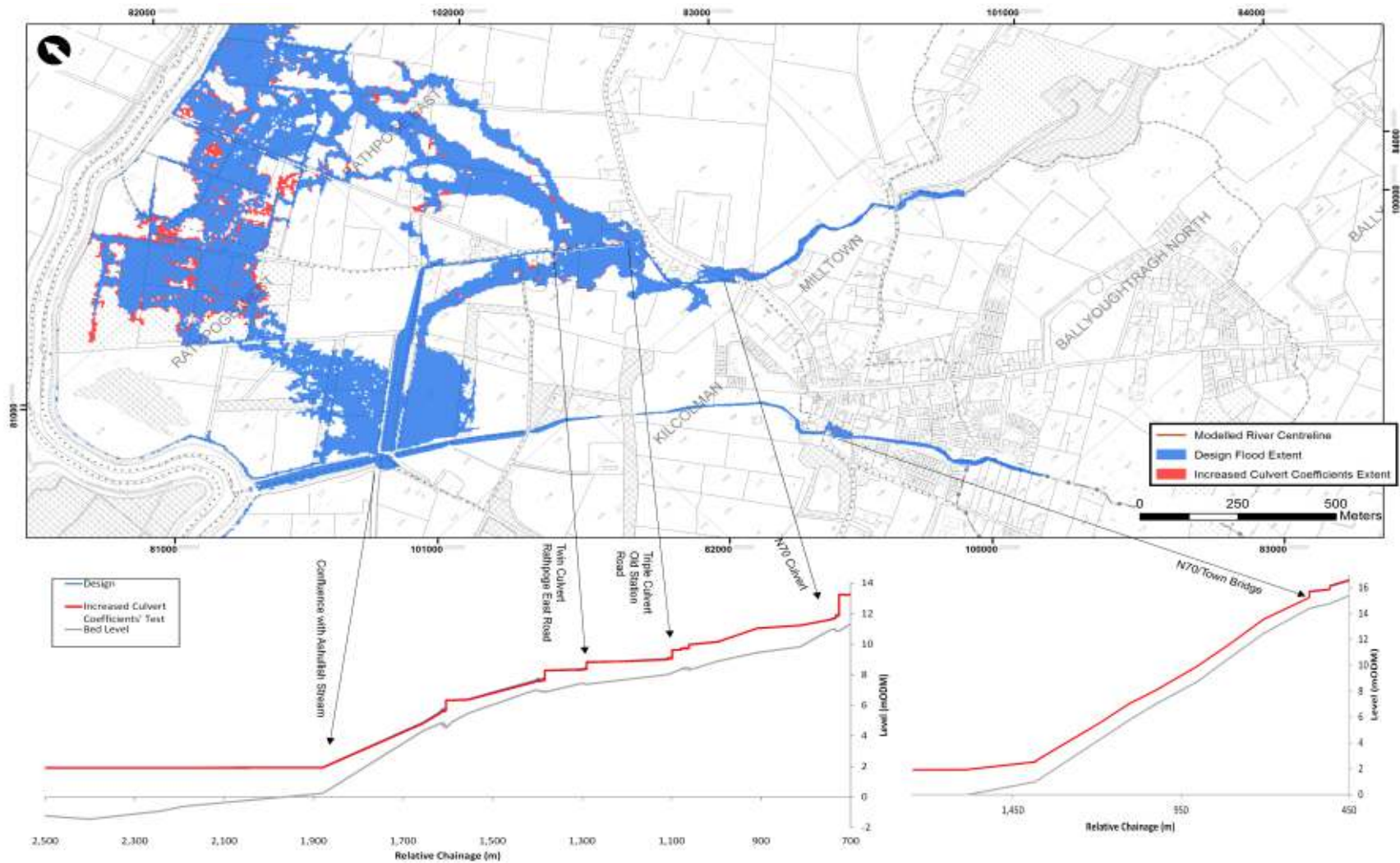
Table 5.4: Sensitivity on Culvert Coefficients

Culvert Coefficients/Parameters	Design (Verified against manual calculations using the CIRIA estimates)	Sensitivity Test (Combination to produce increased head loss)
Unsubmerged inlet control loss coefficient (K)	0.2	0.5
Exponent of Flow Intensity for inlet control (M)	2.0	2.00
Submerged inlet control loss coefficient (c)	0.0398	0.0553
Submerged inlet control adjustment factor (Y)	0.67	0.670

Map 5.16 compares the 1% AEP fluvial current event design results and the 1%AEP fluvial current event with increased loss coefficients along Old Station Road.

The increase in the culvert coefficients did not significantly change the maximum water level ($<0.05\text{m}$) because the 1%AEP design event already causes out-of-bank flooding. However, the increased coefficients increased head loss on the rising limb and reduced the capacity of the culverts by a maximum of $0.7\text{m}^3/\text{s}$ (20%) as shown in the hydrographs of Map 5.9. This causes flooding out-of-bank earlier and to a greater extent. Therefore, the effective capacity of the culverts and any blockage should be carefully considered when interpreting flood maps, deriving flood risk management options and assessing any future flood events.

Map 5.16: Sensitivity to Culvert Head Loss Assumptions – Milltown



5.2.5 Summary

Table 5.5 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 ($\pm 0.2\text{m}$). In some cases there is a significant increase in level but this does not result in a significant increase in flood extent, such as at Glenflesk.

Table 5.5: Summary of Sensitivity Run Performance

Model	Flow		Level		Manning's 'n'		Culvert Coefficients	
	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?	RMSD (m)	Sensitive?
Castleisland	0.19	Yes	0.05	No	0.11	Yes	N/A	
Maine	0.29	Yes	0.41	Yes	<0.01	No	N/A	
Milltown	0.26	Yes	Assessed as part of Maine		0.06	No	0.02	No
Glenflesk	0.36	Yes ^Δ	0.02	No	0.24	Yes ^Δ	N/A	
Killarney	0.38	Yes	0.09	No	0.09	No	N/A	
Laune	0.41	No	0.55	No	0.28	No	N/A	
Dingle	0.38	No	0.53	Yes	0.07	No	N/A	
Portmagee	N/A		0.55*	No			N/A	

RMSD is Root Mean Square Difference.

*RMSD for open coast in Portmagee the absolute increase in water level i.e. 0.55m.

^Δ The increase in water level did not significantly increase flood extent and risk to properties. However, flood risk increased to national infrastructure (N22). Therefore the model has been deemed sensitive to this parameter.

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

- Castleisland, Maine, Milltown and Killarney models are sensitive to assumptions and uncertainties in peak flow. Glenflesk has also been deemed sensitive to flow due to the increased flooding across the N22 in this test rather than an increase in flood extent within the AFA. 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.
- Dingle and Maine models are sensitive to the assumptions and uncertainties in the extreme sea levels. 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 uncertainty in the roughness values only increased flooding in Castleisland and to the N22 in Glenflesk at the 1%AEP. However, maintenance of the channel may provide some benefit for events which are closer to the threshold of flooding.
- The flood risk in Milltown was not deemed sensitive to the culvert coefficients applied at the 1%AEP fluvial event. However, it did reduce the culvert capacity and cause flooding earlier in the event. Therefore, the effective capacity of the culverts and any blockage should be carefully considered when interpreting flood maps and deriving flood risk management options to reduce flooding in more frequent event.

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 UoM22 and design event runs simulated.

Both the fluvial and coastal scenarios have been simulated for Dingle AFA, Laune MPW and Maine MPW as these reaches 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 3.4 of this report. The model results from the fluvial-dominated event and coastal-dominated event will be combined to derive the flood zone mapping described in Chapter 9 of this report. However, the fluvial results and coastal results are presented separately for the flood maps.

No fluvial scenarios have been simulated for Portmagee as the AFA was not identified as being at fluvial flood risk.

In order to calculate the undefended extent from the Flood Zone mapping and Defended Areas, additional undefended scenarios were run for the Maine and Milltown models where the water level would vary without the defences in place due to the capacity of the floodplain. The Flood Zone mapping and Defended Areas for the lower Laune was calculated using horizontal projection as these were entirely tidal.

Table 6.1: Design Event Runs- Defended

Source	Scenario	%AEP	Run Name	Castleisland I33CD	Maine I34ME	Milltown I35MN	Glenflesk I36GF	Killarney I37KY	Laune I39LE	Dingle I40DE	Portmagee I41PE
Fluvial	Current	50%	FCD500	✓	✓	✓	✓	✓	✓	✓	N/A
		20%	FCD200	✓	✓	✓	✓	✓	✓	✓	N/A
		10%	FCD100	✓	✓	✓	✓	✓	✓	✓	N/A
		5%	FCD050	✓	✓	✓	✓	✓	✓	✓	N/A
		2%	FCD020	✓	✓	✓	✓	✓	✓	✓	N/A
		1%	FCD010	✓	✓	✓	✓	✓	✓	✓	N/A
		0.50%	FCD005	✓	✓	✓	✓	✓	✓	✓	N/A
		0.10%	FCD001	✓	✓	✓	✓	✓	✓	✓	N/A
	MRFS	50%	FMD500	✓	✓	✓	✓	✓	✓	✓	N/A
		20%	FMD200	✓	✓	✓	✓	✓	✓	✓	N/A
		10%	FMD100	✓	✓	✓	✓	✓	✓	✓	N/A
		5%	FMD050	✓	✓	✓	✓	✓	✓	✓	N/A
		2%	FMD020	✓	✓	✓	✓	✓	✓	✓	N/A
		1%	FMD010	✓	✓	✓	✓	✓	✓	✓	N/A
		0.50%	FMD005	✓	✓	✓	✓	✓	✓	✓	N/A
		0.10%	FMD001	✓	✓	✓	✓	✓	✓	✓	N/A
	HEFS	10%	FHD100	✓	✓	✓	✓	✓	✓	✓	N/A
		1%	FHD010	✓	✓	✓	✓	✓	✓	✓	N/A
		0.10%	FHD001	✓	✓	✓	✓	✓	✓	✓	N/A
Coastal	Current	50%	CCD500	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		20%	CCD200	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		10%	CCD100	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		5%	CCD050	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		2%	CCD020	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		1%	CCD010	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		0.50%	CCD005	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		0.10%	CCD001	N/A	✓	N/A	N/A	N/A	✓	✓	✓

Source	Scenario	%AEP	Run Name	Castleisland I33CD	Maine I34ME	Milltown I35MN	Glenflesk I36GF	Killarney I37KY	Laune I39LE	Dingle I40DE	Portmagee I41PE
	MRFS	50%	CMD500	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		20%	CMD200	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		10%	CMD100	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		5%	CMD050	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		2%	CMD020	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		1%	CMD010	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		0.50%	CMD005	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		0.10%	CMD001	N/A	✓	N/A	N/A	N/A	✓	✓	✓
	HEFS	10%	CHD100	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		0.50%	CHD005	N/A	✓	N/A	N/A	N/A	✓	✓	✓
		0.10%	CHD001	N/A	✓	N/A	N/A	N/A	✓	✓	✓
TOTAL Model Runs				19	38	19	19	19	38	38	19

Table 6.2: Design Event Runs- Undefined

Source	Scenario	%AEP	Run Name	Castleisland I33CD	Maine I34ME	Milltown I35MN	Glenflesk I36GF	Killarney I37KY	Laune I39LE	Dingle I40DE	Portmagee I41PE
Fluvial	Current	5%	FCU050	N/A	✓	N/A	N/A	N/A	N/A	N/A	N/A
		2%	FCU020	N/A	N/A	✓	N/A	N/A	N/A	N/A	N/A
		1%	FCU010	N/A	✓	✓	N/A	N/A	N/A	N/A	N/A
		0.1%	FCU001	N/A	✓	✓	N/A	N/A	N/A	N/A	N/A
	MRFS	1%	FMU010	N/A	✓	✓	N/A	N/A	N/A	N/A	N/A
		0.1%	FMU001	N/A	✓	✓	N/A	N/A	N/A	N/A	N/A
Coastal	Current	0.5%	CCU005	N/A	✓	N/A	N/A	N/A	*	N/A	N/A
		0.1%	CCU001	N/A	✓	N/A	N/A	N/A	*	N/A	N/A
	MRFS	0.5%	CMU005	N/A	✓	N/A	N/A	N/A	*	N/A	N/A
		0.1%	CMU001	N/A	✓	N/A	N/A	N/A	*	N/A	N/A
TOTAL Model Runs				0	9	5	0	0	*	0	0
										Horizontal projection without defences in place	

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 6.5 show the performance dialog for the 1%AEP fluvial event for the following run performance criteria in AFAs;

- 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 $0.01 \text{ m}^3/\text{s}$ 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 events in all models. There is no 1D convergence plot for Portmagee as there are no 1D components for this model. The brief periods of poor convergence can be explained as follows:

- In Milltown, the outflow varies due to the influence of the tide in the Maine preventing free flow at the outfall as expected. The brief 'spikes' of poor convergence are attributed to the opening and closing of the flapped outfall in to the Maine. These spikes do not impact the peak level or flood duration and are therefore acceptable.
- The brief non-convergence in the Glenflesk model is attributed to overtopping of the Loo Bridge. This causes a minor oscillation in the flow at this node, but this normalises after 0.25 hours and does not impact the river sections upstream or downstream of the bridge.
- There are a couple of brief spikes of poor convergence in Dingle as the water reaches the soffit at key structures which changes the hydraulic regime from free-flow conditions to drowned mode and back again. This is particularly evident at the new access bridge opposite the Brewery Gate, and the long culvert under Spa Road.

The cumulative mass balance for the 2D model components is shown in Figures 6.6 to 6.10. The design models were convergent and within the recommended tolerance of $\pm 1\%$ mass error at the peak flow and/or tide plus surge level with the exception of Portmagee. In this case, the mass error was outside the 1% recommended tolerance due to the small number of wet cells and oscillation in water level with the incoming and outgoing tide. However, the mass error does not affect flood risk as the flood levels are not affected, and the volume overtopping is not critical where the ground rises inland such as Portmagee.

Figure 6.1: 1D Convergence Plot - Castleisland

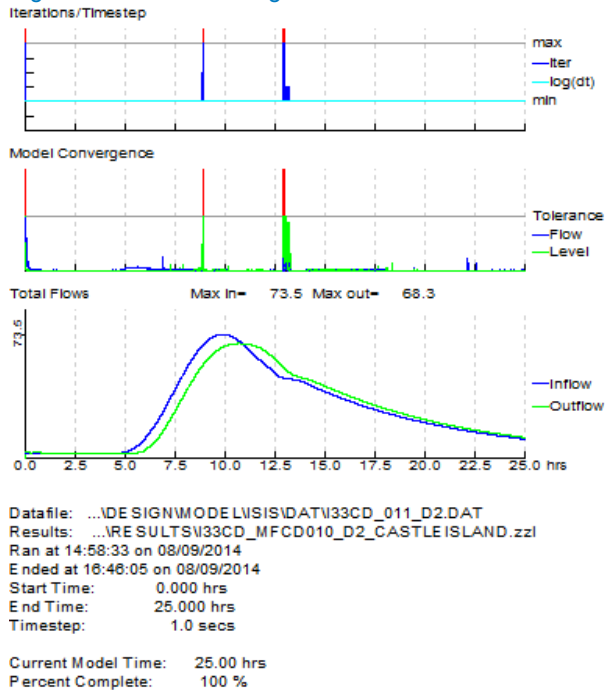


Figure 6.2: 1D Convergence Plot - Milltown

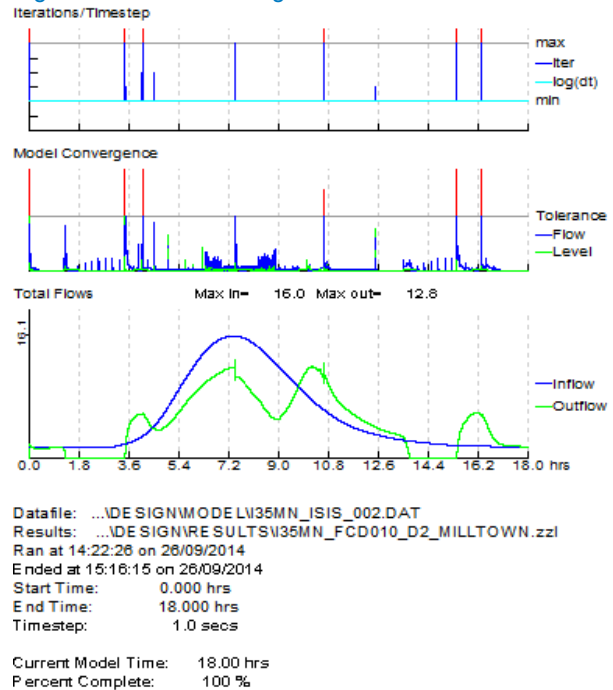


Figure 6.3: 1D Convergence Plot - Glenflesk

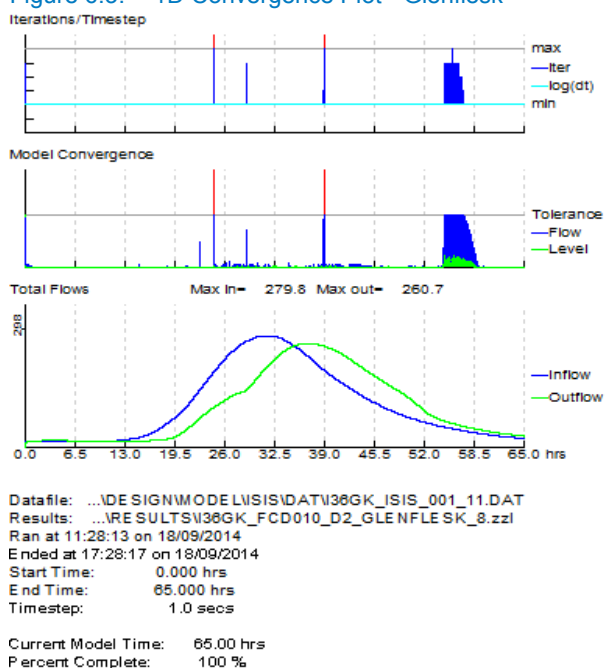


Figure 6.4: 1D Convergence Plot - Killarney

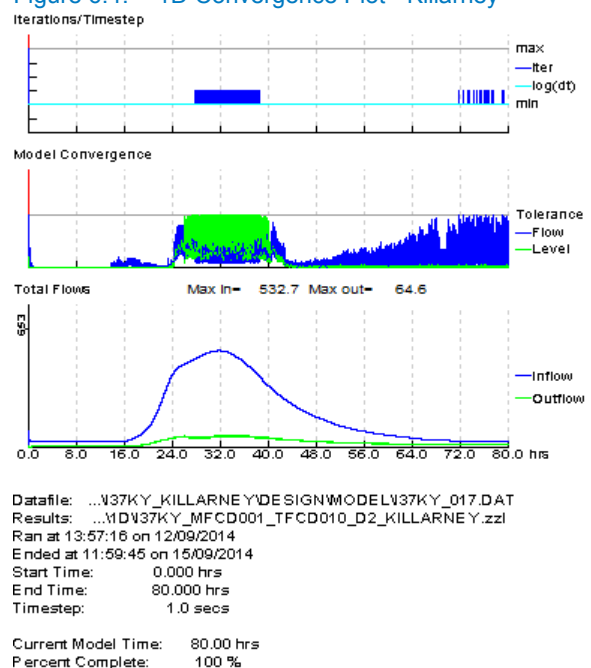


Figure 6.5: 1D Convergence Plot – Dingle

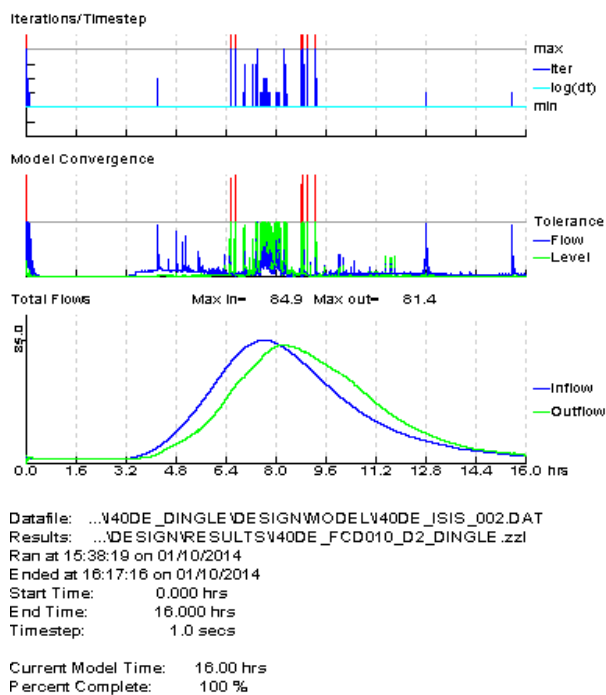


Figure 6.6: 2D Mass Balance Plot - Castleisland

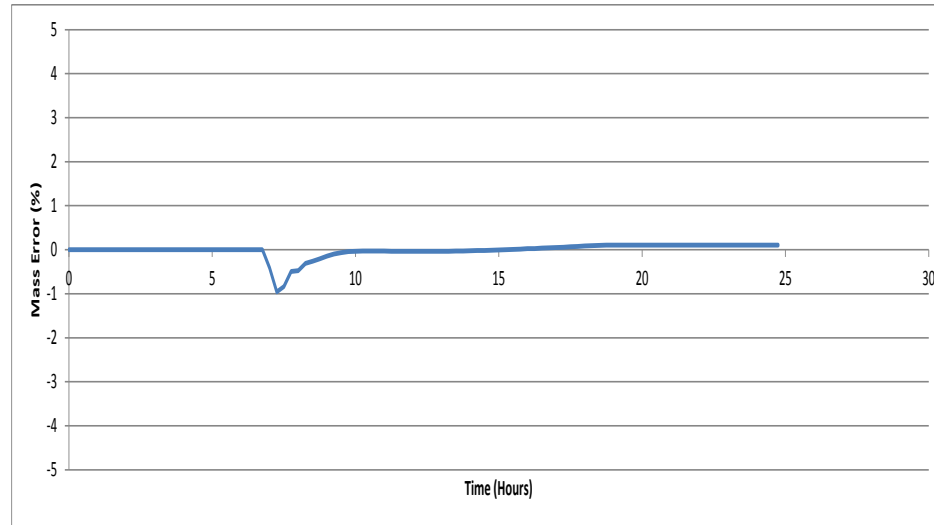


Figure 6.7: 2D Mass Balance Plot - Milltown

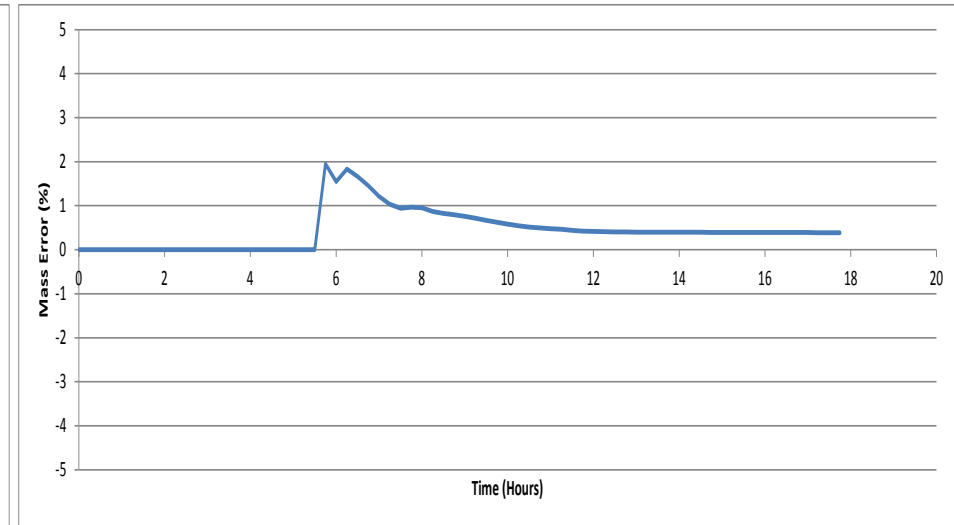


Figure 6.8: 2D Mass Balance Plot – Glenflesk

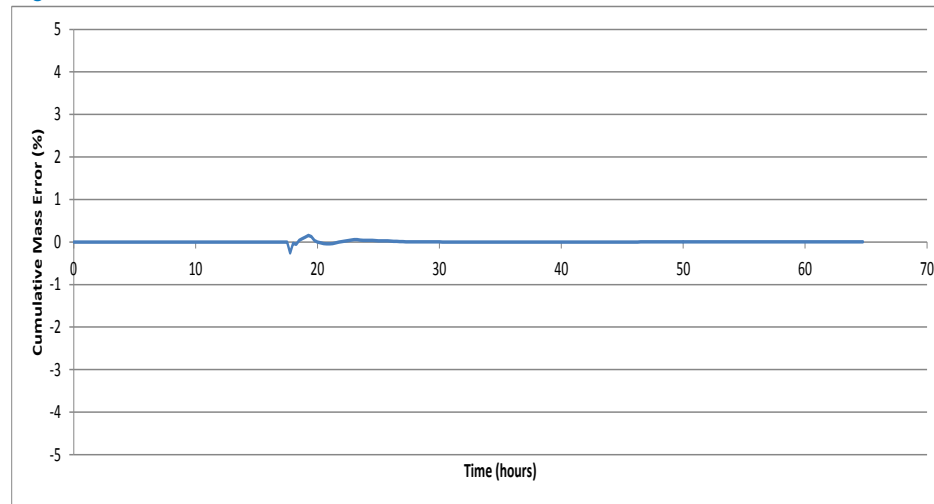


Figure 6.9: 2D Mass Balance Plot – Killarney

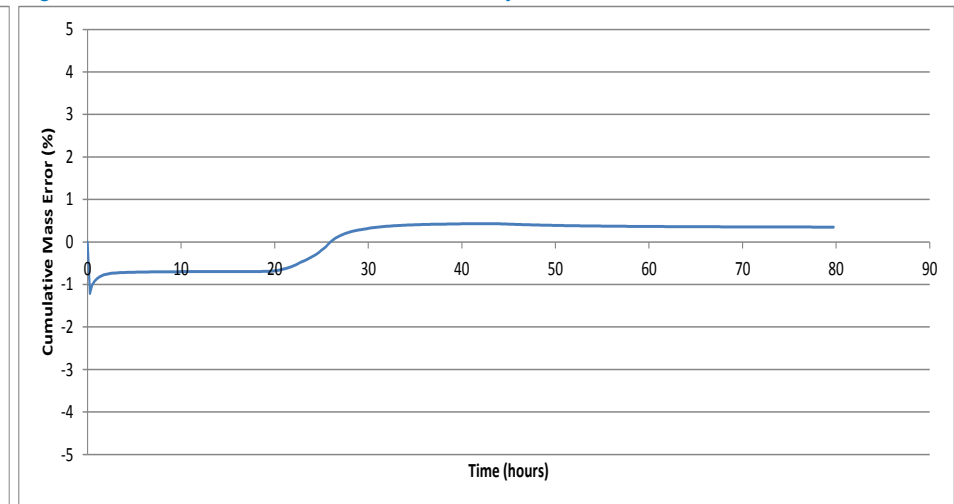


Figure 6.10: 2D Mass Balance Plot - Dingle

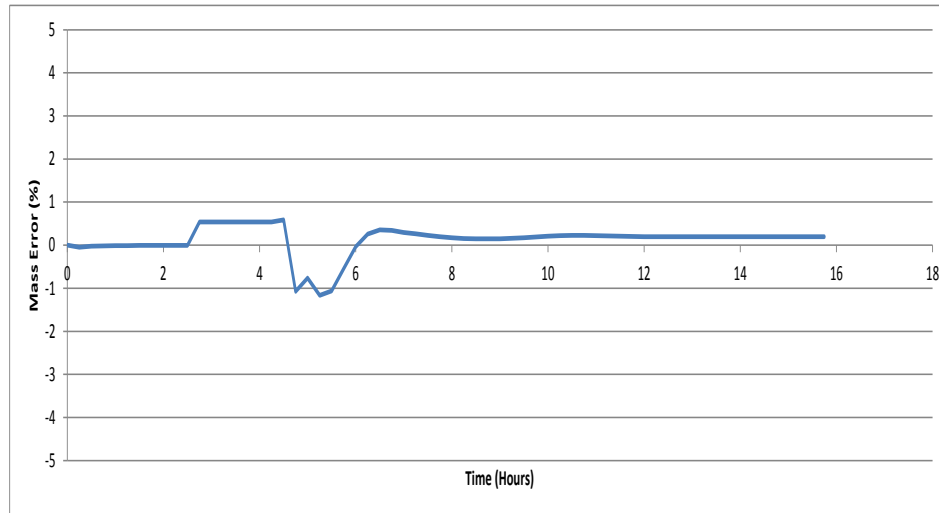
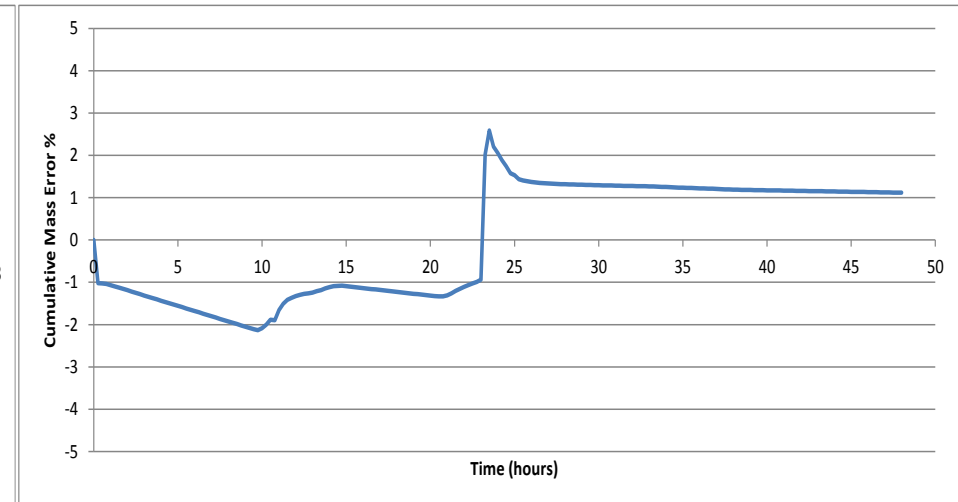


Figure 6.11: 2D Mass Balance Plot - Portmagee



Tables 6.2 compares the model predicted flows with the design peak flows at the target HEPs for the target 1%AEP event.

Modelled flow has been extracted directly from the extended 1D sections from the MPW reaches using a 1D only approach including; the Maine (Castleisland to Currans Bridge); Glenflesk (upstream of Glenflesk); Killarney (N22 to White Bridge); and, Laune models. Modelled flows in AFAs and the lower Maine MPW (downstream of Currans Bridge) 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.

Table 6.3: Summary of Hydrological Routing Performance for Key Fluvial Current Events

HEP ID	Location	Model Node	10%AEP			1%AEP			0.1%AEP		
			Design Target Flow (m3/s)	Model Predicted Flow (m3/s)	Difference (m3/s)	Design Target Flow (m3/s)	Model Predicted Flow (m3/s)	Difference (m3/s)	Design Target Flow (m3/s)	Model Predicted Flow (m3/s)	Difference (m3/s)
Castleisland AFA											
22_1589_3	Shanowen downstream	22SHAN00002A	37.2	37.1	0%	54.2	54.8	1%	79.4	80.5	1%
22_1587_3	22014 Castleisland Gauge	22MAIN04748H	43.6	41.5	-5%	61.8	57.6	-7%	91.8	71.2	-22%
	22014 Castleisland Gauge+ flow exiting catchment	22MAIN04748H	43.6	41.5	-5%	61.8	58.8	-5%	91.8	88.4	-4%
22_2098_3	Maine downstream	22MAIN04669A	56.9	51.4	-10%	80.7	75.4	-7%	119.9	113.2	-6%
22_360_2	Glenshearoon downstream	22GLAN00002H	12.6	11.0	-12%	18.7	15.8	-16%	27.8	22.4	-19%
22_1756_3	Anglore downstream	22ANGL00021I	3.3	3.7	12%	4.7	5.7	21%	7.0	8.3	19%
Milltown AFA											
22_3116_1	Ashullish-Ashullish Trib d/s	22ASHU00144H	3.3	3.2	-2%	5	4.9	-1%	7.5	7.4	-1%
22_3116_4	Ashullish-Ballyoughtragh U/s	22ASHU00000H	5.6	5.4	-4%	8.5	8.2	-4%	12.9	12.1	-6%
22_3617_1	Ashullish-Ballyoughtragh D/s	22TOWN00046B	9.8	10.1	3%	14.9	12.6	-15%	22.5	15.5	-31%
22_3958_2	Ashullish Stream d/s	22TOWN00010A	10.6	11.9	12%	16.1	14.1	-13%	24.3	15.4	-37%
22_3425_9	Ballyoughtragh Stream D/S	22TOWN00100A	4.2	4.0	-4%	6.4	6.2	-4%	9.8	9.1	-7%
Glenflesk AFA											
22_1561_2	Flesk/Annagh Beg Confluence	22FLES01721H	143.2	140.46	-2%	207	206	0%	315	312	-1%
22_2859_1	Flesk d/s Owneyskeagh	22FLES01433B	196.1	187.0	-5%	283	277	-2%	432	415	-4%
Killarney AFA											
22_3340_8	Flesk Upstream	22FLES00547H	204.2	203.71	0%	304	295	-3%	450	449	0%
22_3372_1	Flesk downstream Woodford	22FLES00480A	214.2	212.5	-1%	319	305	-4%	472	470.2	0%
22006	Flesk Bridge Gauge	22FLES00201B	224.4	221.9	-1%	334	322	-4%	494	487	-1%
22_3372_11	Flesk downstream survey extent	22FLES00019H	226.5	221.5	-2%	338	321	-5%	494	494	0%
22009	White Bridge gauge	22DEN00094B	15.4	15.02	-2%	24.3	23.6	-3%	36.9	37.0	0%

HEP ID	Location	Model Node	10%AEP			1%AEP			0.1%AEP		
22_3340_8	Flesk Upstream	22FLES00547H	204.2	203.71	0%	304	295	-3%	450	449	0%
Dingle AFA											
22_3437_5	Downstream of Dingle Stream	22DING00003H	7.42	6.5	-12%	11.2	9.0	-20%	16.9	12.0	-29%
22_1712_2	Milltown Gauge (22022)	22MILL00165G	39.7	39.9	1%	62.0	62.3	0%	88.4	89.3	1%
22_3998_1	Milltown Stream-Ballymoreeagh Trib D/S	22MILL00071H	47.58	40.5	-15%	74.3	62.9	-15%	106	89.6	-15%
22_3999_2	Downstream of Milltown Stream	22MILL00002A	48.79	46.6	-4%	76.2	72.4	-5%	108	105	-3%
Portmagee AFA (No fluvial assessment)											

The modelled flows are typically within 10% of the design flows where the HEP is not bypassed or affected by backwater. The discrepancies in flows at the tidal outfalls (highlighted in the table) are due to backwater effects that limit discharge. This tide-locking is not considered in the design hydrology which assumes free-flow conditions in a fluvial-dominated reach.

The modelled flows tended to underestimate the design flows on the Glenshearoon and overestimate the flows on the Anglore Stream because flow exited the Glenshearoon catchment and were transferred to the Anglore Stream catchment. The modelled peak flows are less than the design flows at Castleisland gauge for extreme events because up to 20 m³/s exits the catchment along the main road. The modelled flow is within 5% when this cross-catchment flow is accounted for.

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 additional inefficiencies in flow around street furniture such as fences have been incorporated into the higher general floodplain Manning's 'n' of 0.06 in urban areas.
- 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 impeded 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.

7.2 Limitations

There are a number of uncertainties associated with the flow estimation and hydraulic modelling methodology used in UoM22. They include:

- There is uncertainty in the derivation of design flows for Anglore Stream (Castleisland) due to the subterranean flow paths and influence of groundwater on this karstic system. There was no gauge data but the flood extents have been calibrated to historic flooding along this watercourse. This level of uncertainty must be considered in the interpretation of design flows, flood mapping and in the development of flood mitigation options.
- The absence of river flow or continuous water level data in Milltown catchment 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. This is of particular relevance to the karstic Glenshearoon and Anglore catchments. However, the CFRAMS modelling does consider the karstic system as an alternative flow route during high flows in accordance with local engineers' observations.

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 maps for each AFA and MPW reach;
- Maximum velocity maps for each AFA;
- Maximum flood hazard maps for each AFA;
- Maximum flood extent maps for each AFA and MPW reach;
- Flood Zone maps for each AFA and MPW reach;
- Specific Risk Number of Inhabitants maps for each AFA and MPW reach;
- Specific risk Types of Economic Activity maps for each AFA and MPW reach; and,
- Specific Risk Density maps for each AFA and MPW reach.

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. 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.

8.2 Flood Depth and Velocity Mapping

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

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

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

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. The flood hazard was modified from the DEFRA FD2320 guidance:

$$\text{Flood Hazard} = \text{Depth} \times (\text{Velocity} + 0.5)$$

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 and velocity. The maximum flood hazard rating value is the maximum of these concurrent flood hazard values but does not necessarily coincide with both the maximum depth and maximum velocity. This is produced directly by the TUFLOW model and requires no post-processing to derive flood hazard.

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

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

Table 8.1: Flood Hazard Categories

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

Source: DEFRA FD2320 Table 2 Hazard to People

8.4 Flood Extent and Zone Mapping

The maximum flood extent was derived from the maximum flood depth grid and converted to a closed polygon. 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.

Raised embankments along the River Maine and Castlemaine Harbour protect the floodplain from coastal flooding. The standard of protection was identified as the %AEP event which was closest to the defence

level but did not cause flooding which ranges from the 50% to 1%AEP in UoM22. The Defended Areas were then derived from the water levels for the 50% to 1%AEP event without the flood embankment in place.

The Flood Zone outlines were derived from the undefended scenarios where there were raised defences e.g. along the River Maine and from the design extents where there were no formal defences. Flood zone A was derived from the 1%AEP fluvial and 0.5%AEP coastal extents. Flood Zone B was derived from the 0.1%AEP fluvial and 0.1%AEP coastal extents.

8.5 Combined Flood Source Mapping

Dingle AFA, Laune MPW and Maine MPW 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.

8.6 Flood Risk (Assessment) Mapping

8.6.1 General Flood Risk Maps

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

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

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

8.6.2 Specific Flood Risk Maps

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

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

8.6.2.1 Indicative Number of Inhabitants

For each AFA, the study area was broken into a number of grids, each 100m² (i.e. 1 Ha). The population density per Ha was calculated by summing the number of residential properties within each grid and multiplying by an average occupancy rate determined by the Central Statistics Office.

8.6.2.2 Types of Economic Activity

Each property within an AFA was assigned a use, which was based on the property survey. The types of economic activity were identified and intersected with the 10%, 1% and 0.1%AEP fluvial extents and 10%, 0.5% and 0.1%AEP coastal extents to generate the datasets listed in Table 8.2. The resultant economic activity at risk have then been mapped for these key %AEP events.

Table 8.2: Derived Datasets of Economic Activity at Risk

Economic Activity	Derived Dataset	Description
Buildings	Buildings in flood (DG)	Buildings located in modelled flood extents
Infrastructure	Infrastructure in flood extents(DI)	Existence of infrastructure in flood extent
Commercial	Commercial use within flood extents(DK)	Existence of commercial land use in flood extent
Rural	Rural land use in flood extents(DJ)	Existence of rural land use in flood extent

8.6.2.3 Economic Risk Density

The maximum depth of flooding was extracted for each building polygon for the full range of AEP events using the results of the hydraulic modelling and flood mapping.

The calculation of flood damages was based on the Flood Hazard Research Centre Handbook of 2010 (FHRC, 2010) and the “Multi-Coloured Manual” of 2005 (FHRC, 2005) as referred to in FHRC 2010, subject to caveats, amendments and clarifications as set out in the National CFRAM Programme Guidance Note No.27 Rev C.

Damage costs were converted to euro by applying a Purchasing Price Parity multiplication factor and an inflation factor. For Residential Properties damage costs were calculated based on the depth of flooding and the corresponding unit cost of damage for property type. For Non-residential Properties damage costs were calculated based on the depth of flooding and the unit cost of damage for property type per m².

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 100m² (i.e. 1 Ha) grid was summed and represented on a map providing the economic risk density (€ AAD / Ha).

9 Model and Mapping Results

9.1 Overview

Based on the model predicted results and flood maps, the greatest fluvial flood risk to properties and infrastructure in UoM22 is located in Castleisland where flooding starts at the 5%AEP. Flood risk is also significant in Dingle and Milltown AFAs from the 10%AEP and the N22 along the Flesk is affected by the 50%AEP and larger events. There is also extensive fluvial flooding of agricultural and pastoral land along the Maine and Lower Laune from the 5 %AEP fluvial event and 50%AEP coastal current event.

Regular fluvial flooding was predicted in Killarney AFA, but this was contained to the valley floodplain and Killarney National Park areas. However, properties were not affected until the 0.1%AEP.

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 the receptors is discussed in the subsequent Flood Risk Assessment.

9.2 Castleisland AFA

Map 9.1 summarises the fluvial flood risk in Castleisland 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 Glanshearoon right bank at Crag Cottages into a swallow hole and connected to increased flows and flooding along Anglore Stream (Figure 9.1)
- Overtopping of the left bank of Anglore Stream at Glebe House Road Bridge and flowing across the road to flood properties.
- Backwater from the downstream culvert on Anglore Stream flooding properties in Tullig – This corresponds with regular flooding along Cordal Road. However the flooding is shallow and the extent may be reduced by local urban drainage not considered in this model.
- Backwater from Herbert's and Barrack Lane Bridge cause overtopping of the Maine downstream of Church Street bridge to flood properties on the left bank.
- Backwater from Church Street Bridge causes overtopping of the Maine upstream of the bridge to flood properties on the right bank.
- In the most extreme events, flood waters flow across the road at Castleisland Community College and into the neighbouring catchment towards Killfinnaun Bridge.

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

- 50%AEP fluvial event overtops the river bank at Crag Cottages and enters the Crag Cave.
- 50%AEP fluvial event exceeds the downstream culvert on Anglore road to flood Tullig (Approximately 50 properties).
- 2% AEP fluvial event overtops the left bank downstream of Church Street but no properties are affected.
- 1% AEP fluvial event overtops the left bank downstream of Church Street and causes limited flooding to less than ten properties.
- 0.1% AEP fluvial event overtops the right bank upstream of Church Street and causes extensive property flooding (> 300 properties).
- 0.1% AEP fluvial event floods across the Community College and into the Killfannaun catchment.

The greatest risk to life is associated with deep and fast flowing water flooding on the left bank downstream of Church Street, and along the R577 from the Maine and across Glebe House Road from Anglore Stream. However, the risk to life at Tullig is low to moderate because the flooding is shallow.

The critical structures in determining fluvial flood risk include:

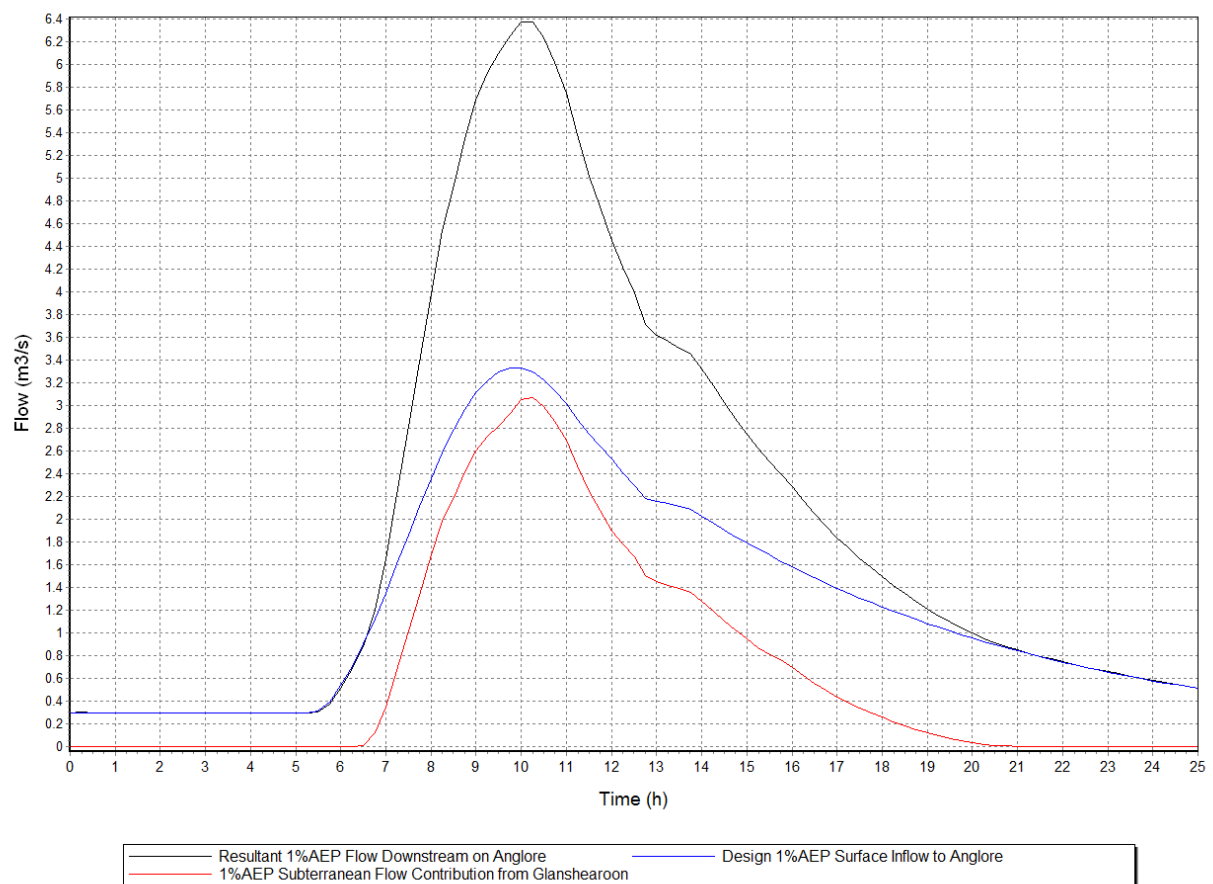
- Herbert's, Barrack Lane and Church Street Bridges on the River Maine including the utility crossing below the soffit on Hebert's and Barrack Lane Bridge.
- The raised river bank at Crag Cottages which determines the threshold at which the Glanshearoon floods into the swallow hole, thereby increasing flows through the cave system to Anglore Stream.
- Glebe House Road Bridge and the downstream culvert on Anglore Stream.

The areas flooded are consistent with the recorded flooded areas in 2008 and the more recent event in January 2014. The subterranean flow path between Glanshearoon and Anglore was highlighted by local authority staff during the flood risk review and confirmed by site visits during this study. The exact route, capacity of the cave system, and travel time of these subterranean flows, are not easily quantified. However, the model was calibrated well with the flood extent in the 2008 event. Therefore there is reasonable confidence in the flood mapping in Castleisland based on the information available at the time of this study.

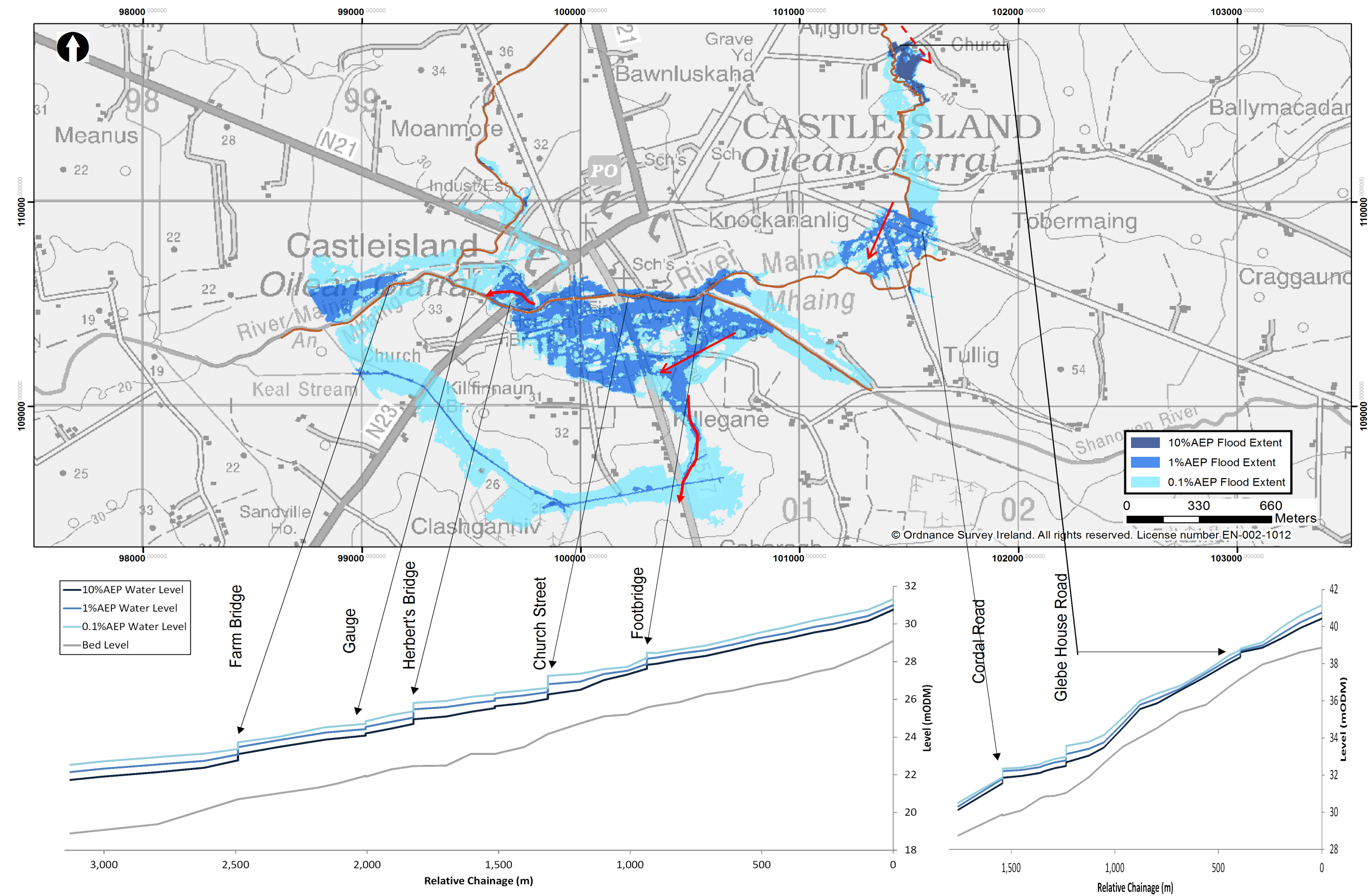
The following recommendations for flood risk management option development can be made:

- Improved conveyance at and around the Maine bridges identified to increase channel capacity without increased erosion.
- Improved conveyance through Glebe House Road Bridge and the downstream culvert on Anglore Stream to improve channel capacity.
- Raised river banks and/or other protection measures to limit the amount of water entering swallow holes from the Glanshearoon.
- Flood warning is likely to be effective given the > 6 hours' time to peak for both the Glenshearoon and Upper Maine catchments.

Figure 9.1: Increased Flow on Anglore Stream from the Glanshearoon-Crag Cave Subterranean route



Map 9.1: Summary of Fluvial Flood Risk – Castleisland



9.3 Milltown AFA

Map 9.2 summarises the fluvial flood risk in Milltown 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 Ballyoughtrough along Old Station Road due to the capacity of the culverts and flooding towards Rathpoge East. Less than 10 houses are affected by flooding.
- Overtopping of Ballyoughtrough along Old Station Road due to the capacity of the twin culvert and flooding towards Ashullish Stream.
- Backing up from the N70 bridge and bypassing of the footbridge upstream in Milltown.
- Overtopping of the right bank and bypassing of the N70 bridge on Bridge Street in predicted climate change conditions.
- Backing up of water in the raised embankment reach during periods of high tide in the Maine, combined with high fluvial flows overtopping the raised embankments at the confluence of Ballyoughtrough and Ashullish Streams.
- The raised embankments at the outfall protect 0.425km² from flooding up to and including the 2%AEP fluvial event.

The key thresholds and areas affected by flooding are:

- 50%AEP event exceeds the capacity of the Ballyoughtragh Stream downstream of the N70 causing water to spill over the right bank in two locations. This is modelled to impact a single property.
- 50%AEP overtops at the confluence of the Ballyoughtragh and Ashullish Streams at low points in the embankments to flood fields.
- 10%AEP causes shallow flooding by the N70 at Hurley's Bridge.
- 2%AEP event causes additional sections of the Ballyoughtragh Stream downstream of the N70 to spill, impacting additional properties.
- 1%AEP event floods the N70 at Town Bridge on the Ashullish Stream
- 0.5%AEP event bypasses town bridge
- Approximately 17 properties are affected by the 1%AEP fluvial event.

It should be noted that the drainage system and tributaries towards Cloonmore and Kilburn has not been modelled as part of the CFRAM study as these are not MPW or HPWs.

The greatest risk to life is associated with deep water on the left bank of Ballyoughtrough Stream upstream of the confluence with Ashullish Stream. However, no properties are affected by flooding in this area. Flooding along Old Station Road is classified as low to moderate risk to life because the flooding is shallow up to the 1%AEP. However, risk to life along Old Station Road increases to significant in the 0.1%AEP fluvial event.

The critical structures in determining fluvial flood risk include:

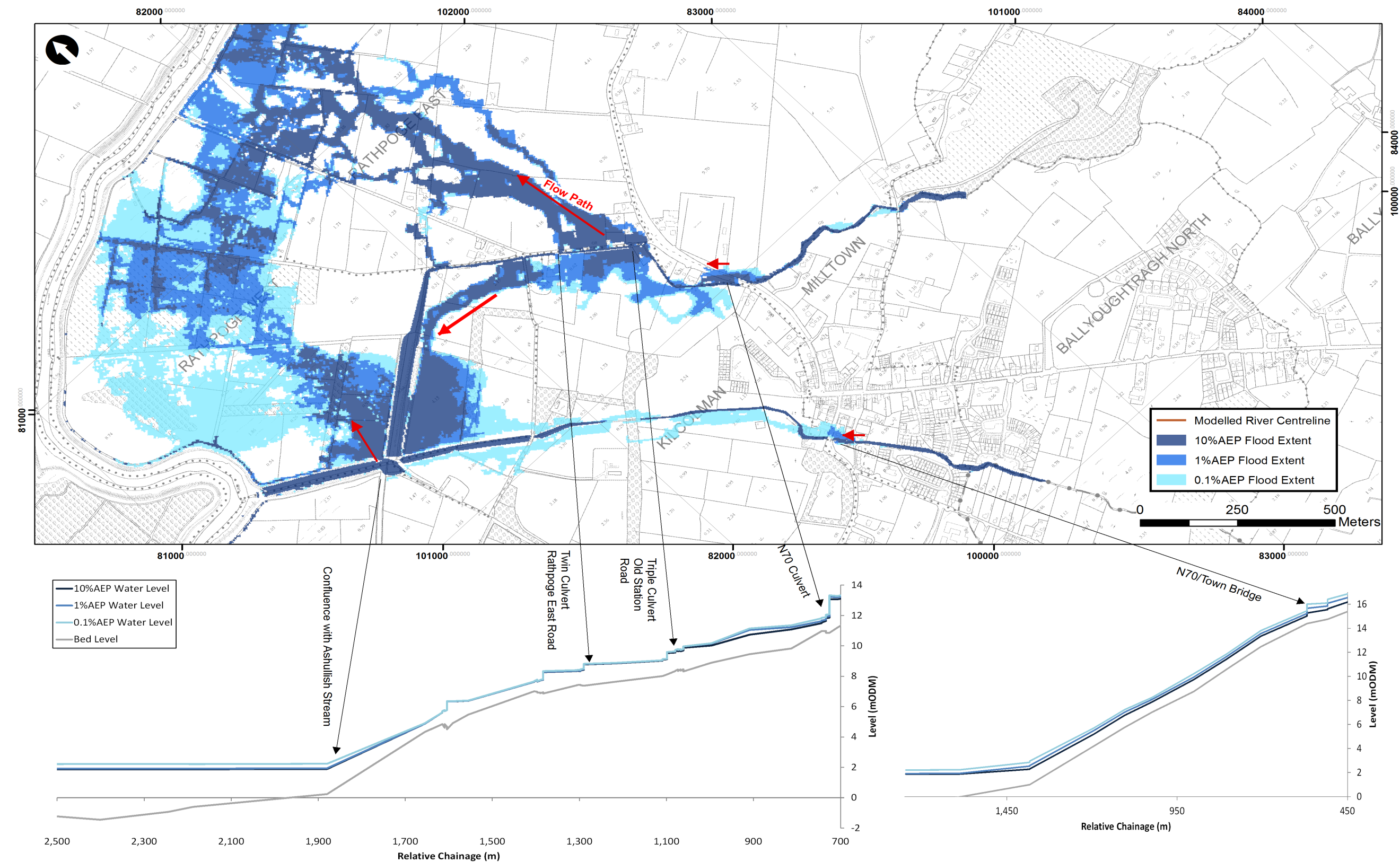
- The culverts on Ballyoughtrough stream along Old Station road, particularly the downstream triple culvert at 082195,101423 in combination with upstream triple culvert at 082432,101235.
- The raised embankments and flapped outfall at the outfall to the Maine for flood risk to the surrounding low-lying ground.
- The N70 road bridge for flood risk near Bridge Street in Milltown.

The areas flooded are consistent with the recorded flooded areas in January 2008 and the comments by local authority staff and local residents during the flood risk review. The sensitivity test demonstrated the uncertainty in flow estimates, roughness and culvert coefficients did not significantly increase levels in the upper reaches. Therefore there is reasonable confidence in the flood mapping in the upstream reaches. However, the uncertainty in flow estimates did significantly affect flood risk near the outfall. Therefore, the flood mapping in this area should be carefully considered with the limitations of the ungauged hydrology methodology.

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

- Improved conveyance at and around the culverts along Old Station Road.
- Improved conveyance at and around the N70 bridge on Bridge Street.
- Flood storage is more appropriate in the upstream reaches beyond the backwater effect from the Maine.
- Flood warning is unlikely to be effective given the short time to peak for these steep catchments.

Map 9.2: Summary of Fluvial Flood Risk – Milltown



9.4 Glenflesk AFA

Map 9.3 summarises the fluvial flood risk in Glenflesk AFA and flood risk to the N22 upstream for the 10%, 1% and 0.1%AEP design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Flooding of the N22 raised road embankment between Garries Bridge and Annagh Beg Bridge due to the limited floodplain capacity between the N22 and Islandmore road parallel to the channel.
- Overtopping of the N22 raised road embankment at Loo Bridge due to backing up from the bridge and flood relief structures.
- Backing up from the rapids at 105030,087205 and the confluence to overtop of the right bank of the Flesk at Glenflesk and left bank of the Owneyskeagh, largely flooding fields but limited property flooding (less than 5 in number) upstream of Curreal Bridge.

The key thresholds and areas affected by flooding are:

- 50% AEP inundates the floodplain and encroaches onto the N22. Surface water runoff onto the road is not considered in the CFRAMS model.
- 10% AEP event overtops the N22 downstream of Garries Bridge and 5%AEP floods the N22 from Glenflesk to Garries Bridge.
- 0.1%AEP causes significant flooding along all modelled watercourses. There is extensive inundation in Glenflesk and Islandmore. The N22 and property along its path are completely flooded upstream of the confluence of the River Flesk and Annagh Beg Stream.
- Limited property flooding along the right bank in the 2%AEP and larger fluvial events.

The greatest risk to life is associated with deep water flooding between the Flesk and Owneyskeagh Rivers but does not affect properties or roads. The risk to life upstream of Annagh Beg Bridge was not calculated as it is MPW and hazard is not required.

The critical structures in determining fluvial flood risk include:

- The rapids at 105030,087205 which determine water levels in Glenflesk AFA.
- The raised road embankments of the N22 and Islandmore road.
- Loo Bridge and the flood relief culvert limiting the flow passing under the N22.

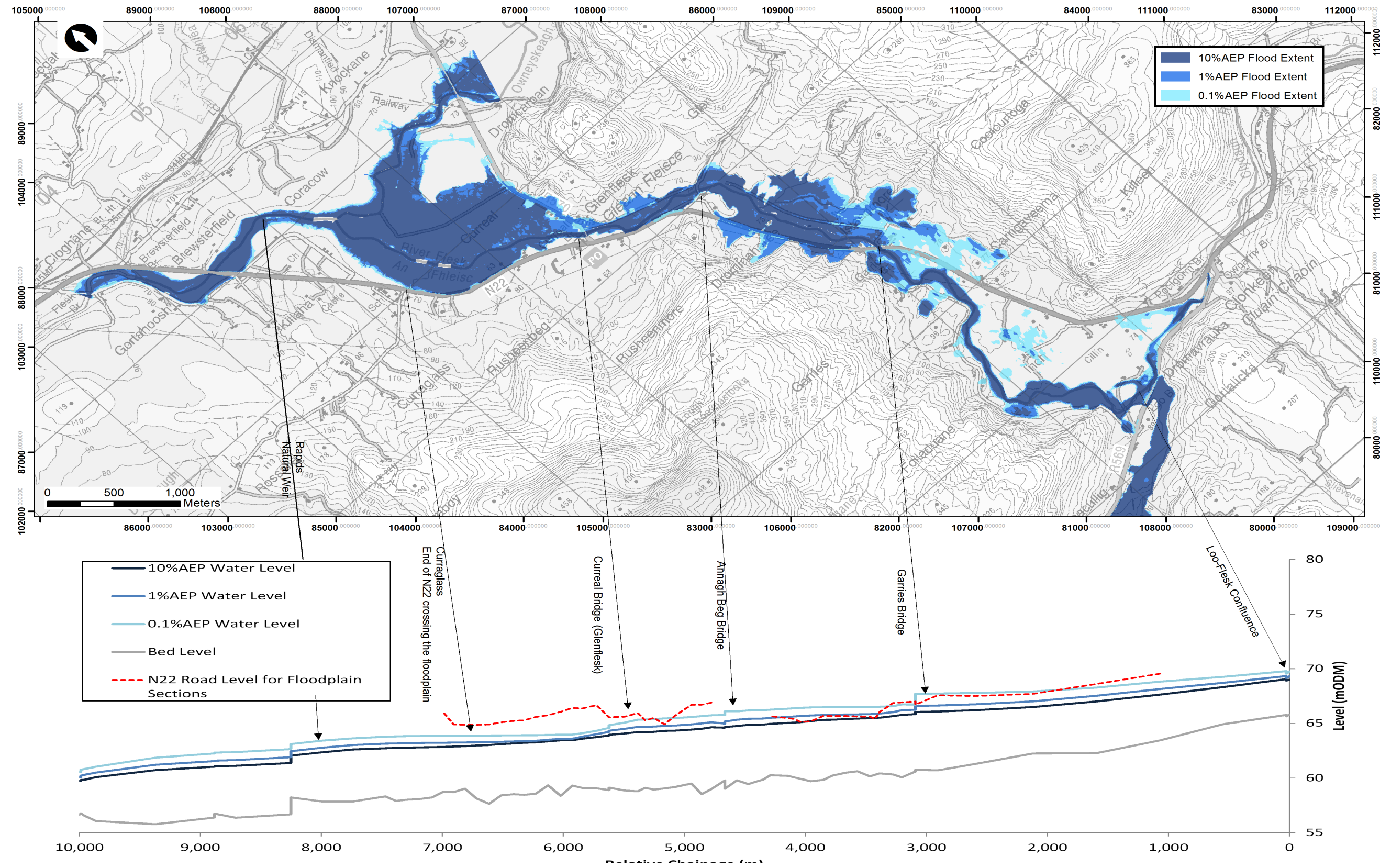
The areas flooded are consistent with the recurring flooding along the N22, which is reported almost every year by local authority staff. The sensitivity test demonstrated the uncertainty in flow estimates and roughness which increased water levels but did not significantly increase flood extent and hazard, as the narrow valley is largely flooded even in the 50%AEP event. Therefore there is reasonable confidence in the flood mapping.

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

- Improved conveyance at the rapids downstream of Glenflesk would control water levels upstream.
- Increased storage on the floodplain and/or raising of the road embankments may reduce flood risk upstream of Glenflesk.

- Improved conveyance at and around Loo Bridge and the flood relief culvert could reduce flood levels upstream of Loo Bridge.
- Flood warning is possible for the receptors at risk given the relatively long lead time and gauges in the upper catchments of the Clydagh and Owneyskeagh Rivers.

Map 9.3: Summary of Fluvial Flood Risk- Glenflesk



9.5 Killarney AFA

Map 9.4 summarises the fluvial flood risk in Killarney AFA for the 10%, 1% and 0.1%AEP design scenarios. The key flow routes and flooding mechanisms predicted by the model are as follows:

- Backing up from Lough Leane inundating the alluvial forests in the National Park.
- Backing up from White Bridge (Flesk) to bypass on the left and right banks, flooding Ballycasheen and Mill Road.
- Backing up from Flesk Bridge to flood the right bank at Muckcross Grove before flowing along Muckcross Road and towards the Deenagh in extreme events only.
- Backing up from the roundabout and Deenagh Lodge Bridge to overtop the right bank and the N22 in extreme events.

The key thresholds and areas affected by flooding are:

- 50%AEP fluvial event floods the National Park downstream of the town on both the Deenagh and Flesk.
- 1%AEP fluvial event bypasses White Bridge (Flesk) to flood Ballycasheen and Mill Road. Properties affected in the 0.1%AEP event only.
- 0.1%AEP fluvial event overtops the right bank upstream of Flesk Bridge.
- 2%AEP fluvial event overtops left bank upstream of the roundabout and Deenagh Lodge Bridge.

The greatest risk to life is associated with deep and fast flow water on the right bank between White Bridge (Flesk) and Flesk Bridge, but this does not affect properties or roads in the 1%AEP event. In the 0.1%AEP event, risk to life is classed as significant to flooded properties along Muckcross, Muckcross Road and Ballycasheen Road due to the velocity of water.

The critical structures in determining fluvial flood risk include:

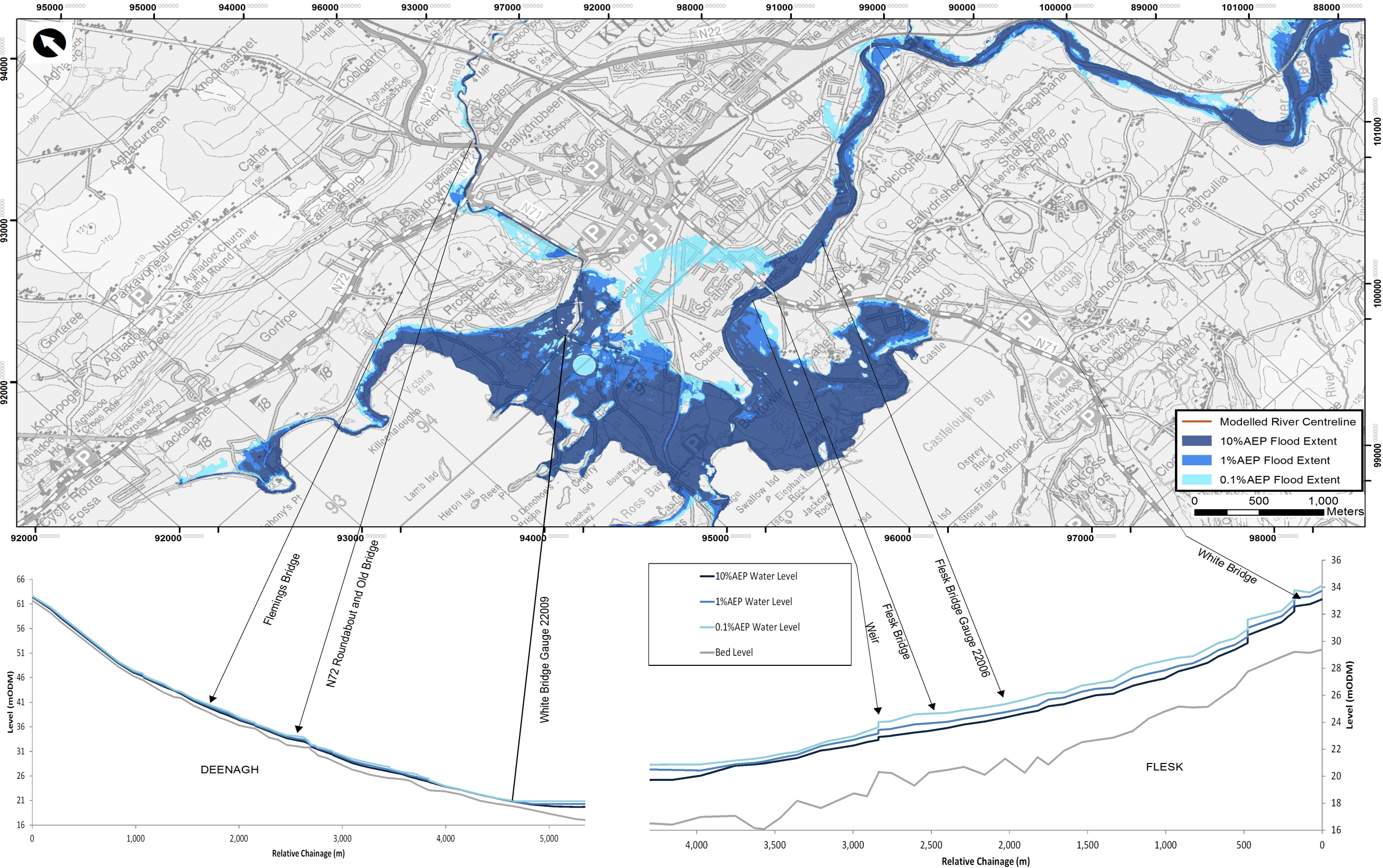
- The culvert under the Ballydowney roundabout and old bridge crossing immediately downstream on the Deenagh.
- Deenagh Lodge Bridge for flooding to Port Road.
- White Bridge on the Flesk for properties along Mill Road and Ballycasheen Road in extreme events.
- Flesk Bridge and the weir downstream but only in the most extreme events.

The areas flooded are consistent with the lakeside property flooding reported in 2009 (See Section 5.1.3) and limited property flooding upstream of Flesk Bridge and Deenagh Lodge. Therefore there is reasonable confidence in the flood mapping.

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

- Improved conveyance along Port Road, focussing on the critical structures at the roundabout and Deenagh Lodge.
- Flood Warning from Lough Leane and the Flesk is possible given the long lead times to the flood peak and active gauges upstream on the Flesk.
- There is a shorter lead time on the Deenagh but flood warning should still be possible subject to a high flows gauge review at White Bridge (Deenagh).

Map 9.4: Summary of Fluvial Flood Risk - Killarney



9.6 Dingle AFA

Map 9.5 summarises the fluvial flood risk in Dingle AFA for the 10%, 1% and 0.1%AEP design scenarios. The key flow routes and fluvial flood mechanisms predicted by the model are as follows:

- Overtopping of the right bank along Spa Road due to flows exceeding the capacity of bridges and culverts although flooding is very shallow ($< 0.1\text{m}$) for smaller events.
- Overtopping of both banks at the low spots of Lana na h'Abhann and Hudson's Bridge causing flooding of The Mall and Bridge Street
- Backing up from Milltown Bridge to overtop the bridge on the right bank and the R459 on the left bank near the junction.

The key fluvial thresholds and areas affected by flooding are:

- 10%AEP Fluvial Current Scenario exceeds the capacity of Dingle Stream to cause very shallow flooding ($< 0.1\text{m}$) along Spa Road and the Mall.
- 2%AEP Fluvial Current Scenario on Milltown Stream overtops Milltown Bridge on the right bank and the R459 on the left bank but does not affect properties.
- 1% - 0.5%AEP Fluvial Current Scenario floods the Library site on Milltown Stream.

The key flow routes and coastal flood mechanisms predicted by the model are as follows:

- Dingle Stream is tidally influenced downstream of the weir near Hudson's Bridge.
- Milltown Stream is tidally influenced downstream of the Ballymoretreagh confluence.
- Overtopping of the road at The Woods but does not affect properties.
- Overtopping of the left bank downstream of the roundabout on Dingle Stream, but does not affect properties.
- Overtopping of Milltown Bridge and the quayside at Strand Street in more extreme events.

The key coastal thresholds and areas affected by flooding are:

- 20%AEP Coastal Current Scenario overtops the road at The Woods, but does not affect properties.
- 10%AEP Coastal Current Scenario overtops the left bank of Dingle Stream downstream of the roundabout but does not affect properties.
- 2%AEP Coastal Current Scenario overtops around Milltown Bridge and the quayside at Strand Street, but does not affect properties.
- 1%AEP Coastal Current Scenario affects properties on Strand Street from overtopping of the quay and flooding upstream of Bridge Street.
- Approximately 20 properties are effected by the 0.5%AEP Coastal Current Scenario.

The greatest risk to life is associated with deep water at Bridge Street and fast flowing water down Spa Road and The Mall, which is classed as significant in the 10%AEP event and greater magnitude events.

The critical structures in determining fluvial flood risk include:

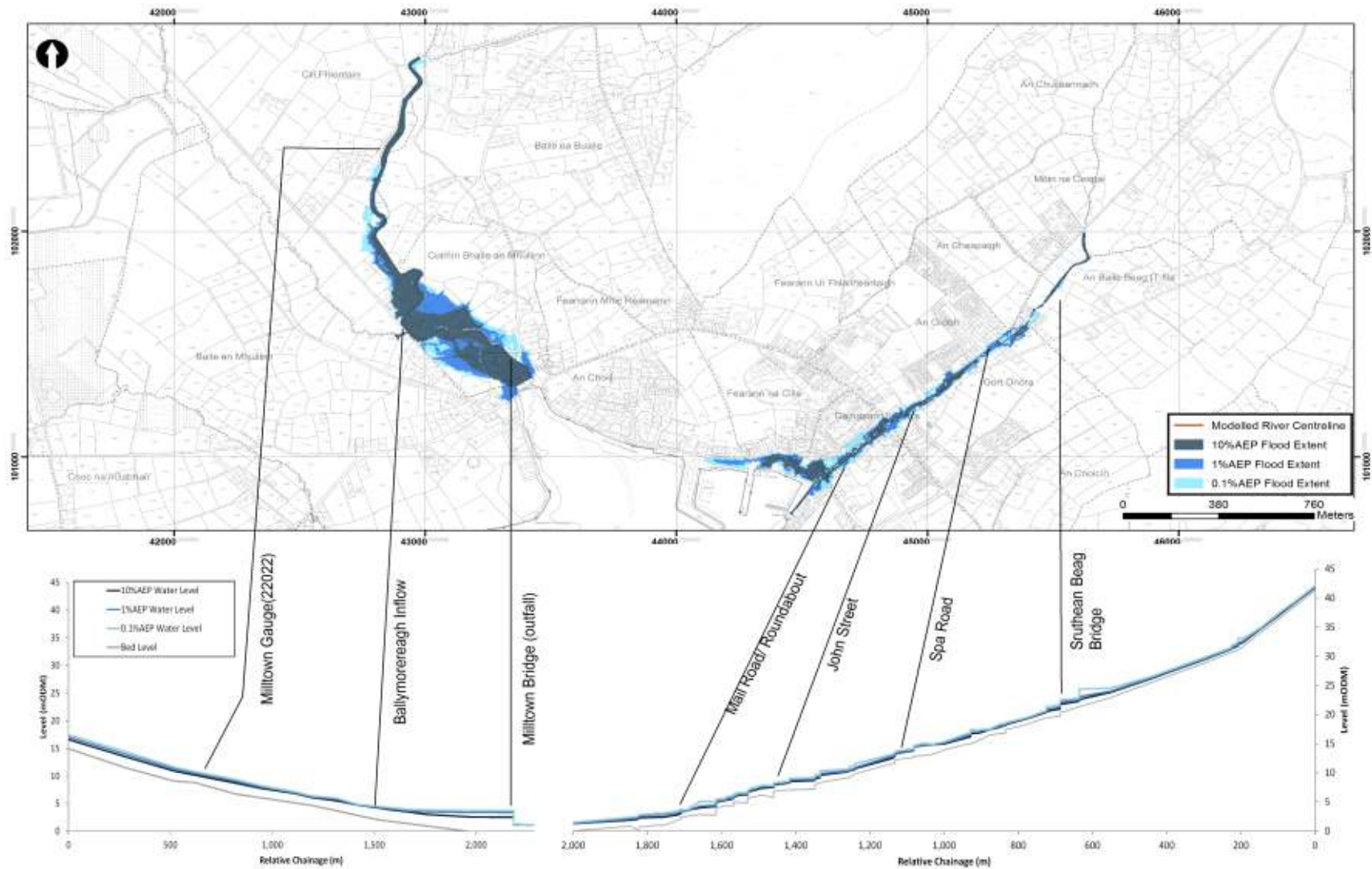
- On Dingle Stream:
 - Access Bridge and river bend downstream of Sruthean Beag estate
 - Brewery Access Bridges
 - Spa Road culvert
 - Lana na hAbhann
 - Hudson's Bridge
 - Bridge Street culvert
- On Milltown Stream:
 - Milltown Bridge

The areas flooded are consistent with those photographed in the January 1988 event and the recurring flood reports on floodmaps.ie. Therefore there is reasonable confidence in the flood mapping.

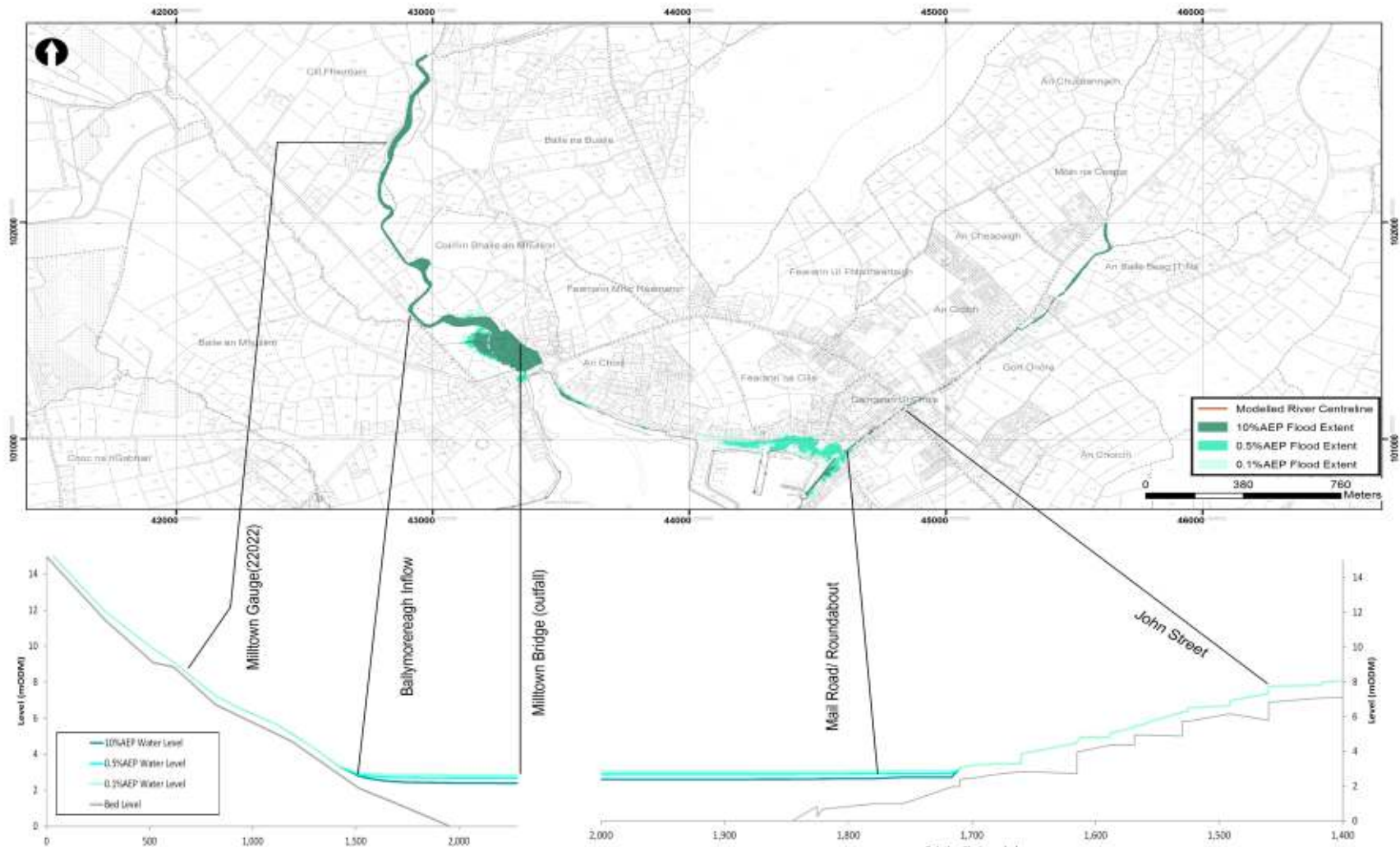
The following recommendations for flood risk management option development can be made:

- Increased conveyance measures should be considered for the critical structures identified above.
- There is limited storage available upstream of the AFA on Dingle Stream to enable any storage or attenuation measures.
- Flood Warning is not likely to be feasible on Dingle Stream given that the time to peak is less than 5 hours.

Map 9.5: Summary of Fluvial Flood Risk in Dingle



Map 9.6: Summary of Coastal Risk in Dingle



9.7 Portmagee AFA

Map 9.6 summarises the coastal flood risk in Portmagee AFA 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:

- Overtopping of the quayside at the Car Park and behind the Restaurant, but no property flooding under current conditions.
- Minor overtopping at The Old School spillway but no property flooding under current conditions.
- Overtopping of the R565 to the east but this does not affect flood risk in the AFA.

The key thresholds and areas affected by flooding are:

- 10%AEP overtops the R565 to the east of the AFA.
- 10%AEP overtops The Old School slipway but no property flooding under current conditions.
- 0.5%AEP overtops the quayside but no property flooding under current conditions.

Risk to life is classified as low within the AFA for the 0.5%AEP coastal event, increasing to moderate in the 0.1%AEP coastal event under current and future climate conditions.

The critical structures in determining fluvial flood risk include:

- The quayside wall/car park level.

It should be noted that the R565 levels are based on IFSAR data rather than LIDAR, so the road level is only accurate to +/- 0.7m. However levels in the AFA are based on LIDAR data and are deemed to be accurate to +/- 0.2m. There are no reports of flooding at this AFA. Therefore there is reasonable confidence in the flood mapping within the AFA.

There is very low coastal flood risk to receptors within the AFA under current conditions. This risk could increase marginally to affect less than 5 quayside properties under the HEFS scenario. The following recommendations for flood risk management option development can be made for the HEFS:

- Raised quayside wall levels and individual property protection.
- Flood Warning is possible given the long lead times of storm surge events and known astronomical tides.
- Flood storage is not applicable for coastal flooding because the flooding is not volume dependent on the open coast.

Map 9.7: Coastal Flood Risk in Portmagee



10 Summary and Recommendations

10.1 Key Findings

The hydraulic analysis for UoM22 has developed eight hydraulic models to assess current and future flood risk from the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP fluvial and tidal flood events. The design flood levels and flows were then processed to map flood extent, flood depth, flood velocity and flood hazard in the six AFAs and flood extent and depth maps for MPW reaches downstream.

Historic flood events

- The Castleisland model matched well with the gauged and flood report information for the 4th October 2008 event. The design model outline was also validated with locations which are known to flood.
- The Killarney model tended to overestimate water level by 0.1 to 0.2m following calibration of the weir coefficient downstream. However the flood extent matched well with recorded flooding in both events.
- The in-bank calibration on the Maine indicated good performance of the model at Riverville and Castlemaine. However, concurrent gauge information for flood events was limited to calibrate the model further.
- Validation of the design Glenflesk, Milltown and Dingle models against historic flood reports of recurring flooding also indicated the models were predicted the correct areas at risk.

Sensitivity test results

- Dingle and Maine models are sensitive to the assumptions and uncertainties in the extreme sea levels.
- Seasonal changes in vegetation or uncertainty in roughness values only increased flooding in Castleisland and to the N22 in Glenflesk at the 1% AEP. However, maintenance of the channel may provide some benefit for events which are closer to the threshold of flooding.
- The flood risk in Milltown was not deemed sensitive to the culvert coefficients applied at the 1% AEP fluvial event. However, increasing coefficients did reduce the culvert capacity and cause flooding earlier in the event. Therefore, the effective capacity of the culverts and any blockage should be carefully considered when interpreting flood maps and deriving flood risk management options to reduce flooding in more frequent event.

Model and mapping results

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

- The greatest fluvial flood risk to properties and infrastructure in UoM22 is located in Castleisland where flooding of properties starts at the 50%AEP in Tullig and the 5% to 2%AEP through the town itself.
- There is regular flooding in the Milltown AFA from and the N22 along the Flesk is affected by the 50%AEP and larger events.
- There is also extensive flooding of agricultural and pastoral land along the Maine and Lower Laune from the 5%AEP fluvial current event and 50%AEP coastal current event.
- Regular fluvial flooding was predicted Killarney AFA but this was contained to the areas around White Bridge and Killarney National Park. However, properties along Muckross Road were not affected until the 0.1%AEP.

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 Castleisland Lower Maine, Killarney, Milltown and Glenflesk.
- 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 Dingle and the Lower Maine and Laune.
- The effective capacity of the culverts in Milltown and any blockage should be carefully considered when interpreting flood maps and deriving flood risk management options to reduce flooding in more frequent events.
- The crest level of the Lower Maine embankments is quite variable. Therefore, infilling works (temporary or permanent) of the low points should be considered to reduce overtopping and protect the integrity of the defence.

The following recommendations can be drawn from the hydraulic analysis for future analysis in UoM22:

- 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 additional calibration be undertaken when concurrent gauge information is available in the Flesk catchment and Maine catchment.

Glossary

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

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