

Western CFRAM Unit of Management 30 and 31 - Corrib and Owengowla

Final Hydraulic Modelling Report

September 2015

**Office of Public Works
Trim
Co. Meath**



**WESTERN
CFRAM
STUDY**
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT



JBA Consulting

24 Grove Island
Corbally
Limerick
Ireland

JBA Project Manager

Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM

Revision History

Revision Ref / Date Issued	Amendments	Issued to
Revision 1 / 04/09/2015	Initial Issue	Richael Duffy, OPW
Revision 2 / 11/09/2015	Final Issue	Richael Duffy, OPW

Contract

This report describes work commissioned by The Office of Public Works, by a letter dated (28/07/11). The Office of Public Works' representative for the contract was Clare Butler. Sam Willis, Elizabeth Russell, Chris Smith and Tom Sampson of JBA Consulting carried out this work.

Prepared Elizabeth Russell BSc MSc CEnv MCIWEM
C.WEM

Reviewed by Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM

Purpose

This document has been prepared as a draft report for The Office of Public Works. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to the Office of Public Works.

Copyright

Copyright – Copyright is with Office of Public Works. All rights reserved. No part of this report may be copied or reproduced by any means without the prior written permission of the Office of Public works.

Legal Disclaimer

This report is subject to the limitations and warranties contained in the contract between the commissioning party (Office of Public Works) and JBA.

Carbon Footprint

A printed copy of the main text in this document will result in a carbon footprint of 206g if 100% post-consumer recycled paper is used and 262g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to reduce its per capita carbon emissions.

Contents

1	Introduction.....	1
1.1	Scope of report	1
1.2	Report overview	2
1.3	Study background.....	3
2	Ballyhaunis.....	4
2.1	Watercourse and catchment overview	4
2.2	Flood history	4
2.3	Existing defences and walls	5
2.4	Model limitations	5
2.5	Key flood risk mechanisms	5
2.6	Sensitivity testing	7
2.7	Flood risk summary	9
3	Corrofin.....	10
3.1	Watercourse and catchment overview	10
3.2	Flood history	11
3.3	Existing defences and walls	11
3.4	Model limitations	12
3.5	Key flood risk mechanisms	12
3.6	Sensitivity testing	12
3.7	Flood risk summary	15
4	Galway City	16
4.1	Watercourse and catchment overview	16
4.2	Flood history	17
4.3	Existing defences and walls	18
4.4	Model limitations	18
4.5	Key fluvial flood risk mechanisms.....	19
4.6	Key tidal flood risk mechanisms	20
4.7	Sensitivity testing	20
4.8	Flood risk summary	23
5	Oughterard	25
5.1	Watercourse and catchment overview	25
5.2	Flood history	26
5.3	Existing defences and walls	26
5.4	Model limitations	27
5.5	Key flood risk mechanisms	27
5.6	Sensitivity testing	28
5.7	Flood risk summary	30
6	Tuam	31
6.1	Watercourse and catchment overview	31
6.2	Flood history	31
6.3	Existing defences and walls	31
6.4	Model limitations	32
6.5	Key flood risk mechanisms	32
6.6	Sensitivity testing	32
6.7	Flood risk summary	35
7	Roundstone.....	36
7.1	Watercourse and catchment overview	36
7.2	Flood history	36

7.3	Existing defences and walls	37
7.4	Model limitations	37
7.5	Key flood risk mechanisms	37
7.6	Sensitivity testing	37
7.7	Flood risk summary	39
8	UoM Summary.....	40

List of Figures

Figure 1-1: AFAs within UoM 30 and 31	1
Figure 2-1: Ballyhaunis AFA catchment overview	4
Figure 2-2: 1% AEP event uncertainty bounds - Ballyhaunis AFA.....	8
Figure 3-1: Overview of rivers in the Corrofin catchment	10
Figure 3-2: Overview of defences in Corrofin	11
Figure 3-3: 1% AEP event uncertainty bounds - Corrofin AFA	13
Figure 3-4: 1% AEP event uncertainty bounds - Clare River	14
Figure 4-1: Galway City AFA catchment overview	16
Figure 4-2: Galway coastal overview.....	17
Figure 4-3: Overview of defences in Galway City.....	18
Figure 4-4: 1% AEP fluvial event uncertainty bounds - Claddagh Area.....	21
Figure 4-5: 1% AEP fluvial event uncertainty bounds - Upper Corrib	21
Figure 4-6: 0.5% AEP tidal event uncertainty bounds - Salthill West.....	22
Figure 4-7: 0.5% AEP tidal event uncertainty bounds - Salthill East.....	22
Figure 5-1: Oughterard AFA catchment overview	25
Figure 5-2: Defences and walls locations	27
Figure 5-3: 1% AEP event uncertainty bounds - Oughterard	29
Figure 6-1: Tuam AFA catchment overview	31
Figure 6-2: 1% AEP fluvial uncertainty bound for the River Clare - Tuam AFA.....	33
Figure 6-3: 1% AEP fluvial uncertainty bound for the River Nanny - Tuam AFA	33
Figure 6-4: 1% AEP fluvial uncertainty bound for the River Suileen - Tuam AFA.....	34
Figure 7-1: Roundstone AFA catchment overview	36
Figure 7-2: Roundstone Quay Wall	37
Figure 7-3: 0.5% AEP tidal event uncertainty bounds - Roundstone AFA	38

List of Tables

Table 1-1: AFA report and model codes.....	2
Table 2-1: Flood history summary in Ballyhaunis.....	4
Table 2-2 - Wrack marks near Ballyhaunis gauge	6
Table 2-3 - Curries watercourse	6
Table 2-4: Flood risk to receptors in Ballyhaunis.....	9
Table 3-1: Flood history summary in Corrofin	11

Table 3-2: Corrofin flood mechanism	12
Table 3-3: Flood risk receptors in Corrofin	15
Table 4-1: Flood history summary in Galway City	17
Table 4-2: Fluvial flood risk to receptors in Galway city	23
Table 4-3: Tidal flood risk to receptors in Galway city	23
Table 5-1: Owenriff River gradient changes	26
Table 5-2 : Flood history summary in Oughterard.....	26
Table 5-3: Flood risk to receptors in Oughterard.....	30
Table 6-1: Flood risk to receptors in Tuam.....	35
Table 7-1: Flood history summary in Roundstone.....	36
Table 7-2: Flood risk to receptors in Roundstone.....	39
Table 8-1: Summary of flood risk to AFAs.....	40

Abbreviations

AEP	Annual exceedence probability
AFA	Area for further assessment
AMAX.....	Annual maximum
CFRAM	Catchment flood risk assessment and management
DAD	Defence asset database
DAS.....	Defence asset survey
DEM	Digital elevation model (Includes surfaces of structures, vegetation, etc)
DTM	Digital terrain model ('bare earth' model; does not include surfaces of structures, vegetation, etc
ESTRY	One-dimensional model from the TUFLOW suite
FRISM.....	Flood risk metrics (a flood risk tool developed by JBA)
FRMP	Flood risk management plan
FRR.....	Flood risk review
FSR	Flood studies report
FSU	Flood studies update
GIS.....	Geographical information system
HEFS	High-end future scenario
HEP.....	Hydrological estimation point
HPW.....	High priority watercourse
HWA.....	Hydrograph width analysis
IBIDEM.....	Interactive bridge invoking the design event method
ICPSS	Irish coastal protection strategy study
ISIS	One-dimensional hydraulic modelling software
LA.....	Local authority
LIDAR	Light detection and ranging
mOD.....	Metres above Ordnance datum (unless stated this refers to the Malin datum)
MPW	Medium priority watercourse
MRFS.....	Mid-range future scenario
NDHM	National digital height model (a DTM by Intermap)
OSi.....	Ordnance Survey Ireland
PFRA	Preliminary flood risk assessment
Q(T).....	Flow for a given return period
QMED	Median annual flood, used in FSU methods
SAAR	Standard annual average rainfall
SoP	Standard of protection (in relation to flood defences)
T	Return period, inverse of AEP
Tp.....	Time to peak

TUFLOW..... Two-dimensional hydraulic modelling software

UoM Unit of Management

* Asterisks at the end of a cross section label denotes interpolated
model cross sections

1 Introduction

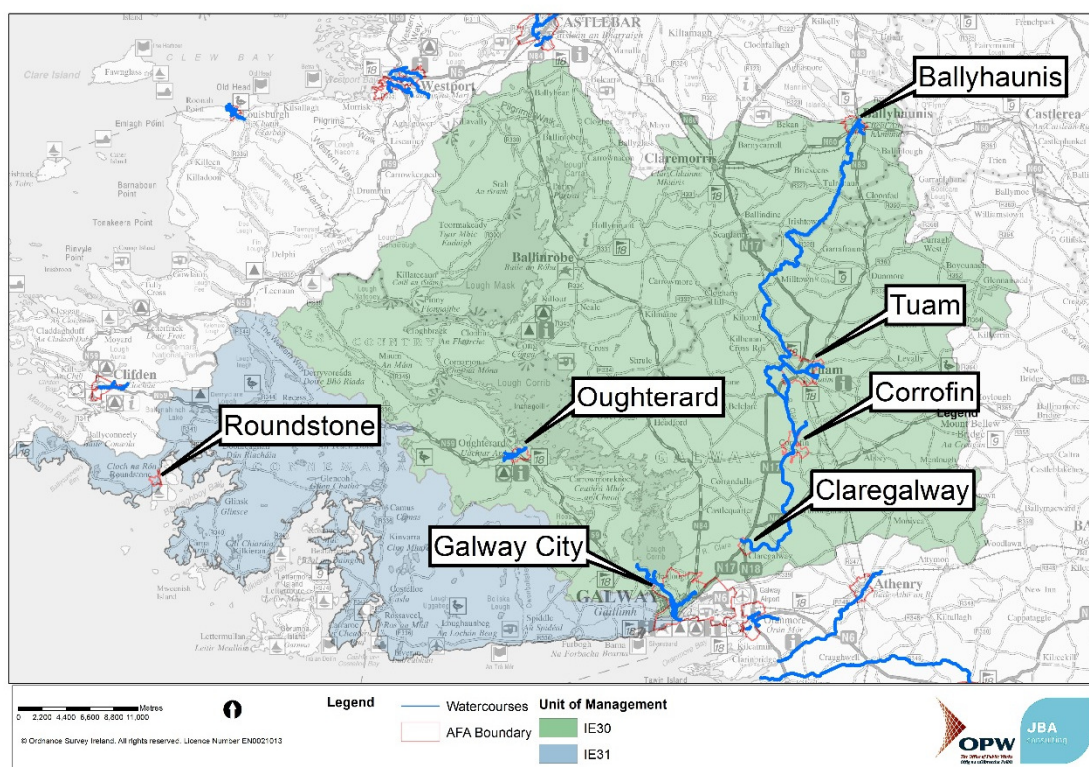
1.1 Scope of report

This report provides an overview of the findings of the modelling phase of the Western Catchment-Based Flood Risk Assessment and Management Study (WCFRAM). The report covers the Areas for Further Assessment (AFAs) within Unit of Management (UoM) 30 and 31, as shown in Figure 1-1:

- Ballyhaunis
- Claregalway
- Corrofin
- Galway City
- Oughterard
- Tuam
- Roundstone

This report is not intended to provide detail in relation to the hydrological assessment or modelling approaches used in any specific location, both of which are detailed in supporting technical reports, as detailed in Section 1.2.

Figure 1-1: AFAs within UoM 30 and 31



This report summarises the main sources of flood risk within each AFA, including details of the watercourses, historical flooding and flood defences. Where limitations in the modelling carried out have been identified they have also been summarised, with further detail provided in the relevant AFA report. An indication of the sensitivity to various parameters, such as increases in flow, changes in channel roughness and the representation of buildings and structures is also provided. For each AFA the main areas of flood risk, and the associated pathways to flooding, are discussed.

A summary of flood risk in each AFA has been included at the end of each section. This is in the form of a count of the number of receptors (for example, residential property, schools or lengths of motorway) which are at risk of flooding in 10%, 1%/0.5% and 0.1% AEP flood extents for fluvial

and coastal scenarios, for both the existing risk (present day), and medium range future scenario (MRFS).

Finally, a summary of risk to the whole Unit of Management is provided in Section 8.

This report does not include a summary of the medium priority watercourses, but each model is supported by its own Report, which forms part of Volume 2 of this series.

1.2 Report overview

This report is one of a series which describe the work undertaken as part of the CFRAM, and together they provide a description of the approach taken to identifying flood risk, and a discussion of the results of the analysis and potential flood management measures, where they are appropriate.

This report should be read in conjunction with the following supporting documents:

- Western CFRAM UoM30-31 Hydrology Report¹
- Western CFRAM UoM30-31 Inception Report²
- Western CFRAM Flood Risk Review Report³
- Western CFRAM SEA Scoping Report⁴
- Western CFRAM Defence Asset Database: Handover Report and accompanying database files⁵

The reports in the suite for the Hydraulic Modelling are:

- Western CFRAM UoM30-31 - Hydraulic Modelling Report (this report)
- Western CFRAM UoM30-31 - Hydraulic Modelling Report: Volume 1a – Hydraulic Modelling Method Statement
- Western CFRAM UoM30-31 - Hydraulic Modelling Report: Volume 1b – Hydromorphology and Coastal Erosion Assessment
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 2 – AFA Modelling Report (for example – 2a - Ballyhaunis AFA Modelling Report)
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 3 – Flood Risk Maps (for example - 3a Ballyhaunis Flood Risk Maps)
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 4a – Hydraulic Model and Check File (for example - 4a Ballyhaunis Hydraulic Model and Check File)

The letter code associated with the deliverables in Volumes 2, 3 and 4 will be consistent for a given AFA, so in the example above the letter 'a' applies to the Ballyhaunis AFA. Volume 4 is the technical output from the study and will only be available on request from the Office of Public Works.

The report and model codes for UoM 30 and 31 are provided in Table 1-1.

Table 1-1: AFA report and model codes

AFA	AFA / MPW code	Model code	Report code	Model type
Ballyhaunis	BLH	D1	a	Fluvial
Ballyhaunis to Tuam	MWB	92	h	MPW
Corrofin	CRF	L1	b	Fluvial
Galway City	GLW	N1	c	Fluvial
Galway City	GLW	C1	d	Coastal (Wave

¹ JBA Consulting (2014), Western CFRAM Unit of Management 30 and 31 – Corrib and Owengowla Hydrology Report, Final Report, Office of Public Works

² JBA Consulting (2012), Western CFRAM Unit of Management 30-31 – Corrib and Owengowla Inception Report, Final Report, Office of Public Works.

³ JBA Consulting (2012), Western CFRAM Flood Risk Review, Final Report, Office of Public Works.

⁴ JBA Consulting (2013), Western River Basin District Catchment-based Flood Risk Assessment and Management (CFRAM) Strategic Environmental Assessment, Scoping Report, Office of Public Works.

⁵ JBA Consulting (2013), Western CFRAM Defence Asset Database, Handover Report, Office of Public Works.

AFA	AFA / MPW code	Model code	Report code	Model type
Coastal				Overtopping)
Oughterard	OTD	U1	e	Fluvial
Tuam	TUM	Y1	g	Fluvial
Tuam to Lough Corrib	MWT	90	i	MPW
Roundstone Coastal	RSN	C4	f	Coastal

1.3 Study background

The Inception Report for UoM 30 and 31 was delivered in October 2012. This report consisted of a baseline review of available data and the development of the proposed methodology for the hydrological and hydraulic modelling investigations to be completed within this phase.

The method statement for the hydrological analysis detailed in the Inception Report has been developed and finalised in the UoM 30 and 31 Hydrology Report. This work has developed design flows at a series of Hydrological Estimation Points (HEPs) along all watercourses to be modelled. The detailed development of the hydrology has not been reiterated here and the reader is referred to the Hydrology Report for full details of the hydrological analysis. Design flows have been extracted directly from the Hydrology report and are summarised in the various editions of Volume 2, relating to the specific AFAs.

The Hydrology Report also provides guidance on the development of appropriate design storm hydrographs for each AFA for the purposes of the hydraulic modelling. These methods are summarised in this report to provide clarity on the application of the design event hydrology as this work has been undertaken in the hydraulic modelling phase.

The Inception Report identified all High Priority and Medium Priority Watercourses (HPWs and MPWs) to be modelled. HPWs are those watercourses that dictate flood risk within an AFA boundary as originally delineated within the Flood Risk Review (FRR) Report. HPWs therefore extend a short distance upstream and downstream of an AFA but do not include watercourses with catchments less than 1km². HPWs have been modelled to a greater level of detail than MPWs. MPWs are the watercourses which link two AFAs together and the watercourses that extend downstream of an AFA to the sea. Coastal AFAs do not have a downstream MPW associated with them.

In total, 56 km of HPW and 102 km of MPW have been modelled within UoM 30 and 31, along with the coastline in Roundstone and Galway City.

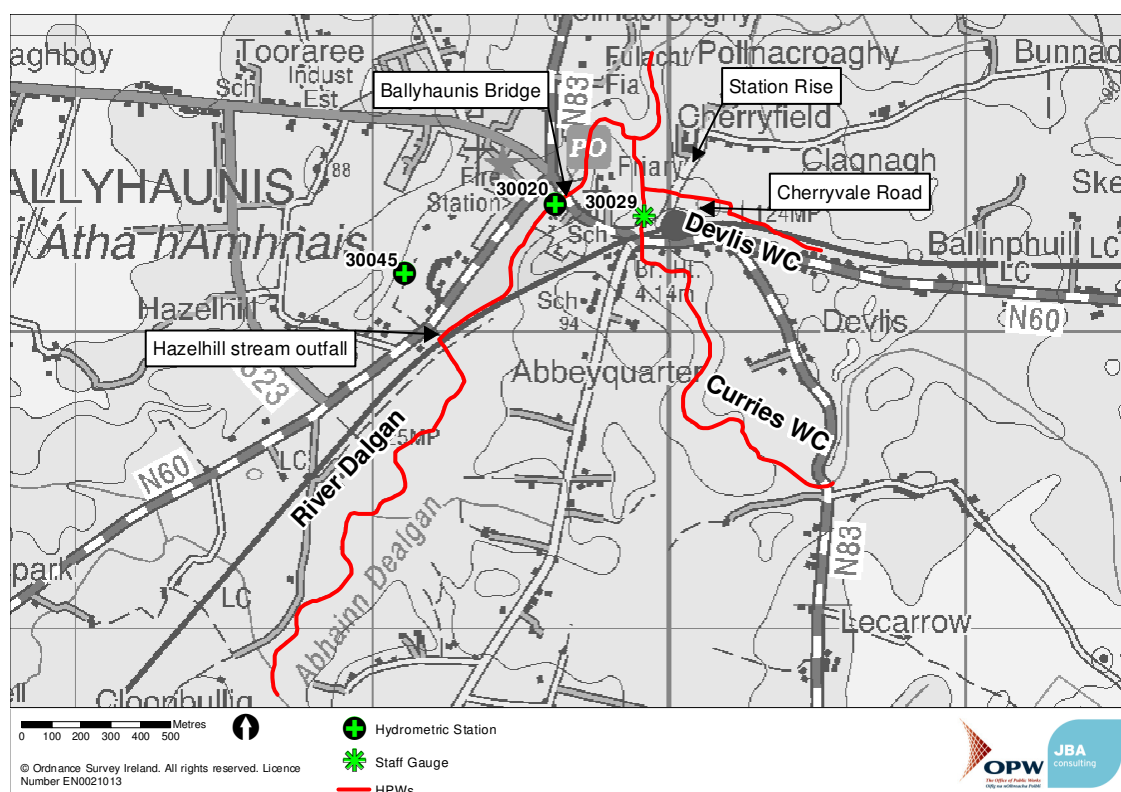
2 Ballyhaunis

2.1 Watercourse and catchment overview

The study area encompasses the Ballyhaunis AFA and includes the River Dalgan, which is the main river passing through Ballyhaunis town centre, Curries Watercourse, a tributary of the River Dalgan, and Devils Watercourse, a tributary of Curries Watercourse. The watercourses are all classed as High Priority Watercourses (HPWs) as they flow through the core of the AFA, and have been included in the model. The main hydraulic structure within the Ballyhaunis is the N60 Road Bridge, or Ballyhaunis Bridge. A gauging station is found shortly downstream of Ballyhaunis Bridge, where two weirs control river levels. Figure 2-1 provides an overview of the area.

There is an additional groundwater fed stream, called Hazelhill, discharging into the Dalgan downstream of the town centre. This stream has not been modelled.

Figure 2-1: Ballyhaunis AFA catchment overview



2.2 Flood history

Further details on the flood history of the town can be found in the AFA specific modelling report and the Hydrology and Inception Reports. A summary of known flood events is provided below.

Table 2-1: Flood history summary in Ballyhaunis

Area affected	Main flood mechanisms	Recorded flood event date	Use in model check
Some out of bank flow through the town centre and at the Dawn Meats Plant. No property flooded.	River Dalgan	Nov 2009	Limited verification using observed extents and flow record.
Right bank, downstream of Ballyhaunis bridge	River Dalgan	Nov 1999	Not used
Prolonged high water levels across the catchment	River Dalgan, but linked to pre-arterial drainage works conditions	1968-69	Not used

2.3 Existing defences and walls

No formal or informal effective defences were identified within the AFA. Six informal ineffective structures (i.e. walls or embankments with gaps in) were identified within Ballyhaunis along the banks of the River Dalgan and two along the banks of the Curries Watercourse. These structures are not assumed to function as flood defences and are either bypassed in the model or have been removed to allow flooding beyond them.

2.4 Model limitations

- Hazelhill has not been modelled. It is a groundwater fed stream and groundwater modelling is beyond the scope of the WCFRAM study. The additional inflow to the River Dalgan from Hazelhill Stream has not been included in the model. There was not enough data from the gauge located on this spring (30045) to allow any analysis of flows to be undertaken, and communication from the EPA indicates that the inflows were only significant at low flows.
- Channel blockage presents a higher level of risk, especially at section 30DEVL000111, below Station Rise - Detailed investigations of blockage and debris build-up have not been undertaken within the scope of the CFRAM. Although not investigated in more detail in this model, it is unlikely that culvert blockage in this location will increase flood risk to property.
- Geomorphological changes - Gravel and silt deposition on the Curries watercourse between the N60 and railway embankment crossing may affect the flood risk, particularly in the future, and could impact on the operation of potential flood management measures.

2.5 Key flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below. In general, the flood extents produced are less than might be expected from a fully natural series of watercourses but there is evidence of man-made intervention on all three of the modelled watercourses so the natural regime is no longer fully in place. The result of these changes has been increased channel capacity, and a change from the natural flow regimes and resulting overtopping patterns. This fact, coupled with the records of two recent extreme events, give confidence that the under-prediction of flood extents is realistic.

2.5.1 Flooding upstream of Ballyhaunis Bridge on the River Dalgan.

Immediately upstream of Ballyhaunis Bridge, a small and confined amount of flooding is predicted to the rear of Delaney's Garden Centre, in the car park / yard area. This is predicted to occur from the 10% AEP event upwards, but no properties are at risk. Water levels in this section of river are dictated by the capacity of the channel, which is a reflection of the fact that even in large events floodplain flow is limited. The modelling has shown that Ballyhaunis Bridge is not a significant control structure in this location, with head losses associated with the structure less than 0.1m in the 1% AEP event.

The River Dalgan is predicted to flood the fields found a short distance upstream of Ballyhaunis Bridge from the 10% AEP event onwards. No properties are at risk as a result.

2.5.2 Flooding downstream of Ballyhaunis gauge

Model results show the River Dalgan exceeds bank top level upstream of the control weir of the Ballyhaunis gauge as a result of the limited channel capacity as frequently as the 10% AEP event (see Table 2-2). The floodplain is well confined around the gauging station itself and this results in no flooding of properties.

Approximately 80m downstream of the gauge, the 1% AEP event is predicted to cause flooding to Donnellan's Joinery on Clare Road and to the rear of neighbouring properties on Clare Road. This flooding can be attributed to a localised drop in the bank levels allowing flows to discharge onto the floodplain.

Table 2-2 - Wrack marks near Ballyhaunis gauge

Wrack marks clearly visible on the wire mesh fencing downstream of the Ballyhaunis gauge, on the right bank. The photograph was taken in June 2013, supporting the relatively high frequency of inundation of footpath.



2.5.3 Flooding at Dawn Meats Plant

Out of bank flow is predicted as frequently as the 10% AEP event upstream of the culvert adjacent to Dawn Meats Plant. However, it is not until the 0.1% AEP event that the river level upstream of the culvert is great enough to cause the river to bypass the culvert. Flood levels are shown to reach Dawn Meats Plant and also Keane Kitchens Ltd upstream in the 0.1% AEP event, but not to flood these properties.

2.5.4 Flooding downstream of railway bridge

The greatest extent of flooding within the Ballyhaunis AFA occurs in the fields downstream of the town; this is the area where drainage works are thought to have taken place. Under regular flow conditions the watercourse is extremely slow flowing, with a wide flat floodplain, and in extreme events it is unlikely water would discharge downstream very quickly; instead it is likely to spread across the floodplain. Downstream of the railway bridge on the River Dalgan, near Clare Road, flooding is predicted from the 10% AEP event onwards, also resulting in flooding to Hazelhill Road. No properties are at risk as a result of this flooding.

2.5.5 Curries Watercourse

The Curries watercourse is shown to present no risk of flooding to properties in Ballyhaunis up to the extreme 0.1% AEP event. Examination of the water level profile through the key culverts at the N60 and the railway bridge shows some head loss through both culverts. Despite this, the water remains in bank. In the Q1000 event, water levels upstream of the N60 are close to the top of the right bank, but if water were to overtop at this location it would be prevented from flowing onto the road by the field boundary wall.

Table 2-3 - Curries watercourse

Looking upstream from the N60. If flood waters were to overtop the left bank, they would be retained by the site boundary wall. However, current design events (to the 0.1%AEP) show water remains in bank.



View from the culvert over the Curries River, looking towards the dip below the railway line. The site shown in the above photo is to the left.



A potential risk of flooding to the N60 as it dips below the railway bridge has been identified. However, as discussed above, it has been shown that the Curries watercourse does not overtop the bank in the current scenario. If the capacity of the culverts were to be reduced or flows to increase (such as in a future scenario) overtopping may occur. However, property is unlikely to be impacted on, although passage through the railway tunnel would be obstructed. Surface water flooding at this location is more likely under the current situation. Water ponds in the dip, and is discharged through road gullies. It is assumed that this discharge is directly to the Curries watercourse. If the discharge through the outfall pipes was blocked, for example through high levels in the river, the surface water would not drain away. As with fluvial flooding though, this will block access through the tunnel, but will not impact on property.

2.5.6 Devils Watercourse

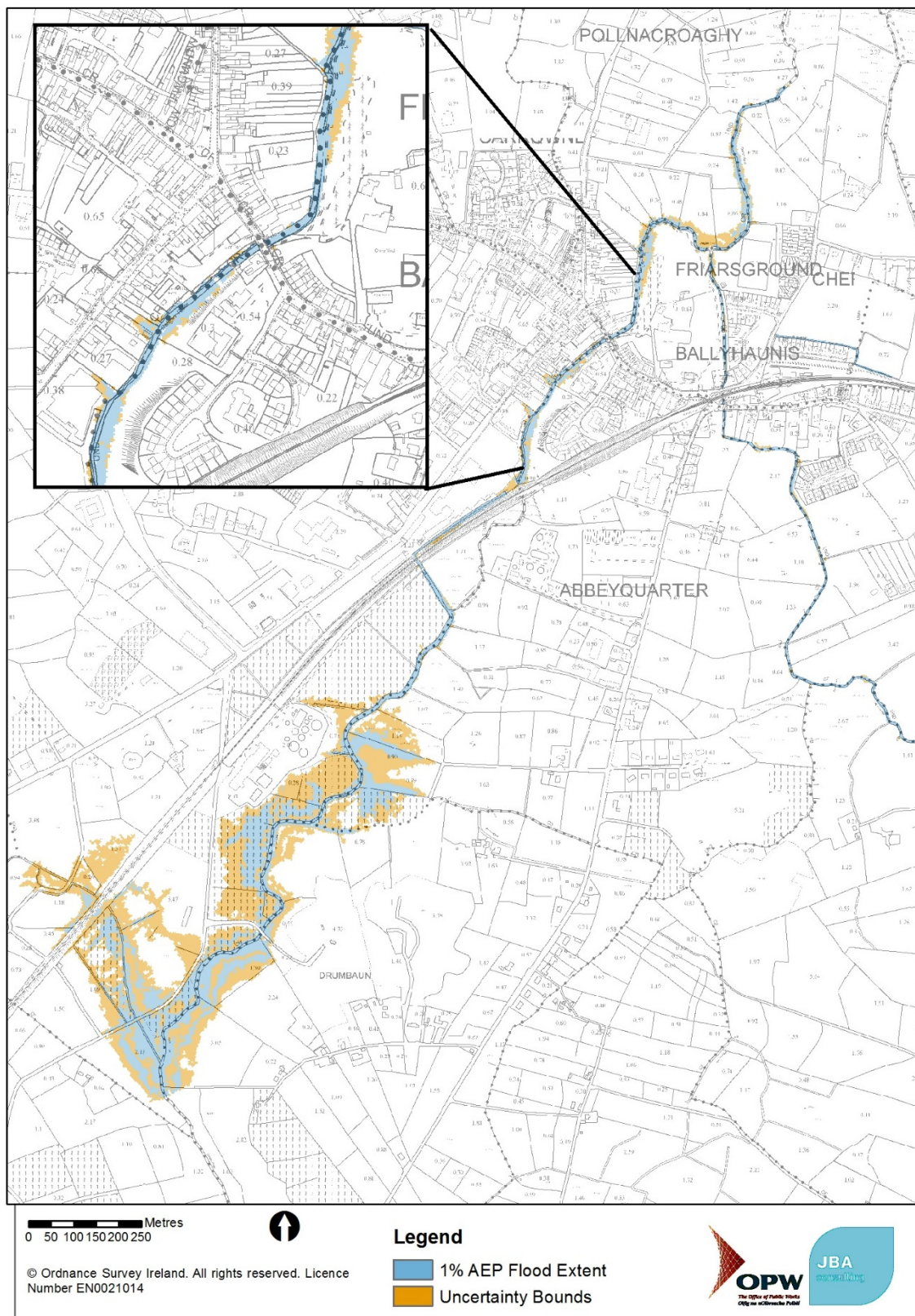
The Devils watercourse is shown to present no risk of flooding to properties in Ballyhaunis up to the extreme 0.1% AEP event, with all flows shown to remain within the drainage channel. This is a reflection of the over-capacity of the manmade drainage channel, coupled with the small catchment area draining into it.

Inspection during the site visit showed blockage of the culvert to be a likely occurrence.

2.6 Sensitivity testing

Sensitivity testing was conducted to determine which variables the model was sensitive to and their respective impacts. The model to the north of the railway line, and indeed for a short distance downstream of the railway crossing is not particularly sensitive to the parameters tested for either the 10% AEP or 1% AEP events, largely because the flooding is mainly in-bank, and the channel has sufficient capacity. The lower part of the model, where there is more out of bank flooding, shows a greater sensitivity to both flows and roughness changes. This is reflected in the increased flood extents, and also in flooding along the unmodelled tributary to the south of the domain, with water backing up under the railway line. Although inundating greater areas of agricultural land, there is no increase in the number of properties at risk. Overall, the hydraulic model was generally not shown to be sensitive with the exception of sensitivity to peak flow.

Figure 2-2: 1% AEP event uncertainty bounds - Ballyhaunis AFA



2.7 Flood risk summary

Table 2-4: Flood risk to receptors in Ballyhaunis

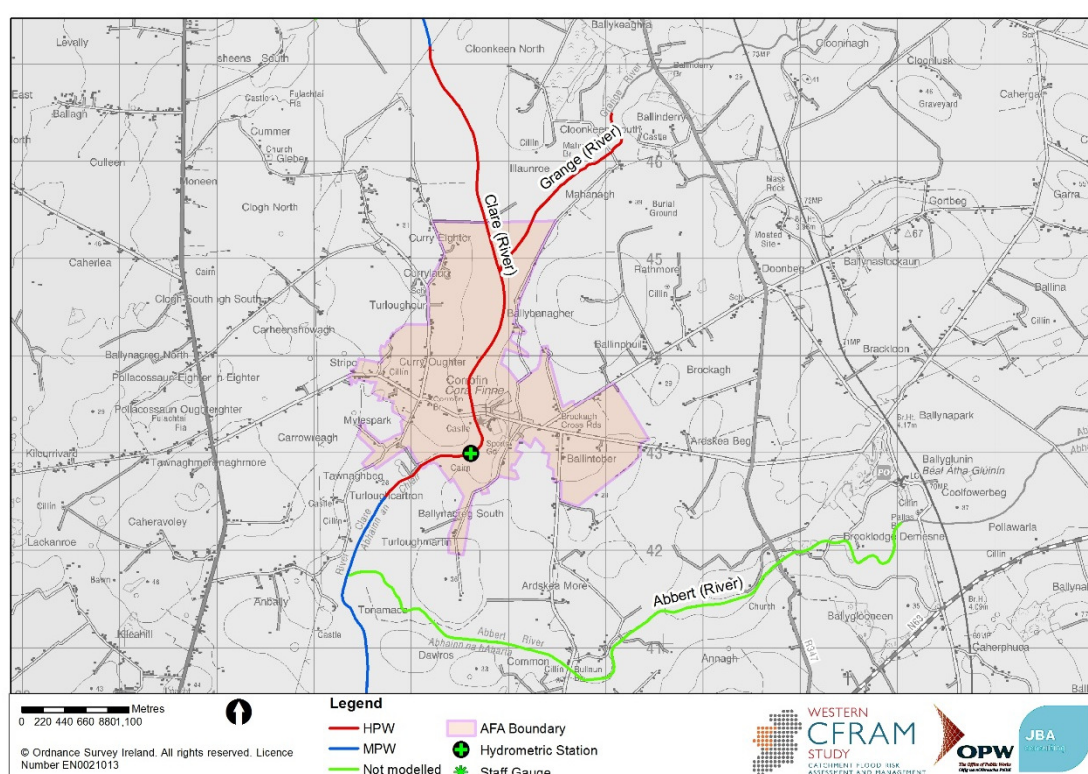
Risk Type	Receptor	Ballyhaunis				
		10% AEP	1% AEP	0.1% AEP	1% AEP MRFS	1% AEP HEFS
Social	Residential property	0	0	0	0	0
	School	0	0	0	0	0
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	0	0	0	0
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	0	0	0	0
	Civil defence HQ	0	0	0	0	0
	Social amenity sites	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SPAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Cultural heritage	National monument	0	0	0	0	0
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.00	0.00	0.00	0.00	0.00
	National roads (km)	0.00	0.00	0.00	0.00	0.00
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	0	0	0
	Commercial building	0	0	0	0	0

3 Corrofin

3.1 Watercourse and catchment overview

The study area encompasses the Corrofin AFA and includes the Clare River, the main river passing through Corrofin village, and Grange River, a tributary of the Clare River. The watercourses are all classed as High Priority Watercourses (HPWs) as they flow through the AFA, and have been included in the model. Figure 3-1 provides an overview of the area. To the south of Corrofin is the Abbert River; this is outside the town boundary and has not been included in the CFRAM study. The area around Corrofin is heavily influenced by the karstic limestone bedrock, with turloughs, springs and swallow holes a common feature. Groundwater modelling and assessment has not been carried out as part of the CFRAM.

Figure 3-1: Overview of rivers in the Corrofin catchment



3.1.1 Clare River

The upstream modelled extent of the Clare is located approximately 1.5 km upstream of the confluence with the Grange River, and the downstream modelled extent is near Daly's Bridge, approximately 4.5km downstream of Corrofin Bridge, or 1.5 km downstream of the confluence with the River Abbert. The gradient of the Clare River through this reach is very shallow, at 0.7m/km.

The Clare River channel has been extensively worked over the centuries and is part of the Clare-Corrib OPW maintained arterial drainage scheme. This is evidenced by the extensive informal embankments along the river channel that appear to be made up of dredged material. Historic OSi mapping also suggests the river has been significantly altered, with changes to the permanence and location of loughs up and downstream of Corrofin and the alignment of the river through Corrofin. It also appears that Corrofin Bridge was changed from a bridge with multiple small arches to the one used today with a main span and remaining small side arches.

3.1.2 Grange River

The modelled length of the Grange River is 2.2 km. The gradient of this watercourse is approximately 1m/km. As with the River Clare, the Grange River has been subject to historical alteration, particularly downstream of Mahanagh Bridge. The Grange River would have flowed

into Cloonkeen Lough, upstream of the town before the improved drainage works were implemented in the 19th century.

3.2 Flood history

Further details on the flood history of the town can be found in the AFA specific modelling report and the Hydrology and Inception Reports. A summary of known flood events is provided below.

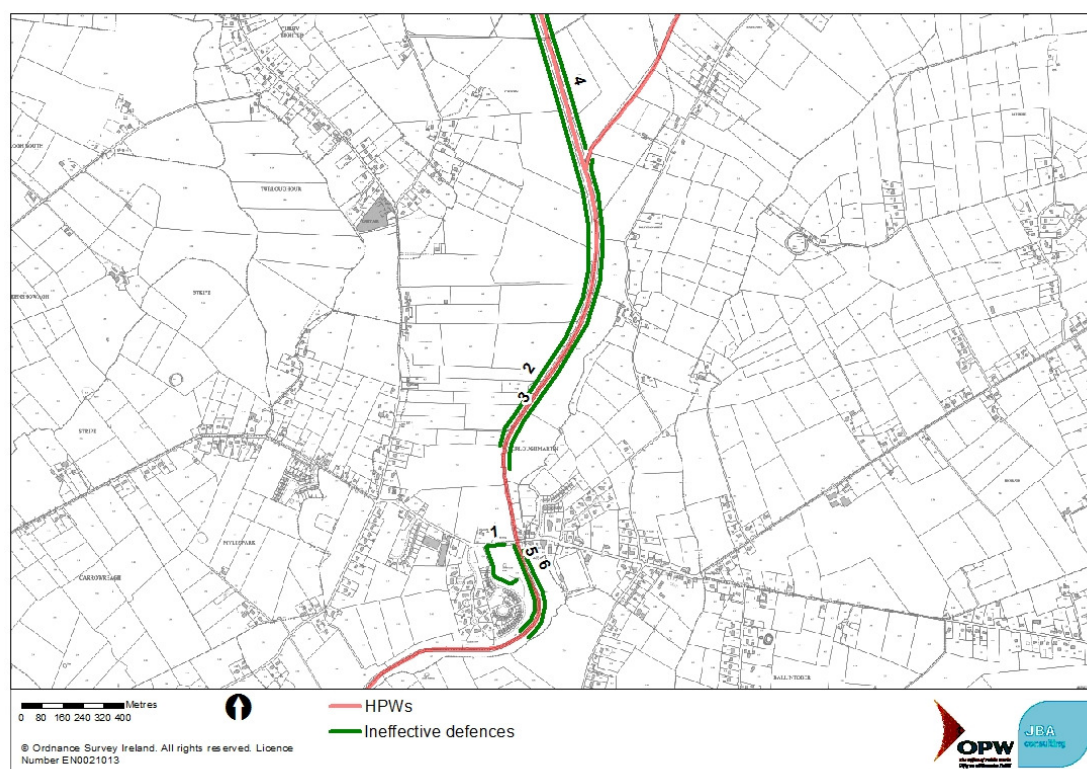
Table 3-1: Flood history summary in Corrofin

Area affected	Main flooding mechanisms	Recorded flood event data	Use in model check
Ballybanagher	Fluvial flooding from Clare and Grange rivers	November 2009	Verification run using aerial photography.
Unknown, but third highest stage recorded. Extensive floodplain inundation anticipated.	Fluvial	2006	Sensibility check
Unknown, but recorded stage is higher than 2009	Fluvial	1968	Not used - measurement is highly uncertain given a datum change since then and likely channel changes
Rural floodplain	Fluvial, surface water, drains and potentially groundwater	Recurring	Sensibility check

3.3 Existing defences and walls

No formal or informal effective defences were identified within the AFA. Six lengths of informal ineffective structures were identified within the AFA along the banks of the Clare River. These structures are not assumed to function as flood defences and are either bypassed in the model or have been removed to allow flooding beyond them.

Figure 3-2: Overview of defences in Corrofin



3.4 Model limitations

- Bankside embankments - The river banks along the Clare and Grange rivers are generally in the form of informal raised embankments that have gaps in them. As far as possible in the model these have been represented as they have been surveyed. The model is, however, a simplification of the actual situation and may not accurately model the flow paths onto the floodplain. Despite this, the overall effect looks realistic when compared with aerial photography of the 2009 flood, and local knowledge.
- Influence of tributaries and groundwater - There is a wide spread of gaugings at the Corrofin gauge and it is suggested that is because of a backwater effect from the Abbert confluence downstream and the influence of groundwater on the system. Further investigation could help understanding of the situation, but such detailed groundwater assessment is outside the scope of the CFRAM. Ideally, additional monitoring of water levels would be required downstream of the gauge location to understand the variation of water surface profile through that area; it is likely that this effect would only be seen during extreme events.

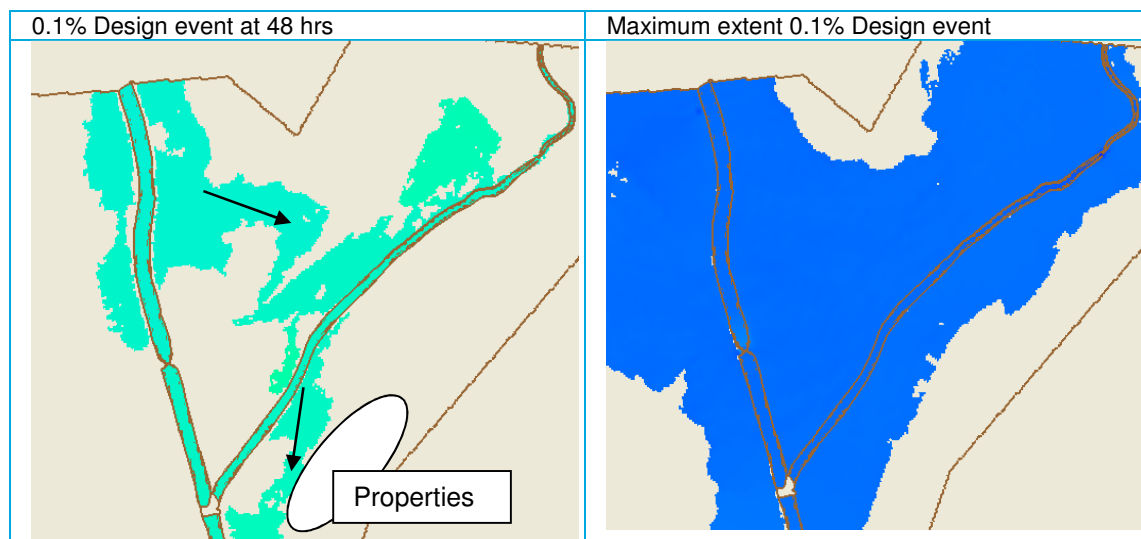
3.5 Key flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below.

The only area of property flooding within the AFA is upstream of Corrofin Bridge on the combined Clare and Grange floodplain. The area where property is at risk is close to the confluence of the two rivers on the eastern edge of the floodplain where development has encroached. The flood risk mechanism in this area is simply the large floodplain filling up to a level to cause flooding to the property.

Flood water on the left bank floodplain of the Clare River flows into the River Grange channel and the combined flow comes out of that channel into the area of the properties. Table 3-2, below shows early flood routes and then the full flood extent during the 0.1% AEP design run. Property flooding is experienced in as low as the 10% AEP event.

Table 3-2: Corrofin flood mechanism



3.6 Sensitivity testing

Sensitivity testing was conducted to determine which variables the model was sensitive to and their respective impacts.

Only the peak flow test was carried out for sensitivity testing in the 10% AEP event; consequently, any increase in flooding shown in the figure is as a direct result of this test. The greatest change in flood extent from sensitivity testing of this event is to the fields upstream of Corrofin Bridge, adjacent to Ballybanagher. One additional property is shown to be at risk of flooding in Ballybanagher. The increase in flooding as a result of the sensitivity testing is not dramatic for the

1% AEP event as shown in Figure 3-3 and is directly attributed to the sensitivity to peak flow. Flooding is shown to reach much nearer to properties in Corrofin itself, however this still does not result in the flooding of any properties in this location. This shows that the model was not sensitive to the other parameters tested. Figure 3-4 presents the extent of flooding where a storm event was run only on the Clare River and it demonstrates quite clearly that the flooding shown on the Grange River is almost completely attributable to the levels within the main river, the Clare River, particularly around the key flood risk area of Ballybanagher.

Figure 3-3: 1% AEP event uncertainty bounds - Corrofin AFA

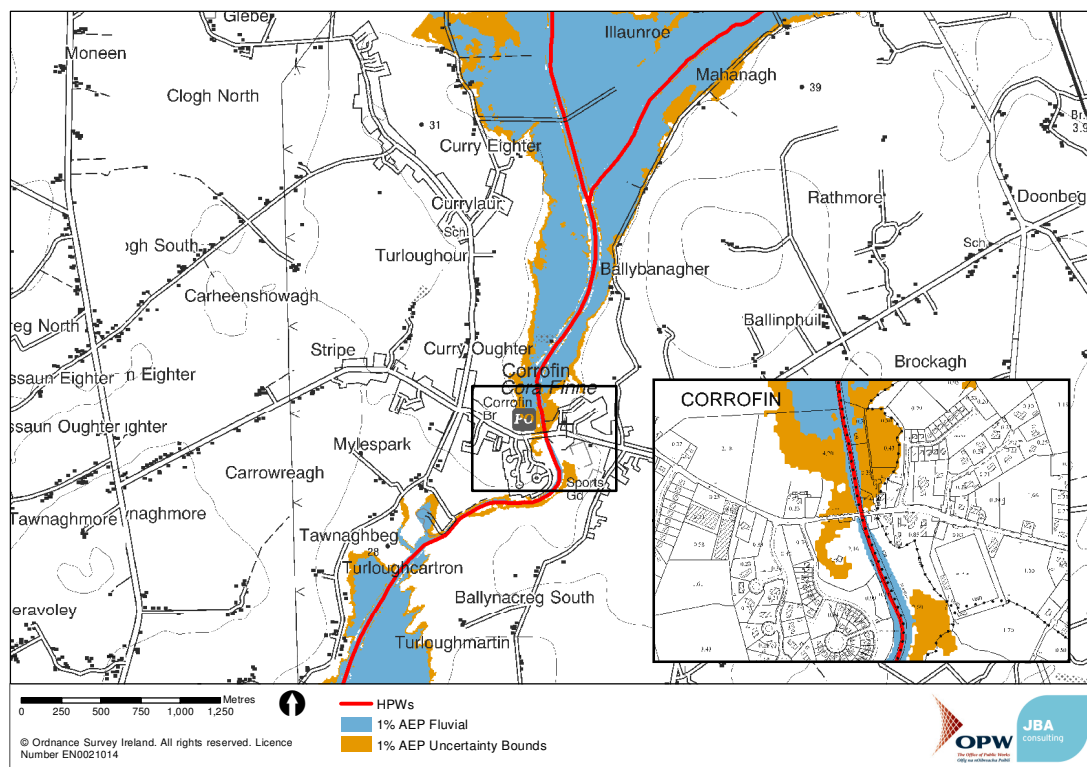
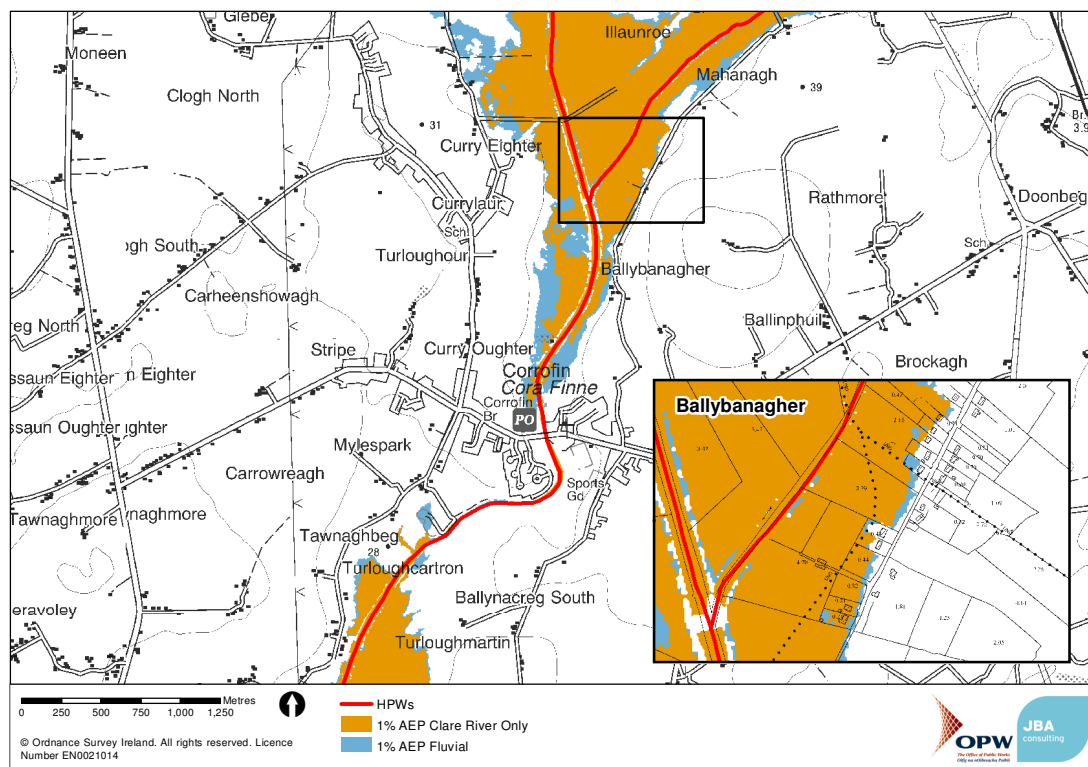


Figure 3-4: 1% AEP event uncertainty bounds - Clare River



3.7 Flood risk summary

Table 3-3: Flood risk receptors in Corrofin

Risk Type	Receptor	Corrofin				
		10% AEP	1% AEP	0.1% AEP	1% AEP MRFS	1% AEP HEFS
Social	Residential property	2	3	9	5	6
	School	0	0	0	0	0
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	0	0	0	0
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	0	0	0	0
	Civil defence HQ	0	0	0	0	0
	Social amenity sites	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SPAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Cultural heritage	National monument	0	0	0	0	0
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.00	0.00	0.00	0.00	0.00
	National roads (km)	0.00	0.00	0.00	0.00	0.00
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	0	0	0
	Commercial building	0	0	0	0	0

4 Galway City

4.1 Watercourse and catchment overview

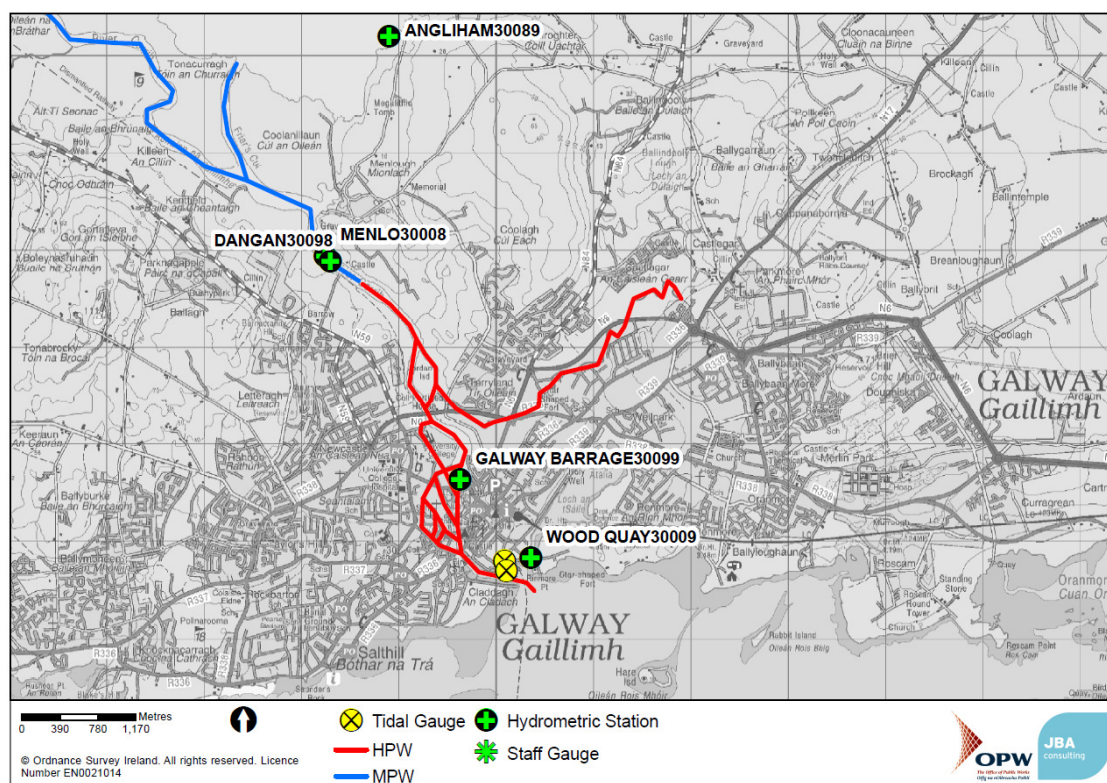
The Corrib flows along a short channel through Galway City which joins the outlet of Lough Corrib to the sea. Its catchment to the outfall is large (3,140km²). Loughs Corrib and Mask form a dividing line between two quite different portions of the catchment. To the east of the Loughs, where the bulk of the catchment lies, the land is low-lying with moderate rainfall and karst limestone geology. The smaller tributaries flowing into the Loughs from the west are much steeper, draining impermeable mountainous catchments with high rainfall.

Lough Corrib is the second largest lake in Ireland, with an area of 178km². It has a major influence on the nature of flood flows along the River Corrib through Galway. The management of Lough Corrib has changed over the years. In the 12th century, the Friars Cut was built to provide another outlet from the Lough into the River Corrib in an attempt to allow boats to access the Lough from the sea. Between 1846 and 1850 the lake was lowered to reduce flooding of surrounding farm land (Freeman, 1957). Between 1848 and 1857, the Eglinton canal was built, connecting the River Corrib to the sea. It allowed boats to access the Lough via a single lock and also made provision for improved operation of over 30 mills.

In 1959, a weir constructed in the 1850s was replaced with a sluice barrage (the Salmon Weir) consisting of 16 gates. The barrage is close to the centre of Galway, 800m upstream of Wolfe Tone Bridge, immediately downstream of the point where the Eglinton Canal leaves the river. This is 7.8km downstream of the main outlet from Lough Corrib. A small amount of flow can bypass this structure via various canals and mill races.

The barrage was intended to keep levels on the Lough between 5.84 and 6.44mAOD Malin (i.e. 28-30 feet above OD Poolbeg). The upper limit is intended to avoid flooding of shoreline and lower reaches of tributary rivers. The original design envisaged that this upper limit level would be reached at a flow of 311m³/s. This upper limit has been exceeded almost every year, apart from 1995 and 2005.

Figure 4-1: Galway City AFA catchment overview



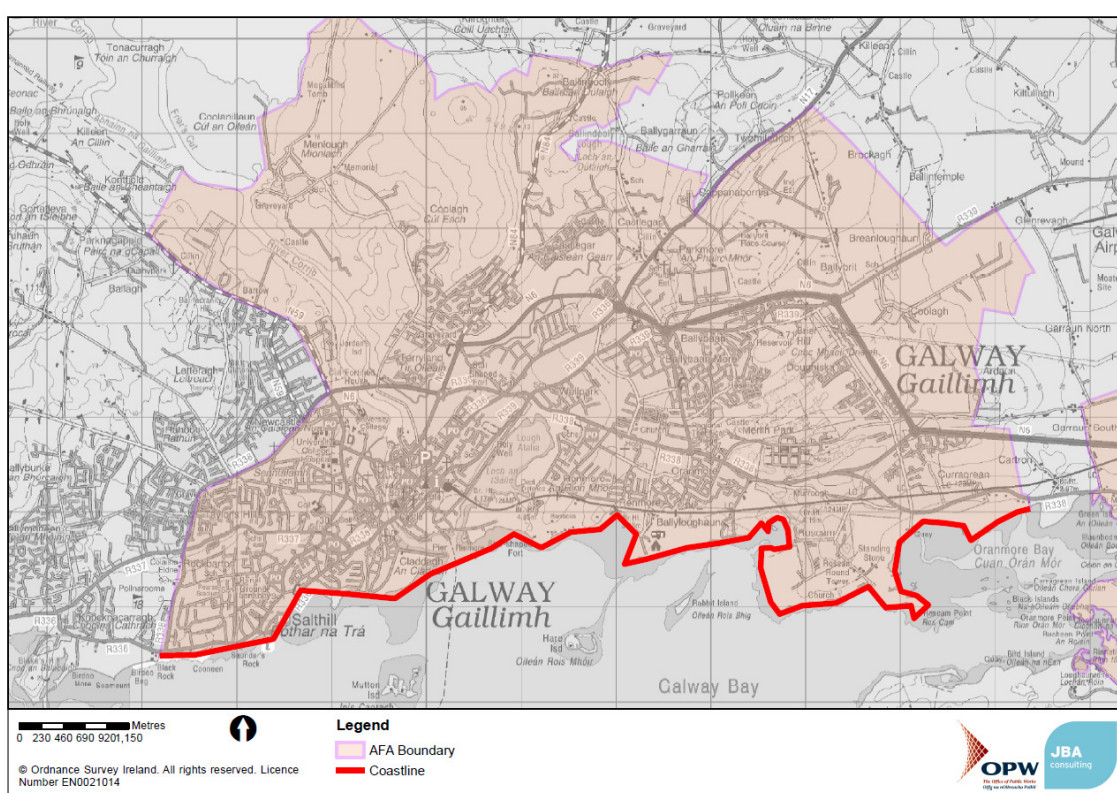
For the purpose of a catchment wide study the Lough Corrib to Galway Bay fluvial reach is divided into two models, the Medium Priority Watercourse (MPW) and the High Priority Watercourse

(HPW). The HPW model is a more detailed study of the flood risk mechanisms required for an urban area. It starts at the Dangan gauge and continues to Galway Bay.

The MPW outputs are to be used to investigate the key controlling elements of the watercourse system and support the AFA modelling decisions. Specifically, this model will be used to assess the impacts of the Salmon Weir Barrage on Lough Corrib and the River Clare. Figure 4-1 shows the overview of the HPW and MPW models.

A third model has been constructed to look at the impacts of direct coastal inundation and wave overtopping. Galway City has a coastline of approximately 13.5km that stretches from Black Rock in the west to Roscam Point to the east. Figure 4-2 shows the extent of the coastline and outline of the AFA area. There is substantial evidence that reclamation of lands from the sea occurred in Salthill during the last century, including White Strand and the land on which Toft Park, the Aquarium and car park are all located.

Figure 4-2: Galway coastal overview



4.2 Flood history

Further details on the flood history of the town can be found in the AFA specific modelling report and the Hydrology and Inception Reports. A summary of known flood events is provided below.

Table 4-1: Flood history summary in Galway City

Area affected	Main Flood Mechanisms	Recorded Flood Event
Spanish Arch, Quay Street, Flood Street and the Docks area, Lower Salthill	Due to high tide, low atmospheric pressure, wind direction, heavy rain.	Jan 1995, High tides in 2006, Jan & Feb 2014, recurring
Grattan Road	Overtopping, high tides and onshore winds.	Recurring
Seapoint Promenade	Overtopping, high tides and onshore winds.	Jan & Feb 2014, recurring
N17 at Two Mile Ditch	Heavy rain	Jan 1995, 1999, 2005, Nov 2009, recurring
Salthill, Fr. Griffin Road, Claddagh and Spanish Arch Areas	Heavy rain, gale force winds, high tide	Feb 2002 recurring
Headford rd/Ballindooley	Ballindooley lake margin	Feb 2002, prone to

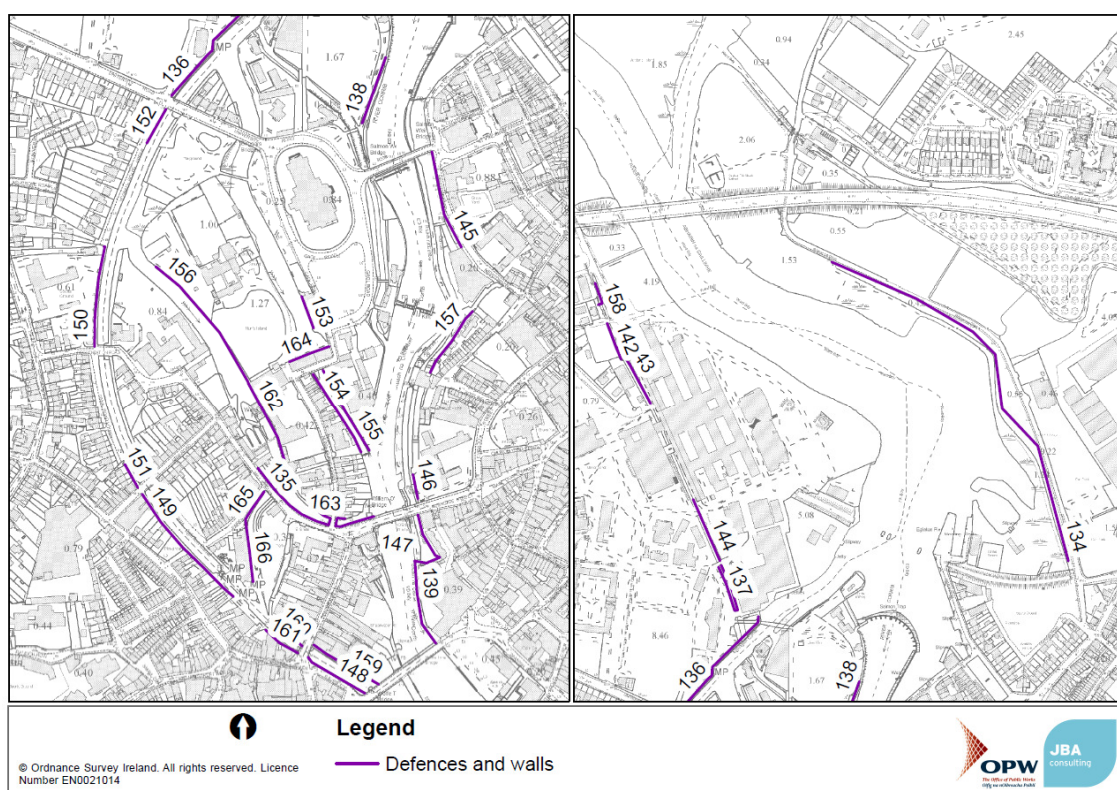
Area affected	Main Flood Mechanisms	Recorded Flood Event
	increased during heavy rain	flooding
Doughiska	Turlough	Recurring
Menlough	Turlough heavy rain	Nov 2009, recurring

4.3 Existing defences and walls

A number of formal (i.e. OPW, local authority or privately maintained defences) flood defences have been identified with the AFA, namely the dyke running in a north-northwest direction, a retaining wall on right bank of River Corrib and a short length of quay wall. The two effective structures have been modelled as surveyed and are assumed to retain flood waters to the crest of the structure. The ineffective quay wall is so called because it forms an incomplete run of defence. However, the full length of the quay has been included in the model as per the survey details.

Informal ineffective structures, identified with the AFA, are not assumed to function as flood defences and are either bypassed in the model or have been removed to allow flooding beyond them. All the identified defences, whether effective or ineffective, are shown in Figure 4-3.

Figure 4-3: Overview of defences in Galway City



4.4 Model limitations

Fluvial model

- Channel blockage - Blockage has not been investigated in more detail in this model as it is outside the scope of the CFRAM but there are high number of culverts, particularly on the canals, where blockage has been witnessed and if exacerbated could give rise to significant flood risk.
- Salmon Weir - Water level upstream of the Salmon Weir Barrage, and the flow to the canals is controlled by the operation of the 16 gates that form the structure. The operation rules are not known but as a conservative (i.e. higher flow) scenario, it has been assumed that the gates would be open for all the design events carried out in this study. This assumption is supported by operational evidence from the 2009 event and feedback that the gates are normally opened during the winter period.
- Structure representation - uncertainty in the modelling of structures where health and safety and technical issues meant there was limited survey data collected, particularly on

the Eglinton Canal, Nuns Island and Distillery River. Flow controls at the water treatments works on the Castlegar River are based on surveyed headloss, rather than knowledge of internal structures.

- Geomorphological change - The channel bed constantly shifts and may be easily eroded by high river flows. Although not examined through modelling, the likely impact on flood relief schemes has been appraised.
- Karst features - The Castlegar River flows into a sink hole, of which the operation and capacity is unknown. Analysis of the groundwater system is outside the scope of the CFRAM.
- Model calibration - There is limited fluvial calibration data available. A programme of water level monitoring across the canal system would to help calibrate the model.
- Inclusion of all the mill races and connections - A number of connections were not modelled due to there being no influence on flood risk.

Coastal model

- ESTRY Components - Width of structures were restricted to a minimum width of four metres due to the model so some structures were modelled with larger areas than their actual area.

4.5 Key fluvial flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below. The downstream tidal boundary has a significant influence on a number of watercourses. The floodwaters from the tidal element are prevented from translating upstream by numerous weirs and sluices in Galway City.

4.5.1 Flooding of Upper Corrib

The flood extents arising from the MPW model on the Upper Corrib show much inundation in the 10% AEP event but this is mainly encroachment onto marsh bog land. Floodwaters enter the turloughs on the left bank, increasing the lake area and causing back watering to various streams that provide drainage to the area.

There are no properties at risk in this area as this land appears to have been liable to flooding in the past and development in this area has been avoided. This is indicated on the OSi historic 25" maps circa 1897-1913.

4.5.2 Flooding at Castlegar

The flooding on Castlegar is controlled by the amount of floodwater that can be conveyed through the sinkhole at the downstream and the inlet from the waterworks structure, both of which are uncertain. The 1% AEP extent shows the Dyke road embankment overtopped and allowing flow into the Castlegar area. This contributes mostly to the flooding in Castlegar. The backing up of drains at the downstream is evident. No properties are affected in this area.

4.5.3 Flooding of Distillery River

The Distillery River is a complex area of the Galway City AFA. The controlling structure of the watercourse is a siphon under the Eglinton canal in the middle of the watercourse. Floodwaters from the Eglinton canal can spill over its banks adding to floodwater in the Distillery River. No flooding occurs in the 10% AEP or 1% AEP event, however the 0.1% shows properties affected. The properties affected are mainly belonging to NUI Galway. There have been improvements to this channel since the CFRAM topographic survey was collected following recommendations in a report titled, NUI Galway Campus Flood Prevention⁶, the impact of which will be investigated in the next stages of the CFRAM.

4.5.4 Flooding of right bank canal system

The right bank canal system represents the canals whose flood waters are fed by the branch upstream of the Salmon Weir barrage. The Eglinton Canal feeds a number of old mill races

⁶ NUI Galway Campus Flood Prevention, University Road Galway

through the heavily urbanised area. Flooding occurs in the 1% AEP from the Galway Mill Run Area canal which overtops its right bank. Similar flooding is shown in the 0.1% AEP event, along with right bank flooding from the Madeira Court watercourse due to the culverts surcharging.

4.5.5 Flooding of downstream at Wolfe Tone Bridge

Flooding is evident in this area in the 1% and 0.1% AEP fluvial events that are run concurrently with a 50% AEP tidal events. In the 1% event the low lying area near the Claddagh basin floods from the downstream of the Eglinton (EGLI) canal and the basin itself over flowing. The area is greatly influenced by the tidal boundary and the fluvial event alone would not be expected to be sufficient to flood this area. The 0.1% event flooding shows further exacerbation on the right bank around Wolfe Tone Bridge. The area is a known area of flood risk from high tides.

4.6 Key tidal flood risk mechanisms

4.6.1 Flooding in the Salthill Area

The flood extent map for the Salthill area of Galway City shows much inundation in the 10% AEP event. Known areas of flooding such as Toft Park and along the promenade are shown flooded in the 10% AEP. The flooding is more extensive through Grattan Road in the 0.5% AEP and shows further properties affected.

4.6.2 Flooding at Spanish Arch and Dock Road

The area around Spanish Arch, Flood Street and Dock Road is a known area of flood risk from historical evidence and the more recent event of winter 2013/14. From the flood maps produced much of the flooding occurs in the 0.5% AEP event in comparison to the 10% AEP event which does not show extensive flooding. There is a marginal difference in flood extent between the 0.5% AEP and the 0.1% AEP.

4.6.3 Flooding at Claddagh Basin

There is no flooding shown at Claddagh Basin in the 10% AEP event but the 0.5% AEP extent shows expansive flooding down Father Burke road onto Father Griffin road. There are historical records of this flooding and the maps reinforce the flood risk potential present in this area.

4.6.4 Wave overtopping flood risk mechanisms

The wave overtopping results indicate that the Salthill area is susceptible to wave overtopping and experiences flooding from this mechanism at an AEP greater than 10%. This was to be expected and is evident from the flood history of the area.

The docklands area is not prone to wave overtopping. There is sufficient slope to return the wave volume breaking onto the ground to the sea.

4.7 Sensitivity testing

Sensitivity testing was conducted to determine which variables the model was sensitive to and their respective impacts.

4.7.1 Fluvial

There is a significant difference between the design 1% AEP model runs and the respective sensitivity runs as evident in Figure 4-4 and Figure 4-5. The most noticeable increase in flood extents along the Castlegar River is due to the increased flow sensitivity test. This causes the Dyke Road defence to overtop and exacerbate the flooding in this area. It was found that an increase in peak flow resulted in the greatest sensitivity. Adjusting roughness had a marginal affect. Adjusting building threshold has an impact of increasing flooding in areas while decreasing it at the same time in other areas.

Figure 4-4: 1% AEP fluvial event uncertainty bounds - Claddagh Area

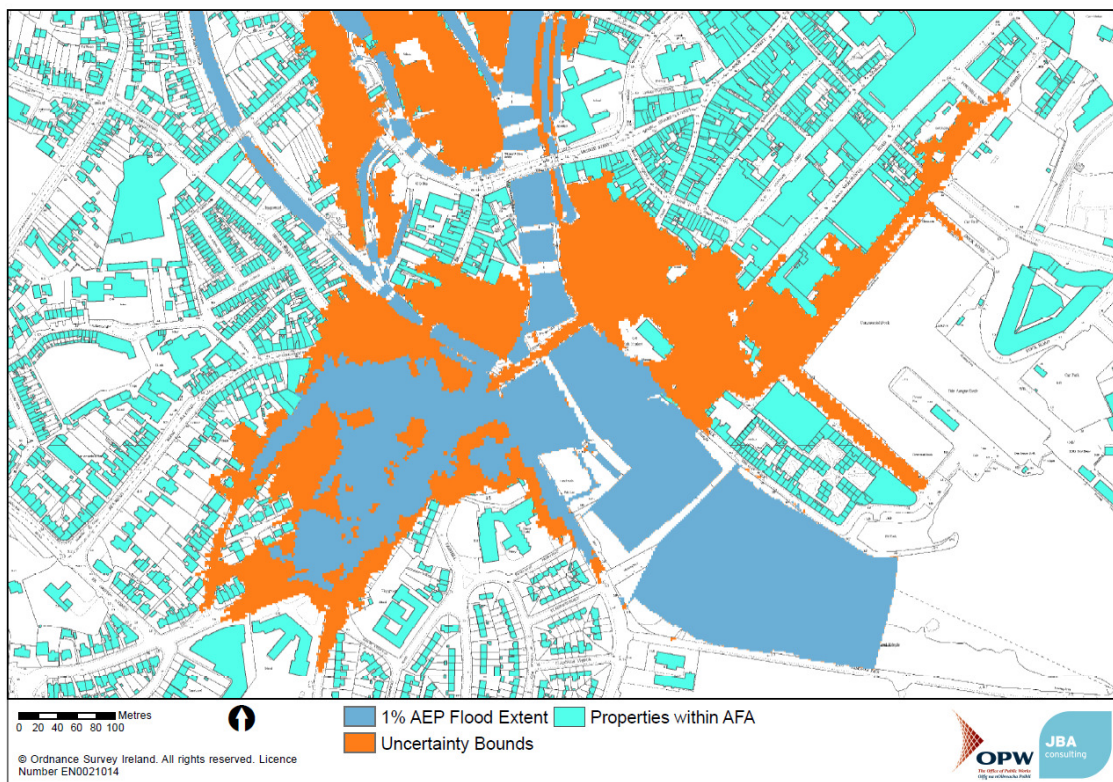
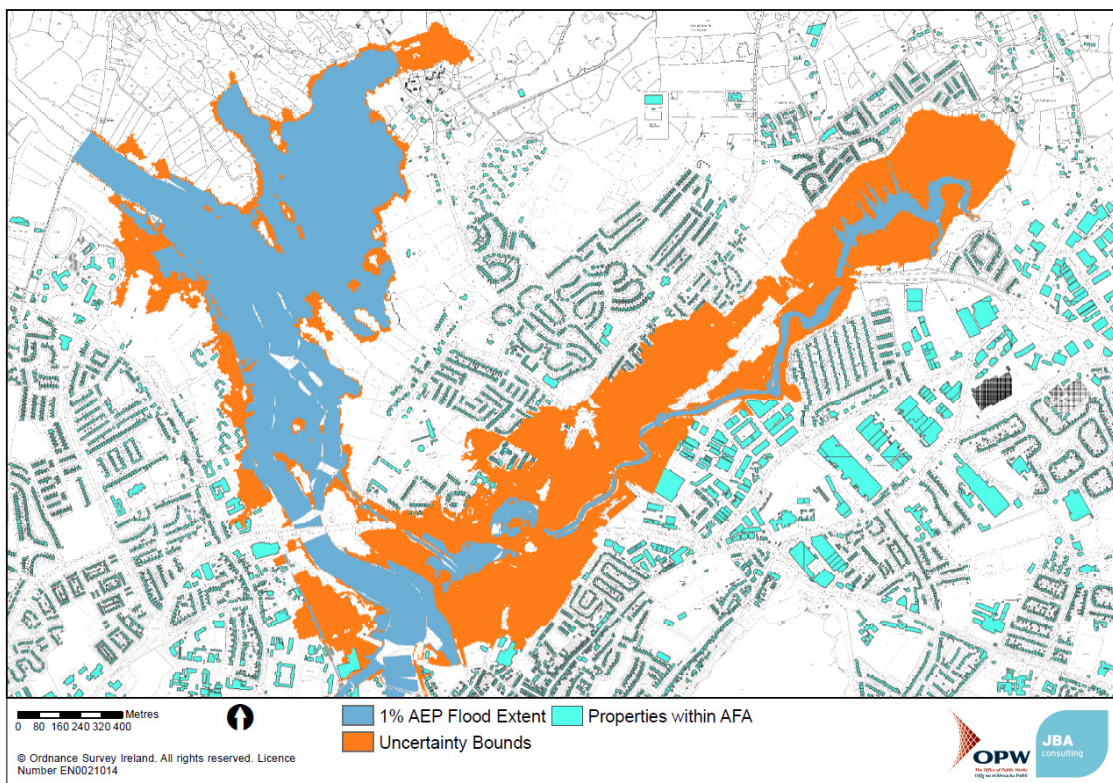


Figure 4-5: 1% AEP fluvial event uncertainty bounds - Upper Corrib



4.7.2 Coastal

The sensitivity of the Galway City coastal model to the tests are very marginal and indicate the model is insensitive to the parameters tested, as can be seen in Figure 4-6 and Figure 4-7.

Figure 4-6: 0.5% AEP tidal event uncertainty bounds - Salthill West

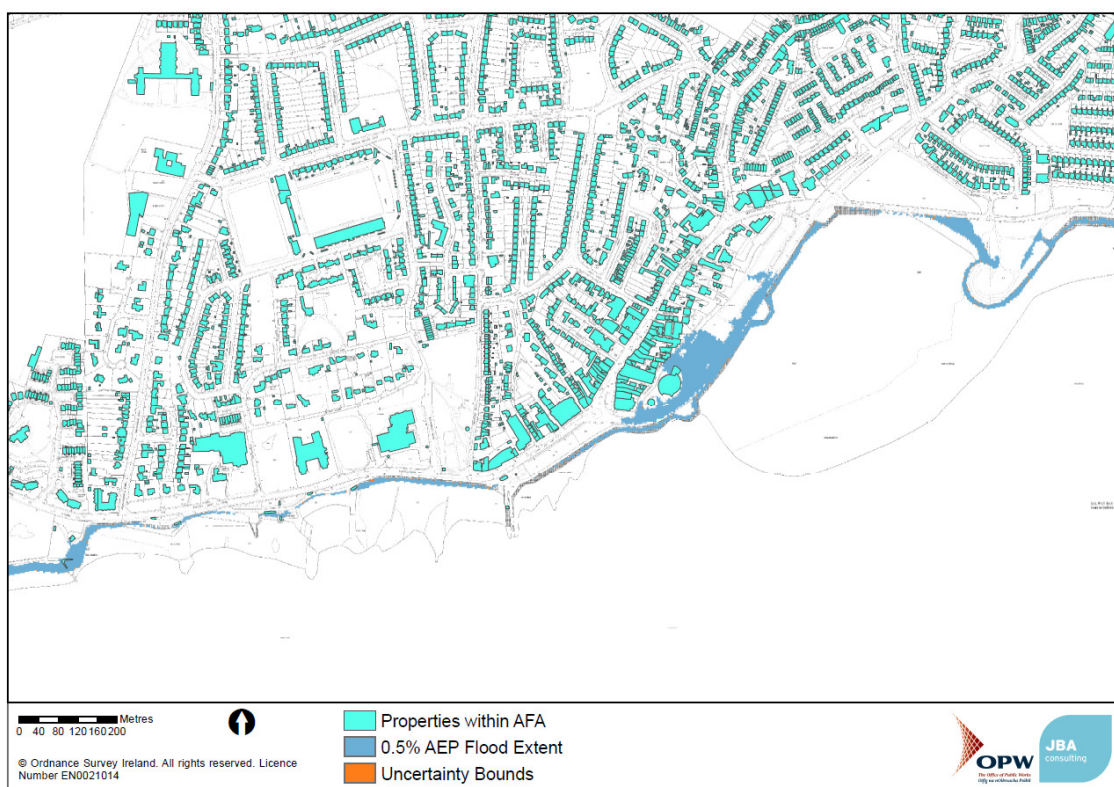
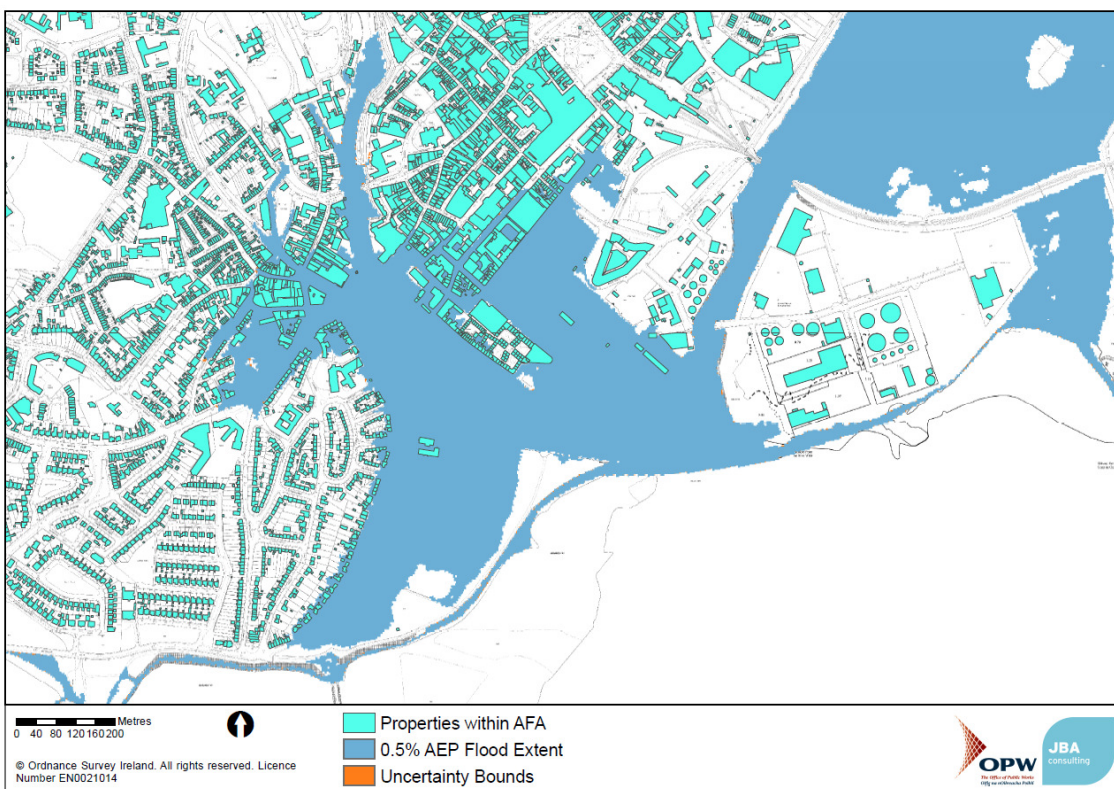


Figure 4-7: 0.5% AEP tidal event uncertainty bounds - Salthill East



4.8 Flood risk summary

Table 4-2: Fluvial flood risk to receptors in Galway city

Risk Type	Receptor	Galway city				
		10% AEP	1% AEP	0.1% AEP	1% AEP MRFS	1% AEP HEFS
Social	Residential property	16	99	895	504	648
	School	0	0	2	0	1
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	0	2	2	2
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	0	1	1	1
	Civil defence HQ	0	0	1	1	1
	Social amenity sites	At Risk	At Risk	At Risk	At Risk	At Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	At Risk	At Risk	At Risk	At Risk	At Risk
	SPAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Cultural heritage	National monument	1	1	5	3	7
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.00	0.00	0.00	0.00	0.00
	National roads (km)	0.00	0.00	3.91	0.00	0.26
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	0	0	0
	Commercial building	16	81	627	350	489

Table 4-3: Tidal flood risk to receptors in Galway city

Risk Type	Receptor	Galway city				
		10% AEP	0.5% AEP	0.1% AEP	0.5% AEP MRFS	0.5% AEP HEFS
Social	Residential property	99	678	934	1266	2525
	School	0	0	0	0	0
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	2	2	2	2
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	1	1	1	1
	Civil defence HQ	0	1	1	1	1
	Social amenity sites	At Risk	At Risk	At Risk	At Risk	At Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at	Not at	Not at	Not at	Not at

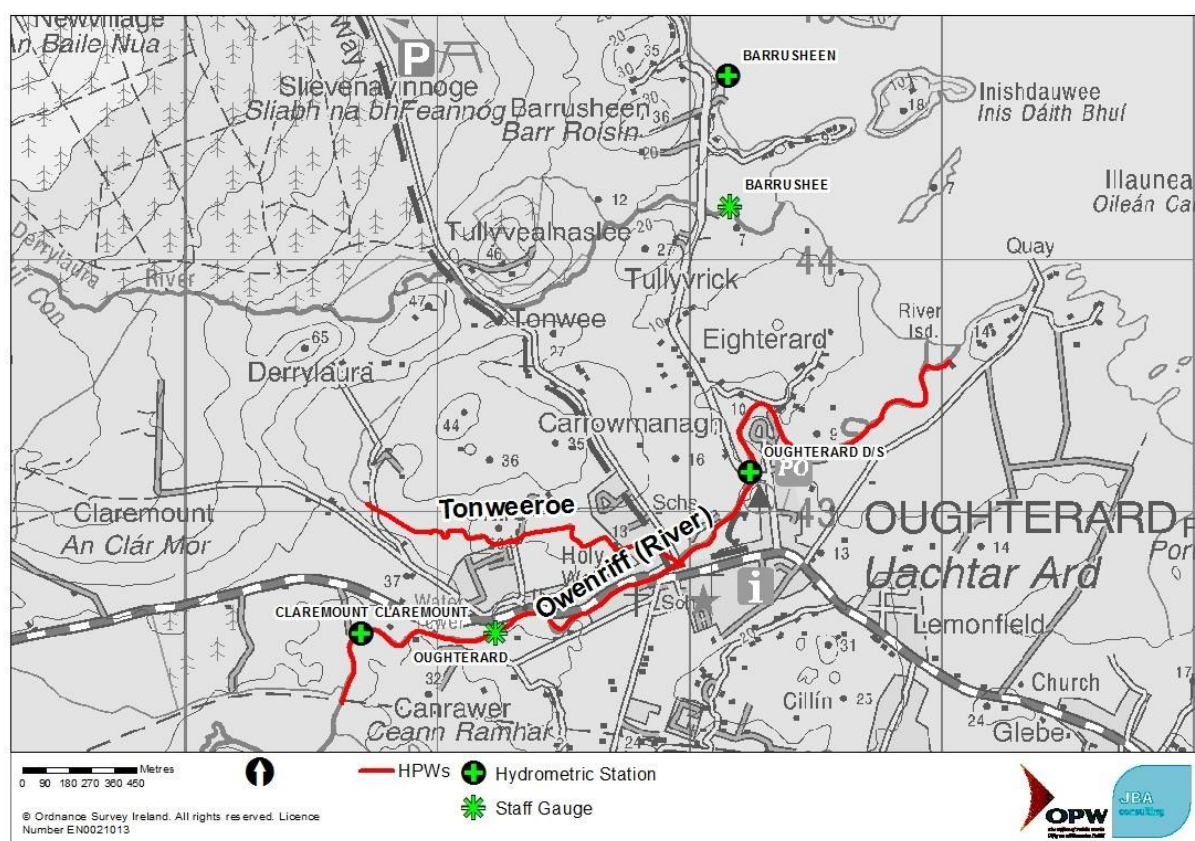
Risk Type	Receptor	Galway city				
		10% AEP	0.5% AEP	0.1% AEP	0.5% AEP MRFS	0.5% AEP HEFS
		Risk	Risk	Risk	Risk	Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	At Risk	At Risk	At Risk	At Risk	At Risk
	SPAs	At Risk	At Risk	At Risk	At Risk	At Risk
Cultural heritage	National monument	3	4	6	10	13
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.08	0.08	0.09	0.09	0.10
	National roads (km)	0.00	0.00	0.00	0.00	0.00
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	0	1	1
	Commercial building	70	508	626	724	1051

5 Oughterard

5.1 Watercourse and catchment overview

The study area encompasses the Oughterard AFA and includes the Owenriff River, which is the main river passing through Oughterard town centre, and Tonweeroe Watercourse, a tributary of Owenriff River. The watercourses are all classed as High Priority Watercourses (HPWs) as they flow through the centre of the AFA, and have been included in the model. Figure 5-1 provides an overview of the area. The Owenriff River discharges into Lough Corrib a short distance downstream of Oughterard.

Figure 5-1: Oughterard AFA catchment overview



5.1.1 Owenriff River

The upstream modelled extent of the Owenriff is located at the old railway crossing approximately 1.5km upstream of the N59 Bridge, and the downstream modelled extent is at Lough Corrib; a distance of 3.7km. Across the modelled reaches the bed elevation drops 21.4m. The gradient of the Owenriff River through this reach is very variable, from extremely steep cascades towards the upstream end to very flat gradient as it flows into the lough.

The Owenriff River channel appears fairly natural and reference to historic maps shows very little change in course over the past nearly 200 years.

Table 5-1: Owenriff River gradient changes

	
<p>Owenriff River approaching Lough Corrib towards the downstream end of the model (30ORIF00064).</p>	<p>Owenriff River cascades towards the upstream end of the model (30ORIF00283).</p>

5.1.2 Tonweeroe Watercourse

The modelled length of the Tonweeroe watercourse is 1.5km. There are several small culverts along the length of the watercourse that can restrict extreme flows. The longest of these is located at the downstream end of the Tonweeroe watercourse beneath the houses of Abhainn Na Coille to the point of discharge into the Owenriff River. This culvert is around 130m long and has a screen on the inlet.

Inspection of the route of the Tonweeroe watercourse during site visits indicated that not all the flows reach this downstream culvert and it is often dry which suggests some water may be lost into the ground. At extreme flows this is likely to be less significant. Analysis of the historical mapping available through the OSi shows the downstream extent of this watercourse does not link to the Owenriff, but it is not clear where the channel does discharge to.

5.2 Flood history

Further details on the flood history of the town can be found in the AFA specific modelling report and the Hydrology and Inception Reports. A summary of known flood events is provided below.

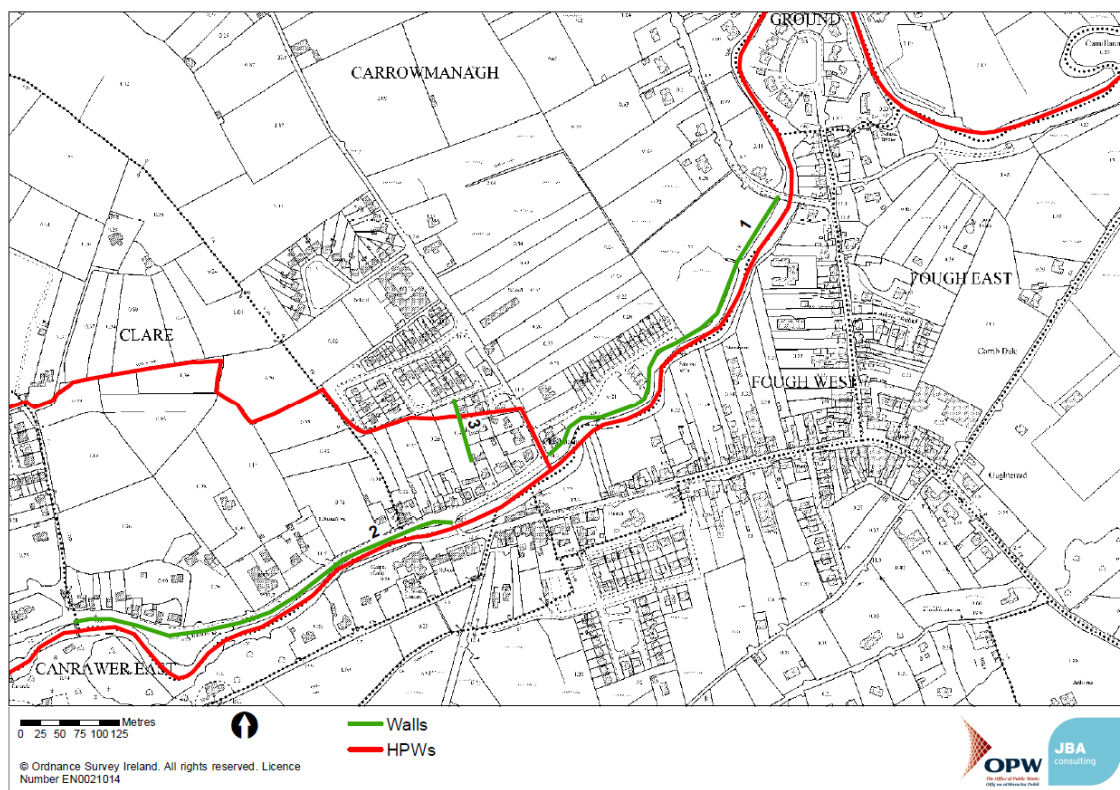
Table 5-2 : Flood history summary in Oughterard

Area affected	Main flood mechanisms	Recorded flood event date	Use in model check
Abhainn Na Coille	Fluvial	07/02/2000	Calibration run
Abhainn Na Coille	Fluvial	28/11/1999	Calibration run
Low lying field near Lough Corrib	Lake	5 year reoccurring	Sensibility check on design extents

5.3 Existing defences and walls

No formal or informal effective defences were identified within the AFA although three informal ineffective structures were identified along the banks of the Ballynageeha Watercourse. These structures are not assumed to function as flood defences and are either bypassed in the model or have been removed to allow flooding beyond them. The exception is the single skin block wall extending over the inlet to the downstream culvert on the Tonweeroe. This structure has been included as depths are shallow and it diverts flows to the south.

Figure 5-2: Defences and walls locations



5.4 Model limitations

- **Channel blockage** - In Oughterard the culverts on the Tonweeroe watercourse look particularly prone to blockage particularly at the screen. If the culvert blocked, water would back up in the channel, before overtopping onto the field into nearby properties. Although not investigated in more detail in this model, it is likely that culvert blockage will increase flood risk to property.
- **Cascades** - The cascades on the Owenriff River are extremely steep and have been simplified into a single drop in the ISIS model. This is considered acceptable to achieve model stability and because there is no flood risk in the immediate area.
- **Swallow hole** on the Tonweeroe watercourse - There appears to be swallow hole on the Tonweeroe watercourse as flow typically seems to be greater towards the upstream end, and almost dry under normal conditions at the downstream end. This has been ignored for design event models as it is assumed the swallow hole is overwhelmed and all the design flow peak will travel down the watercourse channel.

5.5 Key flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below.

5.5.1 Flooding upstream of N59 Bridge on left bank

In events greater than 1% AEP, the water can rise over the left bank onto the N59 Clifden Road and flow along the road parallel to the river. The road is slightly elevated above the river level but there is no consistent raised wall along here. The N59 Bridge causes elevated upstream water levels due to it constricting the flow area. However, given the river's steepness and extent the road can flood, the bridge does not solely control this elevated water level.

5.5.2 Flooding at Oughterard gauging station (Glann Road) bridge

There is flooding on the left bank around the Glann Road Bridge for extreme events greater than 0.5% AEP. The calibration runs have suggested this is primarily from water from the Tonweeroe watercourse which flows overland to this area without entering the Owenriff. There is a connection

from the Owenriff to this area but capacity of the flow route is limited as it is flowing under the channel side wall (flow in 0.1% AEP event is modelled as 0.35 m³/s).

On the right bank is a care home that sits fairly close to the river. Flooding is not modelled as impacting the care home even in the 0.1% AEP event.

5.5.3 Flooding towards Lough Corrib

At the downstream extent of the Owenriff is a wide expanse of low lying land on the fringes of Lough Corrib. This area floods much more extensively from high lake levels than from the river but there is no property at risk here. Modelling the lake levels as peaking after the river means the risk from the lake levels is included for the same return period as the fluvial results.

5.5.4 Flooding from Tonweeroe watercourse

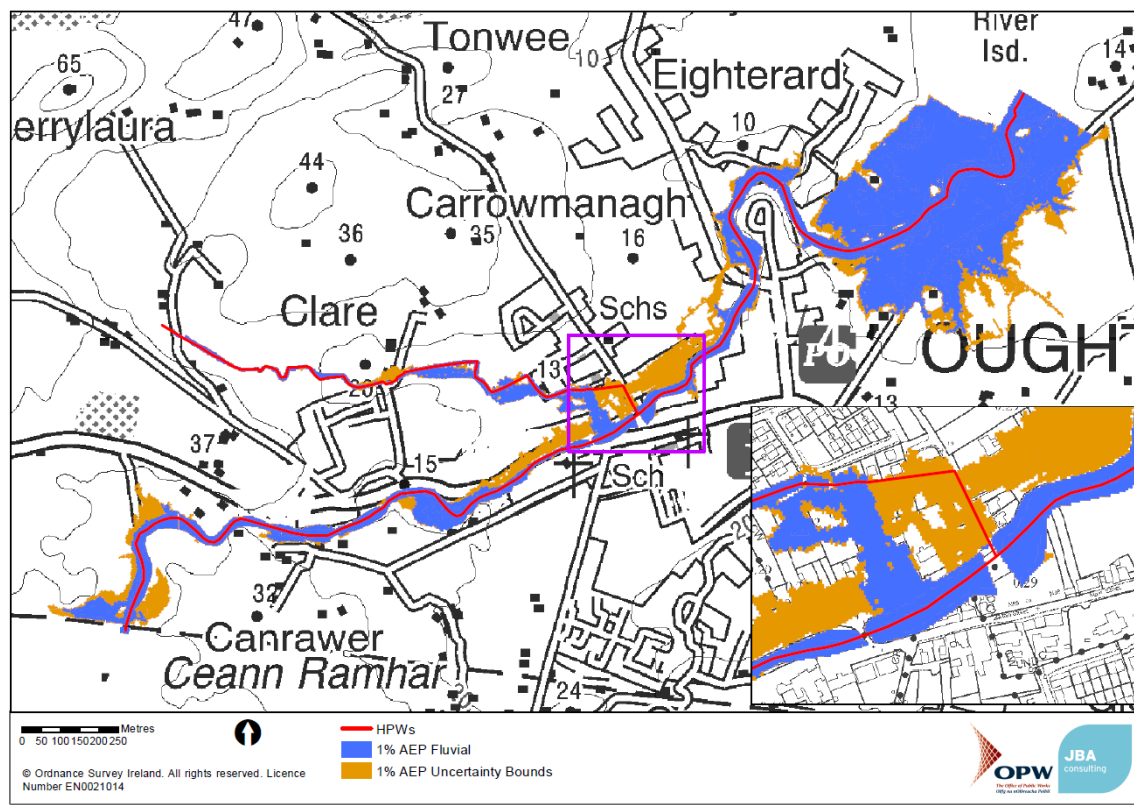
The most extensive property flooding modelled in Oughterard is associated with the Tonweeroe watercourse. The flooding appears primarily related to a small culvert along the lower reach of the watercourse, and particularly the final 130m culvert connecting to the Owenriff River that is undersized and cannot convey the design flows. Flood water flows out of the lower Tonweeroe channel towards property are initiated between the 10% and the 2% AEP event. This flooding has the potential to increase significantly in the event of culvert blockage which is fairly likely given the flat screen on the final (downstream) culvert. There is little evidence of geomorphological issues on this channel from the survey photos and no sign of siltation at the inlet of the final culvert.

The wall above the inlet to this culvert protects flooding to the properties to the west of the culvert entrance (Wall 3 in Figure 5-2). The performance of this wall as a flood defence is key in preventing flooding to properties in Abhainn na Coille and Carrowmanagh housing estates.

5.6 Sensitivity testing

Sensitivity testing was conducted to determine which variables the model was sensitive to and their respective impacts. The major increase in flood extent is seen in the magnified insert in Figure 5-3. This is primary due to the sensitivity result from the local Tonweeroe wall. This wall blocks a flow path that has the potential to flood a significant number of properties. The other increases in flood extent are as a result of the peak flow sensitivity, and the water level sensitivity at the downstream part of the Owenriff. This is an expected result due to the increase in flow and level for these tests. The building representation showed a minor increase, but was not deemed critical. The other tests showed no increase in flood extent.

Figure 5-3: 1% AEP event uncertainty bounds - Oughterard



5.7 Flood risk summary

Table 5-3: Flood risk to receptors in Oughterard

Risk Type	Receptor	Oughterard				
		10% AEP	1% / 0.5% AEP	0.1% AEP	1% AEP MRFS	1% AEP HEFS
Social	Residential property	0	4	21	4	7
	School	0	0	0	0	0
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	0	0	0	0
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	0	0	0	0
	Civil defence HQ	0	0	0	0	0
	Social amenity sites	Not at Risk	Not at Risk	At Risk	Not at Risk	At Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SPAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Cultural heritage	National monument	0	0	0	0	0
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.00	0.00	0.00	0.00	0.00
	National roads (km)	0.00	0.00	0.00	0.00	0.00
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	0	0	0
	Commercial building	0	0	2	0	1

6 Tuam

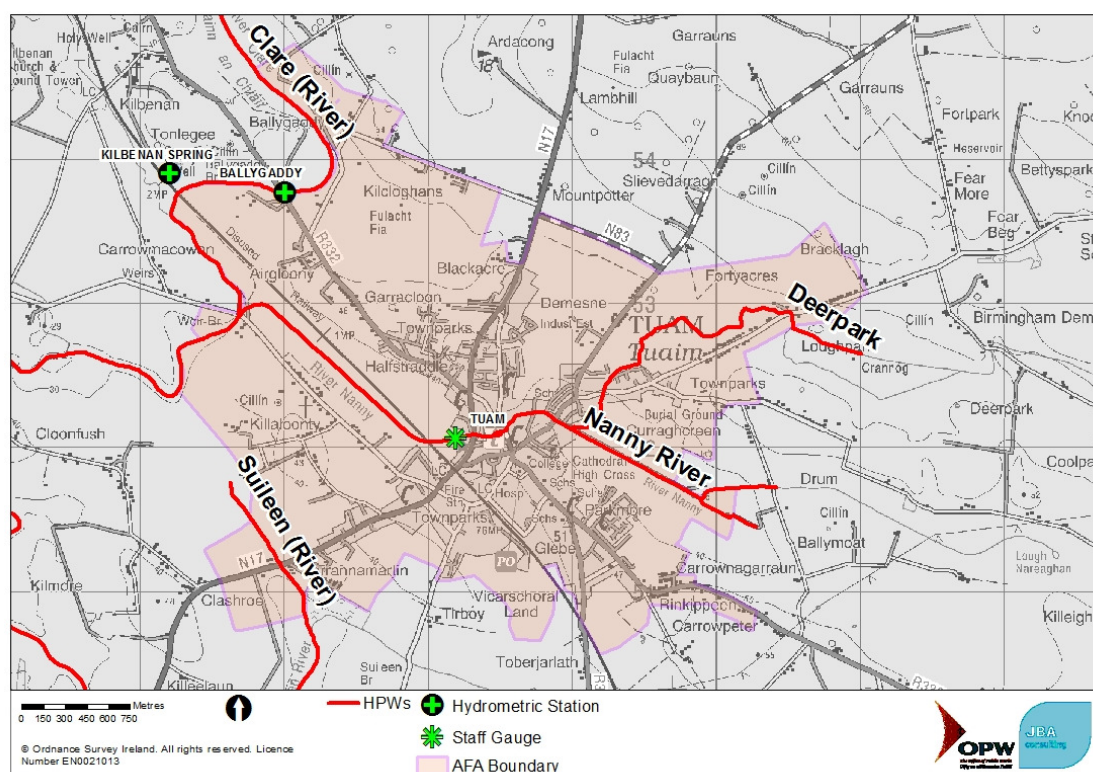
6.1 Watercourse and catchment overview

Three hydraulic models have been created for the Tuam AFA. One of these is of the River Clare which flows along the outskirts of the town. There is a gauge present at the upstream end of Clare River, Ballygaddy, with flood peak data from 1974. The River Clare is a large watercourse with extended floodplains. There has been historical flooding from the River Clare. Both upstream and downstream of the Clare HPW model at Tuam the River Clare is modelled as MPW.

The second model consists of the River Nanny and its tributaries; the Nanny Upper and Deerpark. These water courses flow through the AFA and Tuam town centre. There are many hydraulic structures in the centre of Tuam. These watercourses are much smaller than the River Clare and have no history of flooding. The River Nanny flows into the Clare River downstream of the Ballygaddy gauging station.

The last model is that of the Suilleen, a small watercourse to the South of the AFA. This watercourse flows into the River Clare downstream of the AFA. There is no evidence of historical flooding from this watercourse.

Figure 6-1: Tuam AFA catchment overview



6.2 Flood history

Further details on the flood history of the town can be found in the AFA specific modelling report and the Hydrology and Inception Reports. There have been reports of flooding within the Tuam AFA in 1950, 1991, 1995, 1999, 2002, 2005 and 2009. This flooding is mainly from the River Clare. There have also been some reports of flooding of the River Nanny along its downstream reach. This may be caused from the Clare River backing up into the Nanny floodplain.

6.3 Existing defences and walls

No formal or informal effective defences were identified within the AFA. Ten informal ineffective structures were identified within the AFA along the banks of the River Nanny, one along the banks its upper tributary and three along the banks of the Deerpark watercourse respectively. These

structures are not assumed to function as flood defences and are either bypassed in the model or have been removed to allow flooding beyond them.

6.4 Model limitations

- Hydrology - The Nanny and Suileen models are based on hydrology derived without direct gauge data in their respective catchments, compared with the River Clare which has a flow gauge on it. The combination of events on the different watercourses has not been directly modelled. The River Clare is a much larger river system than the Suileen or Nanny so we are not expecting joint events to occur.
- Channel blockage - In Tuam the culverts along the Nanny watercourse look particularly prone to blockage. If one of the culverts did block water would back up in the channel, before potentially causing flooding. However, detailed investigations of blockage and debris build-up have not been undertaken within the scope of the CFRAM.

6.5 Key flood risk mechanisms

Further to the information presented in the flood risk maps a brief description of the key flood risk sites and flooding mechanisms is provided below.

6.5.1 River Nanny and its tributaries

During the 0.1% AEP event properties on the right bank downstream of Shop Street are at risk of flooding from the River Nanny. Downstream of the Garda station on Abbey Trinity Road there is flooding on the right bank during the 1% and 0.1% AEP events. The extent of flooding in this area is constrained by the right bank of the land drain.

There is limited flooding on the downstream reach of the River Nanny from the River Nanny itself. Historically there has been flooding in this area but it is modelled as arising from the River Clare.

There is a large area of flooding on left bank of the Deerpark watercourse just upstream of its confluence with the River Nanny. The majority of this flooding is a result of a low point on the left bank between cross sections 30DEER00031 and 30DEER00025. In the 1% AEP event, flow from the Deerpark River is diverted to the River Nanny via this floodplain. No properties are affected from flooding in this area.

Further upstream where the Deerpark watercourse flows under Bothar na Greanna, properties are predicted to be at risk of flooding in the 0.1% AEP event.

6.5.2 Clare River

The Clare River is low lying and flooding is predicted along the majority of its reach on the left and right bank for flows as low as the 50% AEP event. In the 0.1% AEP event 18 properties are predicted at risk of flooding, on the left bank upstream of the Ballygaddy gauging station. In the 0.5% AEP event only one of these 18 properties is predicted at risk of flooding.

Flow from the Clare River inundates the flood plains of the Kilbenan river and the Nanny river during the Q2 event and greater. Although no properties within the AFA are predicted to be a risk of flooding along these flood plains up to the 0.5% AEP event, approximately 58 properties on the left bank of the river Nanny are predicted to be a risk of flooding in the 0.1% AEP event.

6.5.3 Suileen River

There is some flooding on the right and left bank of the Suileen water course in the Tuam AFA. There are not properties in the areas predicted to flood.

6.6 Sensitivity testing

Sensitivity testing was conducted to determine which variables the model was sensitive to and their respective impacts.

On the River Clare, Nanny, the results of the sensitivity testing show the peak flow is the greatest uncertainty associated with this model and the test produces the largest extent in all locations. The River Suileen also shows some sensitivity to peak flow, particularly in the 1% AEP event, and is also sensitive to the water level boundary at the downstream end.

Figure 6-2: 1% AEP fluvial uncertainty bound for the River Clare - Tuam AFA

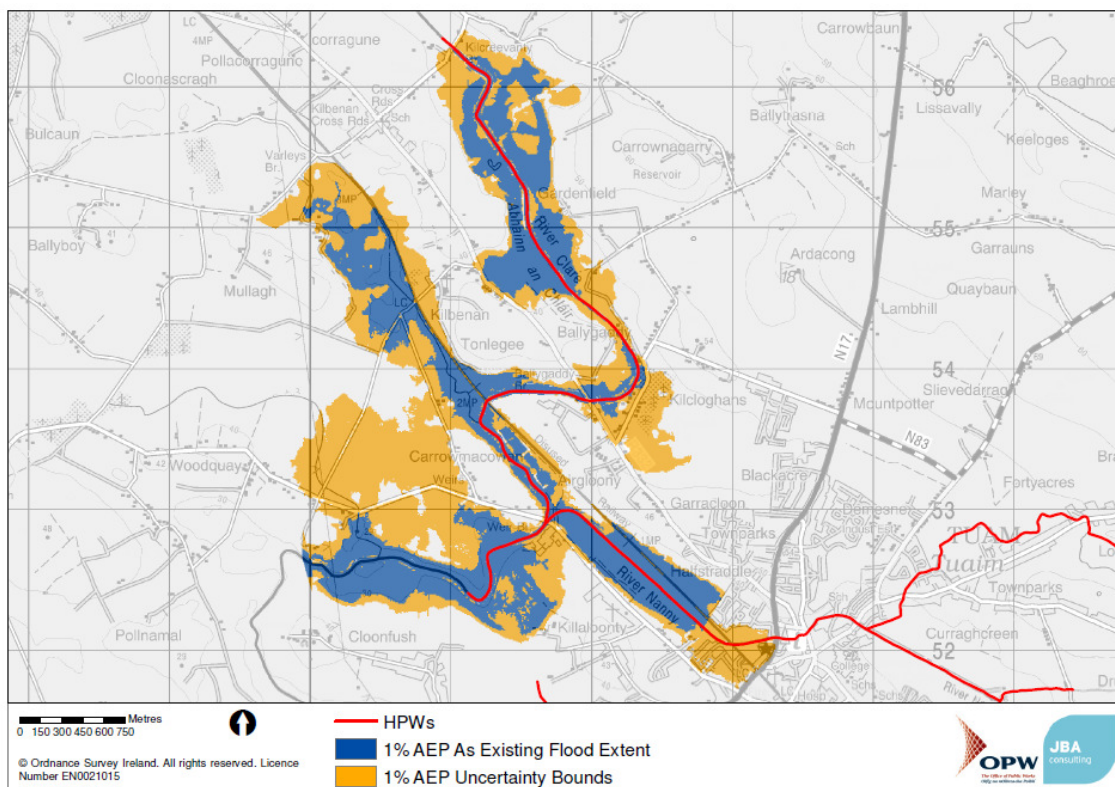


Figure 6-3: 1% AEP fluvial uncertainty bound for the River Nanny - Tuam AFA

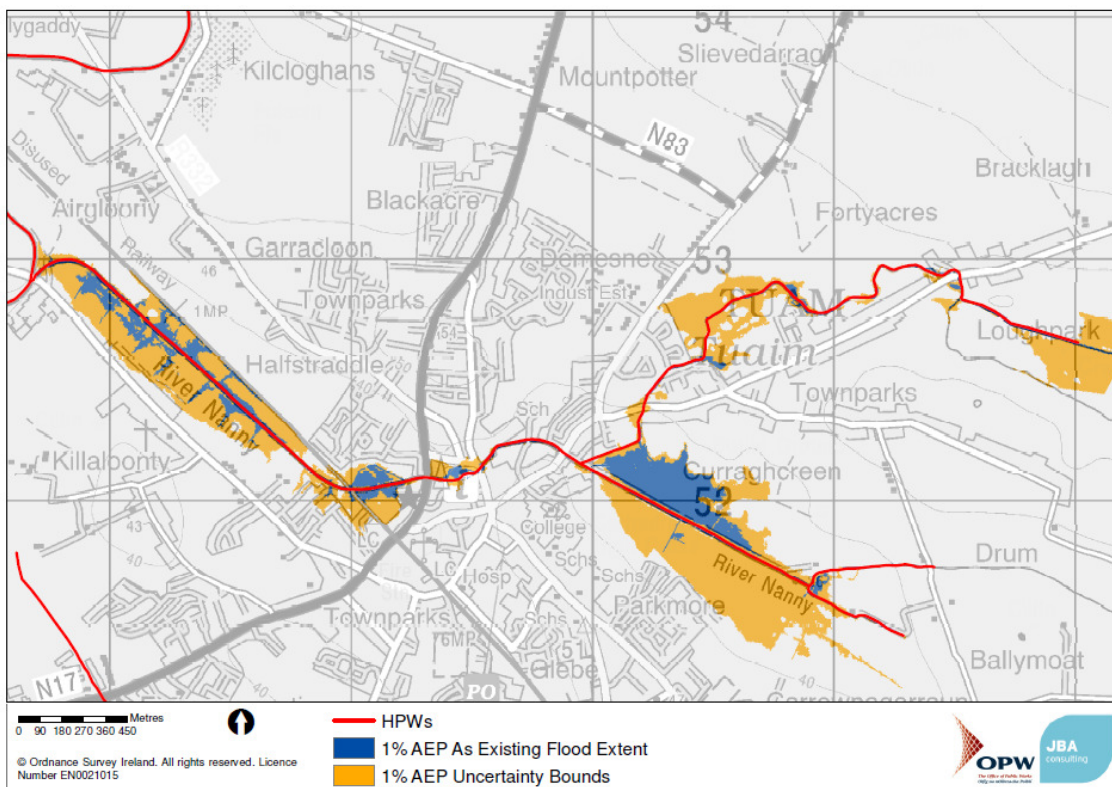
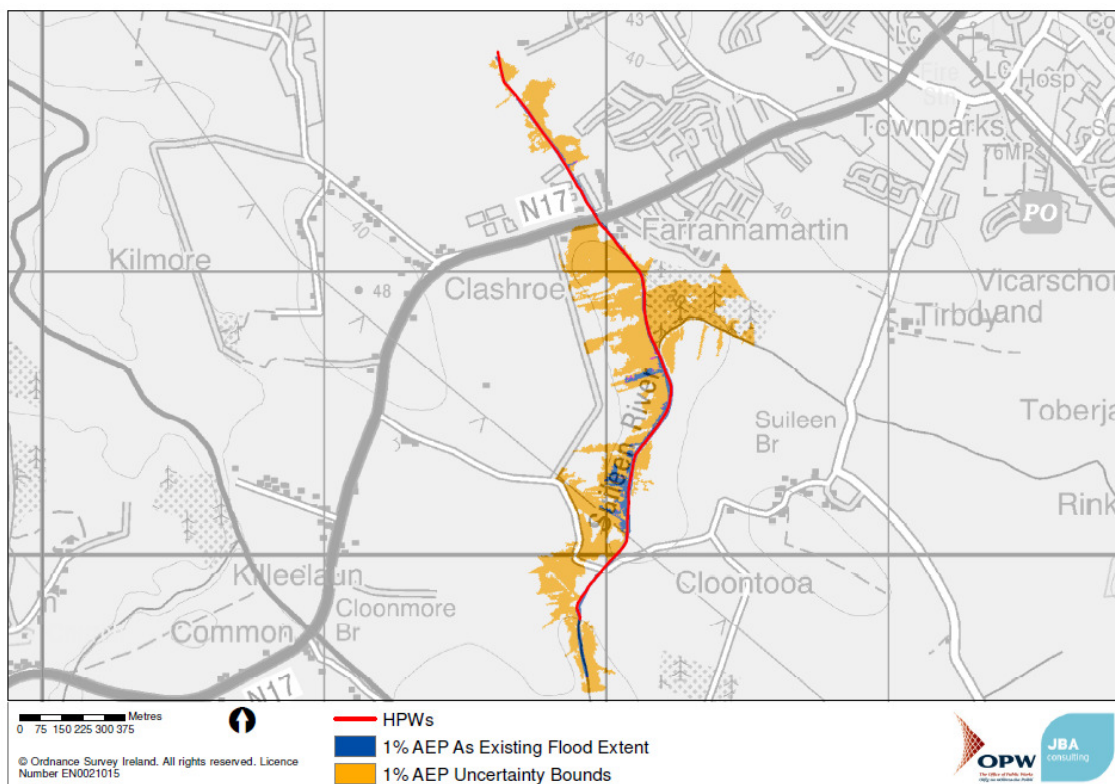


Figure 6-4: 1% AEP fluvial uncertainty bound for the River Suileen - Tuam AFA



6.7 Flood risk summary

Table 6-1: Flood risk to receptors in Tuam

Risk Type	Receptor	Tuam				
		10% AEP	1% / 0.5% AEP	0.1% AEP	1% AEP MRFS	1% AEP HEFS
Social	Residential property	0	0	92	4	38
	School	0	0	0	0	0
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	0	0	0	0
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	0	0	0	0
	Civil defence HQ	0	0	0	0	0
	Social amenity sites	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SPAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Cultural heritage	National monument	1	1	1	1	1
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.00	0.69	1.61	1.03	1.19
	National roads (km)	0.00	0.00	0.00	0.00	0.00
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	1	1	1
	Commercial building	0	1	9	3	3

7 Roundstone

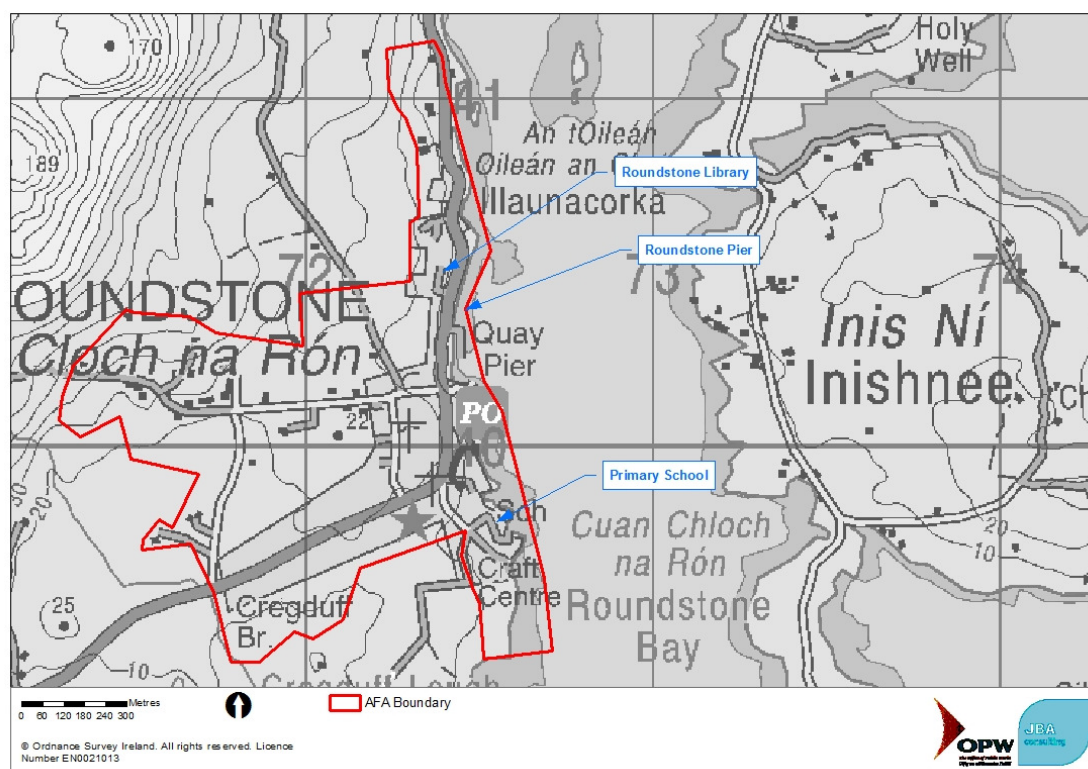
7.1 Watercourse and catchment overview

Roundstone has a coastline of approximately 1.8km within the boundaries of the AFA. The key areas of flood risk within the AFA are properties in the centre of the village, particularly adjacent to the pier.

There is a quay wall that extends north from the pier along the R341 to the outskirts of the village. The ground elevation along the pier is approximately 3.00mOD Malin. The elevation increases to approximately 5.00mOD on the R341 behind the quay wall. The quay wall provides some protection to seaward properties. However, the wall is of old, masonry construction and has some gaps in it. Its structural integrity could be compromised if subjected to extreme water levels. The wall's maximum crest level is 5.04mOD Malin, at the corner of the pier, before falling to a minimum of 3.19mOD Malin approximately 100m to the north. There are no bridges or culverts within the AFA that will impact upon the tidal flooding extents.

The frequency of flooding is relatively high within Roundstone. It can be as frequent as once or twice a year for the R341 road. Flooding within the village is due to a combination of high tides and storm surge, exacerbated by wave action. Floor levels have been raised in properties adjacent to the southern jetty in the centre of the village. Critically, the same cannot be said of the properties directly behind the quay wall in the north of the village.

Figure 7-1: Roundstone AFA catchment overview



7.2 Flood history

Further details on the flood history of the town can be found in the AFA specific modelling report and the Hydrology and Inception Reports. A summary of known flood events is provided below.

Table 7-1: Flood history summary in Roundstone

Area affected	Main flood mechanisms	Recorded flood event date	Use in model check
Roundstone Quay	High tides	January 2014	Sensibility checking of model
Roundstone Quay	High tides	2012	Background information

Area affected	Main flood mechanisms	Recorded flood event date	Use in model check
Roundstone Quay	High tides	Recurring	Corroborates 2014 validation run
R341 (North AFA)	High tides	Recurring	Corroborates 2014 validation run
Monastery Road	High tides	Recurring	Corroborates 2014 validation run

7.3 Existing defences and walls

No formal effective defences were identified within the AFA. However the quay wall present in the village was identified as being an informal effective defence, despite being of low structural integrity. No informal ineffective structures were identified within the AFA.

Figure 7-2: Roundstone Quay Wall



7.4 Model limitations

- Gap in quay wall - Due to model stability and limitations of the grid resolution, the gap has been modelled as a 2m wide opening but is approximately 1m wide in reality. Given the small area that fills behind the wall, the water level across the wall will equalise at the event peak making the size of gap less significant. The impact of this change was also investigated and documented through the sensitivity testing.

7.5 Key flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below.

7.5.1 Flooding at Roundstone pier and quay wall

The model shows that flooding within the village begins with flow coming up through the gap in the quay wall onto the R341 road. The northern extent of the village is at particular risk and this has been confirmed by the flooding on the 3rd of January 2014. In the 0.1% AEP event, there are approximately 11 different properties at risk. The wall in front of these properties is also bypassed from the north.

7.5.2 Flooding at Roundstone National School

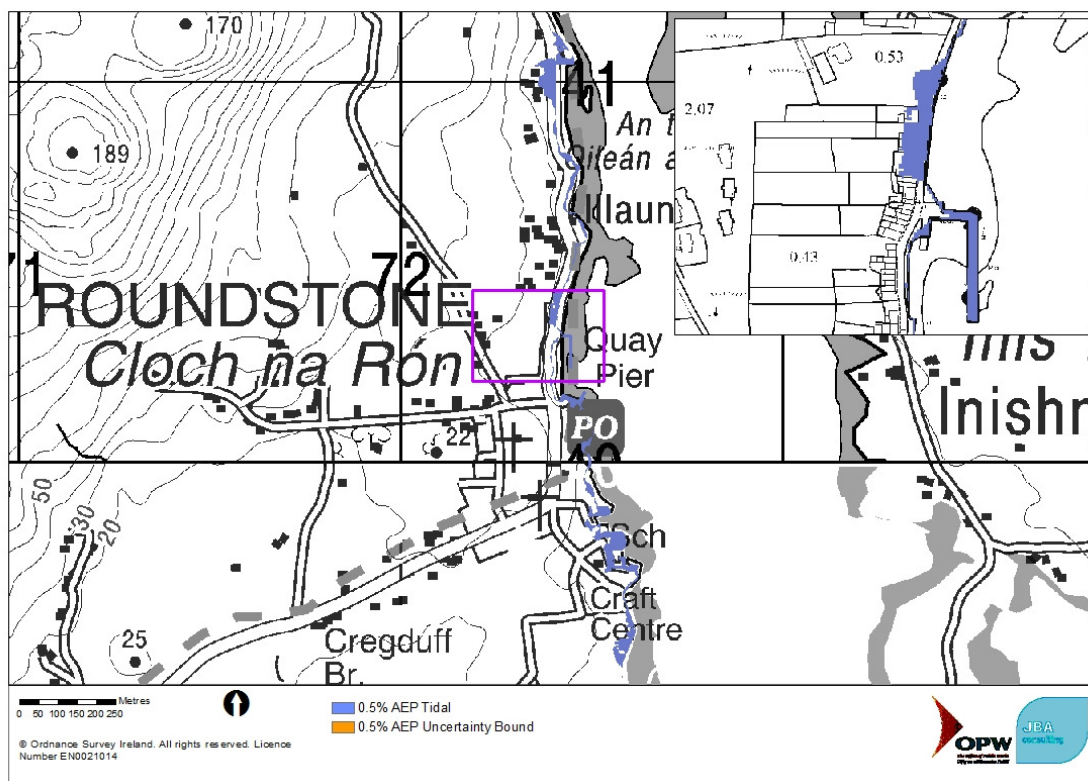
Although the Roundstone National School was not seriously affected in January 2014, risk is likely to be more significant in larger events and particularly in the MRFS and HEFS scenarios. Risk in this location is solely dictated by elevated water levels.

7.6 Sensitivity testing

Sensitivity testing was conducted to determine which variables the model was sensitive to and their respective impacts. Upon conducting these tests it was determined that the model was not sensitive to the adjustment of the various parameters. The reason is that the model simply runs

the design water level to a particular contour on the DTM. This process is not affecting by changing the parameters as they are not variables. Closing of the gap in the quay wall transfers the point at which flow makes land further north along the R341.

Figure 7-3: 0.5% AEP tidal event uncertainty bounds - Roundstone AFA



7.7 Flood risk summary







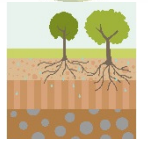
























Table 7-2: Flood risk to receptors in Roundstone





































Risk Type	Receptor	Roundstone				
		10% AEP	0.5% AEP	0.1% AEP	0.5% AEP MRFS	0.5% AEP HEFS
Social	Residential property	0	2	9	24	31
	School	0	0	0	1	1
	Health centre	0	0	0	0	0
	Nursing home	0	0	0	0	0
	Public residential care home	0	0	0	0	0
	Social infrastructure	0	0	0	0	0
	Hospital	0	0	0	0	0
	Gardai station	0	0	0	0	0
	Fire station	0	0	0	0	0
	Civil defence HQ	0	0	0	0	0
	Social amenity sites	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Environment	IED / IPPC sites	0	0	0	0	0
	WFD Annex IV sites	At Risk	At Risk	At Risk	At Risk	At Risk
	NHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	pNHAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SACs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
	SPAs	Not at Risk	Not at Risk	Not at Risk	Not at Risk	Not at Risk
Cultural heritage	National monument	0	0	0	0	0
	UNESCO heritage site	0	0	0	0	0
	Museum/ gallery	0	0	0	0	0
	NIAH building	0	0	0	0	0
Economy	Airport	0	0	0	0	0
	Train station	0	0	0	0	0
	Railway line (km)	0.00	0.00	0.00	0.00	0.00
	National roads (km)	0.00	0.00	0.00	0.00	0.00
	Water treatment plant	0	0	0	0	0
	Utilities	0	0	0	0	0
	Commercial building	0	0	2	5	5

8 UoM Summary

The table below summarises flood risk to the AFAs within UoM30-31.

Table 8-1: Summary of flood risk to AFAs

Parameter	Ballyhaunis	Corrofin	Galway City	Oughterard	Tuam	Roundstone
Primary source of risk:  Fluvial,  Coastal,  Wave overtopping  Groundwater (note, groundwater has not been assessed through the CFRAM).		 	  			
Receptors at risk in the 1% AEP fluvial or the 0.5% AEP coastal event.  Property  Infrastructure  Rural Land Use  Economic Risk  Activity At Risk  Activity Not At Risk						
Effective defences						

Parameter		AFA					
		Ballyhaunis	Corrofin	Galway City	Oughterard	Tuam	Roundstone
Current flood risk:	10% AEP						
	1%/0.5% AEP						
	0.1% AEP						
Future MRFS flood risk:	10% AEP						
	1%/0.5% AEP						
	0.1% AEP						



Registered Office

**24 Grove Island
Corbally
Limerick
Ireland**

t: +353 (0) 61 345463
e: info@jbaconsulting.ie

**JBA Consulting Engineers
and Scientists Limited**

Registration number 444752



Visit our website
www.jbaconsulting.ie

Western CFRAM Unit of Management 30 and 31

**Final Hydraulic Model Report:
Volume 1a: Hydraulic Modelling Method
Statement**

September 2016

**Office of Public Works
Trim
Co. Meath**



**WESTERN
CFRAM
STUDY**
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

JBA Consulting

24 Grove Island
Corbally
Limerick
Ireland

JBA Project Manager

Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM

Revision History

Revision Ref / Date Issued	Amendments	Issued to
Version 1.0 26/01/2014	Initial Issue	Rosemarie Lawlor
Version 2.0 12/09/2014	Responding to OPW comments	Rosemarie Lawlor
Version 3.0 11/09/2016	Final Issue	Clare Butler

Contract

This report describes work commissioned by The Office of Public Works, by a letter dated (28/07/11). The Office of Public Works' representative for the contract was Rosemarie Lawlor. Sam Willis, Elizabeth Russell, Chris Smith and Tom Sampson of JBA Consulting carried out this work.

Prepared Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM
 Elizabeth Russell BSc MSc CEnv MCIWEM C.WEM
 Reviewed by Chris Smith BSc PhD CEnv MCIWEM C.WEM MCI
 Jonathan Cooper BEng MSc DipCD CEng MICE MCIWEM C.WEM MIO

Purpose

This document has been prepared as a draft report for The Office of Public Works. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to the Office of Public Works.

Copyright

Copyright – Copyright is with Office of Public Works. All rights reserved. No part of this report may be copied or reproduced by any means without the prior written permission of the Office of Public works.

Legal Disclaimer

This report is subject to the limitations and warranties contained in the contract between the commissioning party (Office of Public Works) and JBA.

Carbon Footprint

A printed copy of the main text in this document will result in a carbon footprint of 140g if 100% post-consumer recycled paper is used and 178g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to achieve carbon neutrality.

Contents

Legal Disclaimer	ii
1 Introduction	1
1.1 Scope of report	1
1.2 Report overview	1
1.3 Study background	2
1.4 Geometric data	3
2 Fluvial hydraulic modelling	5
2.1 Modelling approaches	5
2.2 1D model development	6
2.3 2D model development	10
3 Coastal hydraulic modelling	14
3.1 Modelling Approach	14
3.2 Coastal model development	14
3.3 Wave overtopping	14
4 Defence modelling	17
4.1 Classification and modelling of raised structures adjacent to watercourses	17
4.2 Undefined modelling	18
4.3 Breach analysis	19
5 Model calibration and sensibility checking	20
5.1 Objectives and categorisation	20
6 Application of hydrology	21
6.1 Hydrological estimation points	21
6.2 Application of design flow estimates	21
7 Sensitivity testing	24
7.1 Screening	24
7.2 Sensitivity analyses	25
7.3 Joint probability analysis	29
7.4 Sensitivity testing results	29
8 Model outputs and mapping	30
8.1 Model run scenarios and design events	30
8.2 Flood hazard mapping	30
8.3 Long section plots	31
8.4 Presentation of uncertainty	31
8.5 Flood risk maps	32

List of Figures

Figure 1-1: HPW and MPW modelled watercourses within UoM 30 and 31	3
Figure 2-1: Schematisation of cross sections in a typical MPW model	5
Figure 2-2: Schematisation of a typical HPW model	6
Figure 2-3: Typical section of a weir within bank	9
Figure 8-1: Sensitivity run example outputs	32

List of Tables

Table 1-1: AFAs within the UoM	2
Table 1-2: Shortlist of culverts for CCTV survey	4
Table 2-1: Cross section survey label structure suffix codes	7
Table 2-2: Typical roughness values for river channels	8
Table 2-3: Typical roughness values for river banks	8
Table 2-4: 2D model floodplain roughness values	12
Table 2-5: Overtopping spill coefficients	13
Table 4-1: Ineffective structure classifications	17
Table 4-2: Breach analysis screening results	19
Table 7-1: Sensitivity tests	24
Table 7-2: Flow sensitivity test scoring mechanism	25
Table 7-3: Flow sensitivity scaling factors	25
Table 7-4: Roughness bounds for river channels	26
Table 7-5: Roughness bounds for river banks	26
Table 7-6: Roughness bounds for floodplain surfaces	26
Table 7-7: Flood duration multipliers for flow volume sensitivity test	27
Table 7-8: Coefficients for contraction and expansion head losses	28
Table 7-9: Roughness bounds for culverts	28
Table 8-1: Allowances for future scenarios	30
Table 8-2: Flood mapping requirements	30
Table 8-3: Risk map receptors	32

Abbreviations

AEP	Annual exceedence probability
AFA	Area for further assessment
AMAX	Annual maximum
CFRAM	Catchment flood risk assessment and management
DAD	Defence asset database
DAS	Defence asset survey
DEM	Digital elevation model (Includes surfaces of structures, vegetation, etc)
DTM	Digital terrain model ('bare earth' model; does not include surfaces of structures, vegetation, etc
ESTRY	One-dimensional model from the TUFLOW suite
FRISM	Flood risk metrics (a flood risk tool developed by JBA)
FRMP	Flood risk management plan
FRR	Flood risk review
FSR	Flood studies report
FSU	Flood studies update
GIS	Geographical information system
HEFS	High-end future scenario
HEP	Hydrological estimation point
HPW	High priority watercourse
HWA	Hydrograph width analysis
IBIDEM	Interactive bridge invoking the design event method
ICPSS	Irish coastal protection strategy study
ISIS	One-dimensional hydraulic modelling software
LA	Local authority
LIDAR	Light detection and ranging
mOD	Metres above Ordnance datum (unless stated this refers to the Malin datum)
MPW	Medium priority watercourse
MRFS	Mid-range future scenario
NDHM	National digital height model (a DTM by Intermap)
OSi	Ordnance Survey Ireland
PFRA	Preliminary flood risk assessment
Q(T)	Flow for a given return period
QMED	Median annual flood, used in FSU methods
SAAR	Standard annual average rainfall
SoP	Standard of protection (in relation to flood defences)
T	Return period, inverse of AEP
Tp	Time to peak
TUFLOW	Two-dimensional hydraulic modelling software

UoM Unit of Management

* Asterisks at the end of a cross section label denotes interpolated model cross sections

1 Introduction

1.1 Scope of report

This report details the generic hydraulic modelling methodologies applied in the modelling phase of the Western Catchment-Based Flood Risk Assessment and Management Study (WCFRAM). The report is therefore applicable for all Areas for Further Assessment (AFAs) within Unit of Management (UoM) 30-31, Corrib and Owengowla, but in general does not go into detail regarding the specifics associated with a given AFA. Modelling assumptions specific to an AFA are discussed in the relevant AFA modelling reports.

The report covers the overall hydraulic modelling processes from model build through to the development of design runs.

Whilst it has been necessary to develop a general methodology for the hydraulic models across the WCFRAM to ensure consistency, it is not possible to pre-empt the approach required at all locations. This document does therefore not preclude changes to the approaches, which are applied at an AFA level where appropriate. Where local knowledge justifies an alternative approach, this will be reported in the AFA modelling report. However, the AFA reports do not duplicate the generic methodology detailed in this report.

1.2 Report overview

This report is one of a series which describe the work undertaken as part of the CFRAM, and together they provide a description of the approach taken to identifying flood risk, and a discussion of the results of the analysis and potential flood management measures, where they are appropriate.

This report should be read in conjunction with the following supporting documents:

- Western CFRAM UoM30-31 Hydrology Report¹
- Western CFRAM UoM30-31 Inception Report²
- Western CFRAM Flood Risk Review Report³
- Western CFRAM SEA Scoping Report⁴
- Western CFRAM Defence Asset Database: Handover Report and accompanying database files⁵

The reports in the suite for the Hydraulic Modelling are:

- Western CFRAM UoM30-31 - Hydraulic Modelling Report
- Western CFRAM UoM30-31 - Hydraulic Modelling Report: Volume 1a – Hydraulic Modelling Method Statement (this report)
- Western CFRAM UoM30-31 - Hydraulic Modelling Report: Volume 1b – Hydromorphology and Coastal Erosion Assessment
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 2 – AFA Modelling Report (for example – 2a - Ballyhaunis AFA Modelling Report)
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 3 – Flood Risk Maps (for example - 3a Ballyhaunis Flood Risk Maps)
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 4a – Hydraulic Model and Check File (for example - 4a Ballyhaunis Hydraulic Model and Check File)

The letter code associated with the deliverables in Volumes 2, 3 and 4 will be consistent for a given AFA, so in the example above the letter 'a' applies to the Ballyhaunis AFA. Volume 4 is

¹ JBA Consulting (2014), Western CFRAM Unit of Management 30 and 31 – Corrib and Owengowla Hydrology Report, Final Report, Office of Public Works

² JBA Consulting (2012), Western CFRAM Unit of Management 30-31 – Corrib and Owengowla Inception Report, Final Report, Office of Public Works.

³ JBA Consulting (2012), Western CFRAM Flood Risk Review, Final Report, Office of Public Works.

⁴ JBA Consulting (2013), Western River Basin District Catchment-based Flood Risk Assessment and Management (CFRAM) Strategic Environmental Assessment, Scoping Report, Office of Public Works.

⁵ JBA Consulting (2013), Western CFRAM Defence Asset Database, Handover Report, Office of Public Works.

the technical output from the study and will only be available on request from the Office of Public Works.

The AFAs covered by this report, and the corresponding models, are listed in Table 1-1

Table 1-1: AFAs within the UoM

UoM	Model type: Fluvial, Coastal (with Wave Overtopping), MPW, Undefended, Breach	AFA / MPW code	AFA / MPW name	Model codes within AFA / MPW
30	F	BLH	Ballyhaunis	D1
30	MPW	MWB	Ballyhaunis to Tuam	92
30	F	CRF	Corrofin	L1
30	F, U	GLW	Galway City	N1
30	C (WO), U	GLW	Galway City Coastal	C1
30	F	OTD	Oughterard	U1
30	F	TUM	Tuam	Y1-3
30	MPW	MWT	Tuam to Lough Corrib	90
31	C	RSN	Roundstone Coastal	C4

1.3 Study background

The Inception Report for UoM 30 and 31 was delivered in October 2012. This report consisted of a baseline review of available data and the development of the proposed methodology for the hydrological and hydraulic modelling investigations to be completed within this phase.

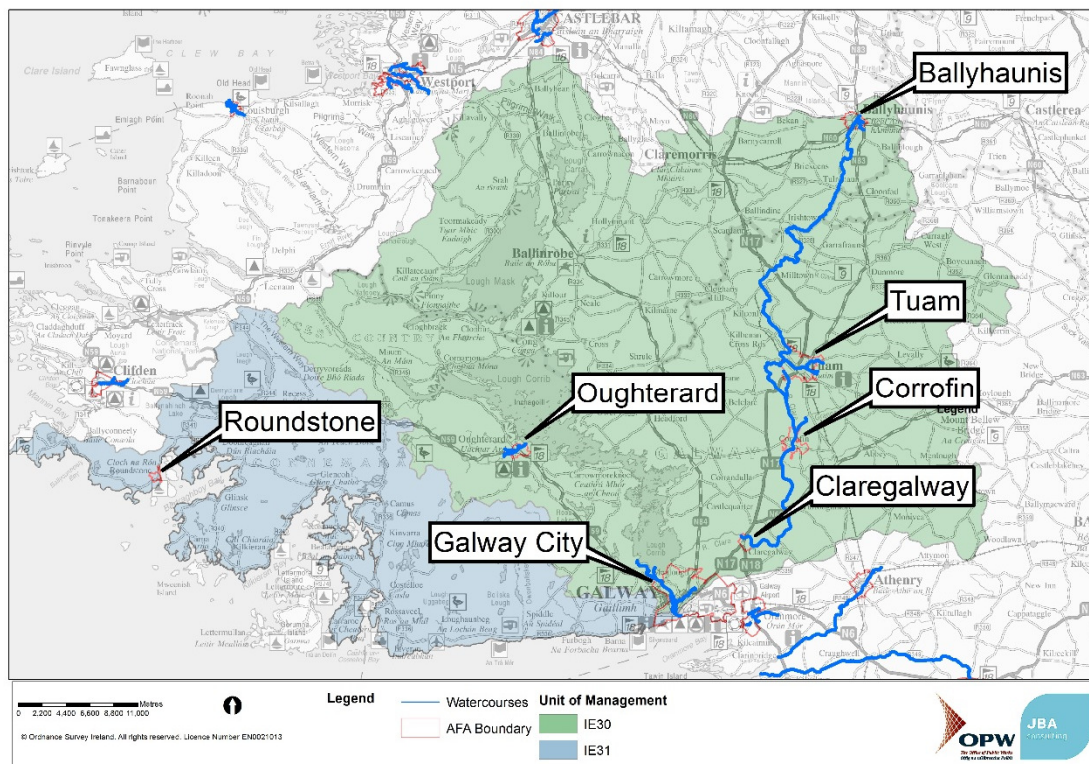
The method statement for the hydrological analysis detailed in the Inception Report has been developed and finalised in the UoM 30 and 31 Hydrology Report. This work has developed design flows at a series of Hydrological Estimation Points (HEPs) along all watercourses to be modelled. The detailed development of the hydrology has not been reiterated here and the reader is referred to the Hydrology Report for full details of the hydrological analysis. Design flows have been extracted directly from the Hydrology report and are summarised in the various editions of Volume 2, relating to the specific AFAs.

The Hydrology Report also provides guidance on the development of appropriate design storm hydrographs for each AFA for the purposes of the hydraulic modelling. These methods are summarised in this report to provide clarity on the application of the design event hydrology as this work has been undertaken in the hydraulic modelling phase.

The Inception Report identified all High Priority and Medium Priority Watercourses (HPWs and MPWs) to be modelled. HPWs are those watercourses that dictate flood risk within an AFA boundary as originally delineated within the Flood Risk Review (FRR) Report. HPWs therefore extend a short distance upstream and downstream of an AFA but do not include watercourses with catchments less than 1km². HPWs have been modelled to a greater level of detail than MPWs. MPWs are the watercourses which link two AFAs together and the watercourses that extend downstream of an AFA to the sea. Coastal AFAs do not have a downstream MPW associated with them.

In total, 56 km of HPW and 102 km of MPW have been modelled within UoM 30 and 31, along with the coastline in Roundstone and Galway City, Figure 1-1.

Figure 1-1: HPW and MPW modelled watercourses within UoM 30 and 31



1.4 Geometric data

1.4.1 Topographic Survey data

The hydraulic models have been constructed from topographic survey of the river channels and ground level survey of the floodplain.

Topographic survey has been collected as cross sections perpendicular to the direction of flow at regular intervals along watercourses and along the faces of key structures, and as spot level survey along the bank tops between cross sections. Cross sections have been surveyed at 50-100m intervals along HPWs and 500m-1,000m intervals along MPWs. The spacing of the bank top survey was 10m, with additional points collected where elevations changed by 250mm or more. No bank top survey has been collected along MPWs.

The survey data was reviewed as part the QA process for the relevant survey contract, and the review certificates were included as part of the survey report deliverables. Additional checks were undertaken as part of the model development. These included checking all structures had been surveyed, the full length of reaches had been covered and sufficient detail was surveyed on sluices and other complex structures. A comparison between surveyed levels and LIDAR has also been carried out, and is reported on in the relevant AFA modelling reports.

The main topographic survey was collected over three survey contracts and data delivered between December 2012 and June 2013. Further infill survey contracts were commissioned to collect additional survey data where it was required to supplement the three major survey contracts, as follows.

- National Survey Contract No. 6, by CCS - July 2012 - February 2013
- Western Survey Contract 1 - Maltby Land Surveys - June 2012 - October 2012
- Western Survey Contract 2 - Murphy Surveys Limited - November 2012 - July 2013
- Infill Survey Contract 4 - CCS Surveys - August 2013
- Infill Survey Contract 6 - Blom 6West - December 2013
- Infill Survey Contract 7 - Murphy Surveys - January 2014
- Infill Survey Contract 9 - 6West - May 2015

The cross section survey key plan for each model reach is included in the maps contained in Volume 3 of this report.

1.4.2 Digital terrain model

Ground level survey is available from LIDAR data for AFAs only, so covers HPWs and associated floodplains. Data has been provided in both filtered and unfiltered formats in a 2m grid resolution. The LIDAR was flown between November 2011 and August 2012. For MPWs, floodplain data has been extracted from a coarse Informar Digital Terrain Model (DTM). This is the Office of Public Work's National Digital Height Model (NDHM), flown between 2007 and 2009. This 5m resolution DTM was supplied by the Office of Public Works in 2013.

A comparison between surveyed levels and LIDAR has also been carried out, and is reported on in the relevant AFA modelling reports. In all cases, the LIDAR and survey matched to an acceptable level and no AFA scale changes were required.

1.4.3 CCTV survey

Where long and non-uniform culverts were identified along river systems, the need for CCTV survey was considered as follows:

- Is further information needed to model the culvert?
- Is this information available from other sources (design drawings or previous CCTV survey for example)?
- If a blockage occurred, would properties be at risk of flooding?
- Is connectivity between the upstream and downstream faces confirmed?
- Can sufficient additional information be gathered through a detailed engineering site visit?

Table 1-2 lists the culverts which were considered on the basis detailed above. In all cases it was determined that sufficient data was available to allow the culvert to be modelled appropriately without requiring CCTV survey.

Table 1-2: Shortlist of culverts for CCTV survey

AFA Name	Upstream Culvert XS_ID	CCTV survey commissioned
Ballyhaunis	30DEVL00011I	No
Oughterard	30TONW00016I	No
Oughterard	30TONW00019I	No
Oughterard	30TONW00037I	No
Oughterard	30TONW00040I	No

2 Fluvial hydraulic modelling

2.1 Modelling approaches

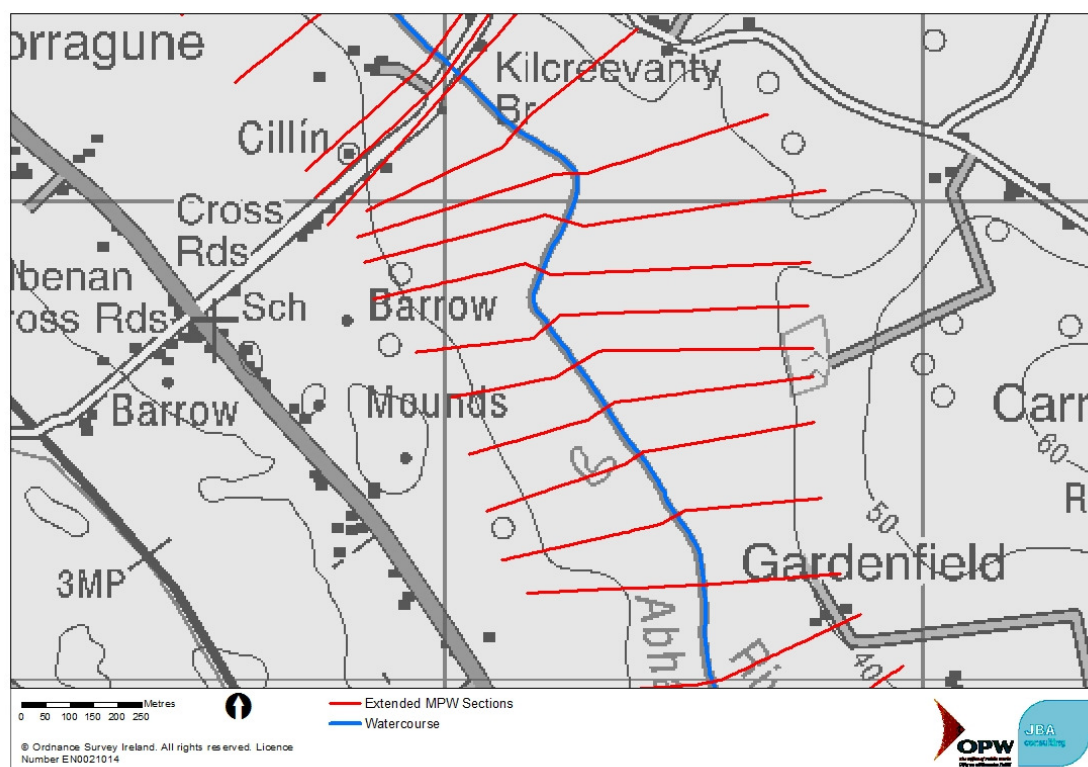
Different modelling approaches have been adopted for HPWs and MPWs. The outputs from the HPW models are to a greater level of detail and accuracy than those from MPW models, reflecting the focus of the WCFRAM study on those areas where the greatest numbers of receptors are located. This increased level of detail is reflected in the quality and quantity of the survey data collected for each watercourse and also in the modelling methodologies described below.

2.1.1 MPW models

MPW models have been constructed using only cross section survey data in the software package ISIS; this is referred to as 1 dimensional or 1D modelling. Cross sections for these models are located at 500m-1000m intervals and at key hydraulic structures, such as bridges, embankments and significant weirs. To represent inundation of the floodplain, the surveyed cross sections have been extended using data from the Infomar 5m grid.

Figure 2-1 provides an example of the combined cross sections, the surveyed sections are the elements spanning the channel, and are perpendicular to the watercourse. The extended sections are aligned so that they are perpendicular to the flow in the floodplain. This approach results in the dog leg effect along the banks of the watercourse for each cross section shown in the figure below.

Figure 2-1: Schematisation of cross sections in a typical MPW model



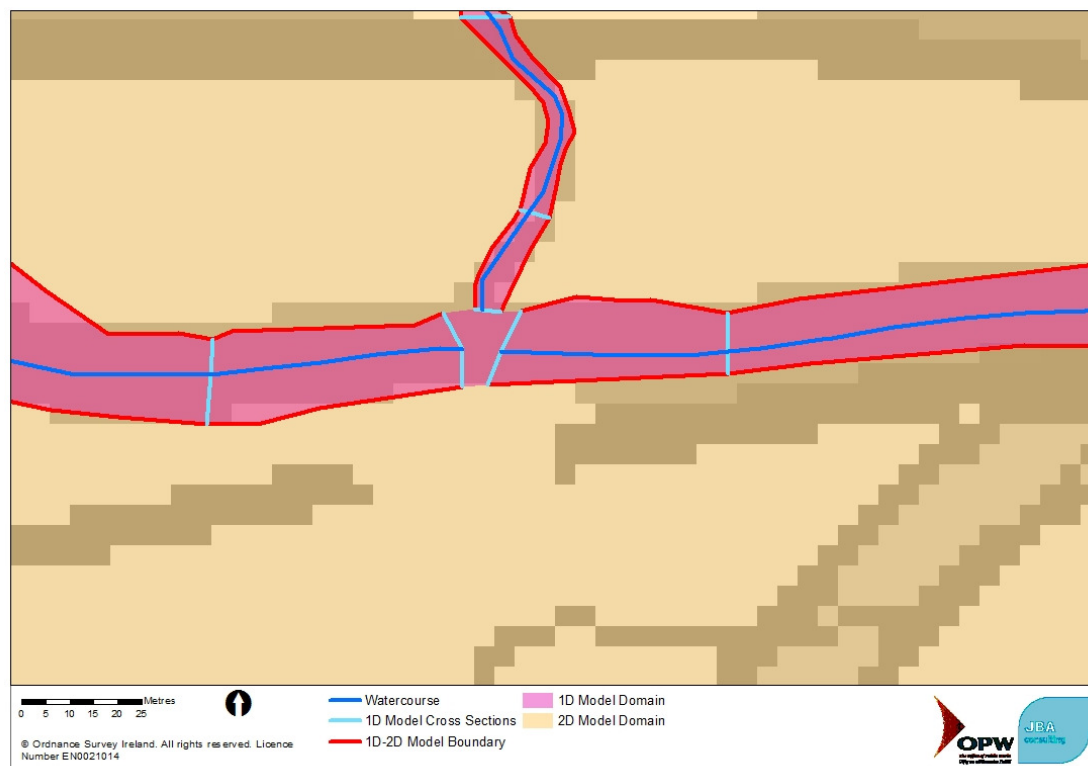
The low frequency of model cross sections and the coarse resolution of the NDHM grid result in a higher level of uncertainty associated with the MPW model outputs when compared with HPW models.

2.1.2 HPW models

HPW models have been constructed using both the cross section survey to represent the channel and the LIDAR data to represent the floodplain, rather than extended cross sections. This approach uses two software packages, ISIS and TUFLOW, and is referred to as a linked 1

dimensional and 2 dimensional (or 1D-2D) model, where 2D refers to the modelling of flow in the floodplain. Cross sections are located at approximately 50-100m intervals and at all structures; as with the MPWs, these form the basis of the 1D model. The 1D model consists of the river channel itself and generally extends to the top of the river bank. The 2D model consists of the floodplain beyond the river channel (as represented in the 1D model) and has been developed from the LIDAR data, which forms a grid of floodplain levels rather than the cross section levels used in the MPW models. Bank top survey collected along HPWs provides greater detail at the interface of the 1D and 2D models (or the river channel and floodplain flow regimes). Figure 2-2 provides an example of schematisation of a linked 1D-2D model.

Figure 2-2: Schematisation of a typical HPW model



Flood maps from the HPW models are derived from the 2D model and so the level of detail in these outputs is directly related to the accuracy of the underlying LIDAR data. The resolution and accuracy of the data in the HPW models provides significantly greater certainty in the model results compared to the MPW models. More details relating to the flood mapping methodology are provided in Section 8.

2.2 1D model development

2.2.1 Labelling system

Nodes within the model have been labelled using a 12 digit code. This is compiled from a 2 digit code detailing the UoM, a 4 letter code representing the watercourse name, a 5 digit figure representing the chainage along the watercourse from its downstream end, in units of 10m, and a single letter code representing the structure and face the section is representing. If the node is not associated with a structure the last letter code is omitted. For example 30CLAR02888 is in UoM 30 on the Clare River, is 28880m upstream from the downstream limit of the watercourse and is not representing a structure. The codes for the structure suffix are detailed in Table 2-1.

Table 2-1: Cross section survey label structure suffix codes

Identifier	Description
A	Upstream node at JUNCTION
B	Downstream node at JUNCTION
C	CONDUIT section
D	Upstream node at BRIDGE
E	Downstream node at BRIDGE
F	Upstream node at FLOODPLAIN section
G	Downstream node at FLOODPLAIN section
H	Upstream node at CULVERT BEND
K	Downstream node at CULVERT BEND
I	Upstream of CULVERT INLET
J	Downstream of CULVERT OUTLET
L	Lateral SPILL on left bank
O	Upstream node at ORIFICE
P	Downstream node at ORIFICE
R	Lateral SPILL on right bank
S	Upstream node at a SPILL
T	Downstream node at a SPILL
W	Upstream node at a WEIR
X	Downstream node at a WEIR

In general as part of the modelling process, identifier labels from the survey which are associated with the channel cross section at a structure have been moved to the structure unit itself within the ISIS model, and the open channel has lost the structure suffix code. For consistency, a junction unit has been included in the model between all river units and structures. The river unit adjacent to a structure in the model will therefore have a junction identifier rather than the structure identifier. For example, at the upstream face of a bridge the surveyed cross section was supplied with a D suffix; within the model, the D suffix is associated with the bridge unit, a junction inserted and an A suffix added to the cross section.

For interpolates, due to limited space in the software for labels, the UoM code has been removed, the correct chainage detailed, and an asterisk (*) appended. This allows for situations with more than one interpolate section in a 10m reach. An example of where this would occur is at culverts of less than 10m length: these require an upstream and downstream conduit unit, which without the proposed changes would have the same label. Conduit units that have the same chainage in the survey have had the UoM code removed, the correct chainage detailed and a C identifier added (BRUS002304C).

2.2.2 Hydraulic Roughness

In both HPW and MPW models, the hydraulic roughness within the 1D model has been appraised over three panels across the channel as follows:

- Left bank – from left bank top (or end of model left bank section) to a typical water level
- Channel bed – typically inundated part of cross section
- Right bank – from right bank top (or end of model right bank section) to a typical water level

For MPW models, the roughness applied for left and right bank panels typically includes the full width of the floodplain. This approach is considered suitable given the low resolution nature of the MPW models.

The determination of initial suitable hydraulic roughness values for each watercourse was based upon a combination of survey photographs, notes on survey drawings and observations from site visits. Reaches of similar roughness were identified and values reflective of these reaches extracted from published tables, summarised in Table 2-2 and Table 2-3. The majority of critical storms are expected to be winter storm and high roughness values based on summer vegetation in these instances are not considered to be appropriate. The assessment has therefore focused on the more permanent vegetation on banks, e.g. bushes and trees, when determining values.

Further adjustments to hydraulic roughness have been made where calibration data is sufficient to justify variations away from the quoted values.

Table 2-2: Typical roughness values for river channels⁶

Channel substrate	Typical value (Manning's n)
Bedrock	0.025
Cobbles (64-256mm)	0.055
Coarse Gravel	0.035
Gravel (2-64mm)	0.03
Sands	0.025
Silt	0.022
Clay	0.02
Concrete	0.02

Table 2-3: Typical roughness values for river banks⁷

Bank material	Typical value (Manning's n)
Scrub/Long Grass	0.04
Bushes	0.06
Trees – flood level not reaching branches	0.07
Trees – flood level reaching branches	0.15

2.2.3 Hydraulic structures

The representation of hydraulic structures in all instances should be case specific and reflect the hydraulic controls at the local site. The following sections outline general principles for the representation of hydraulic structures that have been recommended to deliver consistency across all WCFRAM models. However, these are not prescribed approaches and modelling judgement has been used where these principles do not appropriately capture the key controls at a given site.

2.2.3.1 Bridges

In general, bridges have been modelled using the Bridge (Arch) unit, with the USPBR unit reserved for larger sized bridges. Head loss at surcharged bridges has been calculated using the orifice equation and so will be consistent for either bridge type.

Bridge skew has been represented by entering the angle in the unit and not by adjusting local cross section chainages manually. This is consistent with the format the survey has been delivered in, which surveyed the full face of the bridge and recorded the skew angle.

Overtopping of bridge (and culvert) structures has been modelled in a number of ways. For HPW models, out of bank bypassing of structures has been represented in the 2D model. Overtopping of structures between banks has either been represented in the 1D domain, where the structure is relatively short and flows are expected to return to the channel on the downstream face, or in the 2D domain, where the structure is larger and flows spilling over the deck may not return directly to the channel. In the latter case either no spill over the structure has been included in the 1D model, or the spill has been included but deactivated to allow sensitivity testing to be carried out.

For MPW models, the full width of the bypass route (i.e. the width of the floodplain) is included in the 1D spill over the structure.

2.2.3.2 Culverts

Culverts have generally been modelled with culvert inlets and outlets to represent head losses at the upstream and downstream limits of the structure. This reflects the methodology outlined in the Culvert Design and Operation Guide⁸. Where these units have been found to be unstable or

⁶ Simplified version of Table 10 from Reducing Uncertainty in River Flood Conveyance. Roughness Review. By Karen Fisher and Hugh Dawson. DEFRA / Environment Agency Flood and Coastal Defence R&D Programme, Project W5A-057. July 2003.

⁷ Simplified version of Table 16 and 23 from Reducing Uncertainty in River Flood Conveyance. Roughness Review. By Karen Fisher and Hugh Dawson. DEFRA / Environment Agency Flood and Coastal Defence R&D Programme, Project W5A-057. July 2003.

⁸ Culvert Design and Operation Guide, CIRIA C689, 2010.

water levels are consistently exceeding the culvert soffit then the orifice unit and equation has been considered as an alternative.

Overtopping of culverts has been modelled using the approach detailed for bridges in Section 2.2.3.1.

2.2.3.3 Weirs

Weirs have been modelled using two parallel flow routes in HPW models. The first of these represents the weir structure itself within the stream bed and the second represents the banks either side of the weir but within bank top (Figure 2-3). The purpose of this is to allow the different hydraulic efficiencies of the weir and channel sides to be appropriately represented. Out of bank flows have been modelled in the 2D domain.

The weir itself has been modelled using the relevant equation for the structure, for example broad crested, sharp or jagged weir.

Figure 2-3: Typical section of a weir within bank



Typical coefficients for a jagged spill unit used are 1.7 for a well-constructed weir, dropping to 1.3 where the weir is in a poorer state of repair. The bank side coefficients may range from 0.8 to 1.3, depending on condition and vegetation growth. The out of bank portions of the weirs through AFAs are represented in the 2d domain.

2.2.4 Model boundaries

Hydrological inflows have been located at the upstream limits of the watercourses, at un-modelled tributaries along the watercourse length and over intervening reaches where the catchment is sufficiently large for lateral inflows to contribute to the peak flow observed in the watercourse. The methodology for the application of the hydrology is specific to each hydraulic model. However, a general approach was devised and reported in the Hydrology Report; this is discussed in further detail in Section 6.2. The detailed application of the hydrology to a particular model is provided in the Volume 2 AFA Modelling Report.

Where the downstream boundary of one model forms the upstream boundary of another model, the rating relationship from the downstream model has been applied to the upstream model.

This ensures the relationship between the two models is maintained, and provides confidence that the downstream boundary is responding correctly to flood flows.

In cases where a watercourse outfalls into a lake, the boundary was developed from gauge data where available. Tidal boundaries were developed for all required sites around the WCFRAM coastline based on the ICPSS, and are reported on in the UoM30-31 Hydrology Report.

2.2.5 Siltation

The presence and impact of silt in a channel is a function of the flow velocities experienced, both under normal and extreme conditions. High flow velocities could be expected to clear the temporary build-up of sediments, but lower flow velocities allow deposition and suggest sediments are a long term issue. This understanding is supported by the hydro-geomorphological assessment which has assessed the sediment regime within the modelled watercourses. In conjunction with this information and preliminary modelling results, a screening assessment of structures where sediment build up is apparent has been completed. The approach to the hydro-morphological assessment is provided in the UoM30-31 Volume 1b Method Statement, and specific details of the impact of sedimentation and gravel movements is provided in the relevant Volume 2 Hydraulic Modelling Report.

The representation of siltation within channels has generally been modelled as surveyed. The preferential output from the survey was for hard bed levels, which excludes siltation. Where the surveyors had observed a depth of silt on the open channel bed it was noted on the survey drawing as a second bed level. This information was used in the hydro-geomorphological assessment but did not impact on the model. The exception to the use of hard bed was where bed profiles were surveyed using remote techniques, such as an echosounder. In this case, the first return was used to represent the bed profile, which may have been the surface of the silt. This was the case in deep rivers where wading was not possible. For bridges and culverts, inclusion of siltation has been approached on a case by case basis. The modeller has made a judgement on the level of siltation observed in the survey, photographs and through site visit, and has included an appropriate level of accumulation. The decisions were influenced by the type of material and level of permanence, and the likely distance the deposits extended into the culvert.

2.2.6 Blockage

The assessment and modelling of the impact of blockage of bridges and culverts, whether through natural or manmade causes was not part of the CFRAM specification. However, where the potential for blockage was noted (evidence of accumulations of debris or fly tipping for example), and where there was the potential for such blockage to be a source of flood risk to neighbouring receptors this has been noted in the Volume 2 Hydraulic Modelling Reports.

A screening assessment has also been undertaken to identify locations where historical flooding has been attributed to culvert or bridge blockage and recommendations for modelling and development of remedial measures have been made. The method for undertaking this screening is detailed in Volume 1b: Hydromorphology Method Statement.

2.3 2D model development

2.3.1 1D-2D model boundary

The hydraulic boundary between the 1D and 2D models has been situated along the crest of the river banks. Crest levels, and hence the point at which water transfers from the 1D to the 2D domain have been determined in one of three ways. In order of accuracy (and therefore preference) these are:

- Directly extracted from bank top survey - this means any low spots between sections is represented in the spill between model domains
- Interpolated between surveyed cross sections - where detailed crest survey has not been collected it is likely the crest level is relatively consistent so interpolation is appropriate

- Extraction of bank heights from LIDAR data - where there are data gaps and the LIDAR data gives sufficient detail to be incorporated, or where the modeller has determined the crest top to be in a different position than that captured in the survey.

The AFA report indicates which of the above approaches has been taken for each watercourse within the model domain.

Along HPW reaches, the surveyed cross sections extended approximately 20m from top of bank. This was to allow comparison with LIDAR and to ensure the full requirements of the modeller were met, but was longer than was required to develop the 1D model. There are two methods for 'removing' the out of bank sections of the cross section:

- Deactivation markers can be assigned in ISIS, normally in the same place as the bank marker. This means ISIS ignores the portion of the section outside the marker. However, in some versions of ISIS (pre-3.7) the out of bank elements are still displayed on the cross section plots even though they are not accounted for in the hydraulic calculations.
- Rather than using deactivation markers, the cross section can be truncated at the end of the 1D domain. This means the visualisation of the cross section in ISIS relates directly to the length over which hydraulic calculations are carried out. This approach is required in spill and structure units, where deactivation markers cannot be used.

On open channel sections, neither approach is right or wrong so the modeller has used their judgement in each case. The main advantage of the first approach is that the cross section can easily be made wider by relocating the deactivation markers and the full extent of the cross section survey data is readily available for inspection.

A number of the watercourses being modelled as part of the WCFRAM are small and narrow. In these instances it has occasionally been necessary to situate the hydraulic boundary beyond the crest of the river bank. The reason for this is that low volume channels can cause model instability or significant fluctuations in water levels when proportionately large volumes, compared to the capacity of the channel, discharge into the 2D domain; this is particularly likely to occur where these small watercourses are tidally influenced or are situated within the floodplain of much larger watercourses. In these instances the capacity of the channel has been increased in the following ways;

- By widening the channel in the 1D domain but the level at which water spills into the 2D domain has remained fixed at the river bank crest level. The additional volume allowed for in the 1D channel will be small compared to the volume in the floodplain and so should have a minimal effect on the final model results.
- By moving the bank crest markers out from the channel top, and extracting the elevations from the topographic survey or LIDAR (as detailed above). This retains the volume in-channel, whilst increasing the stability of the model.

The approach taken was influenced by the geometry of the specific channel, and the detail of the elevation data that was available.

2.3.2 Cell size

The 2D model floodplain is represented as a ground level grid and has been constructed from the filtered LIDAR data. An appropriate grid resolution has been determined considering the size of the watercourse, floodplain complexity and model run times. The floodplains of narrow channels, or areas where complex overland flow paths may develop (such as around buildings and alleyways) are better represented with a small grid resolution, for example 2m, but model run times will increase accordingly.

The selected model cell sizes are specific to the waterbody being modelled and the resulting flood extent. Where possible, one model has been constructed per AFA but in some cases it has been necessary to construct separate models. The main reason for this is because of the requirement for different cell sizes for each watercourse. In AFAs where fluvial and coastal models have been built, cell size may also vary between domains.

Where a larger than 2m cell size has been used, this is reflective of a wide, uncomplicated floodplain and large model domain which would otherwise take a long time to run (in nearly all cases, the models developed can be run overnight). Where cell sizes greater than 2m have

been used, and there is the potential for complex flow paths to develop, the implications have been considered as part of the sensitivity testing (see Section 7.2).

2.3.3 Floodplain roughness

The complexity of the floodplain itself has been represented using a varying hydraulic roughness to represent the different surfaces apparent within the floodplain, Table 2-4. The different surface types have been derived from OSi NTF data. The data has been incorporated into the 2D model in the order listed so that coarse, wide ranging surfaces, such as woodland, do not overwrite more complex surfaces, such as roads.

Table 2-4: 2D model floodplain roughness values

2D model order	Land use type	Manning's n roughness value
1	Inland Water	0.035
2	General Natural Surfaces	0.04
3	Coniferous Trees	0.1
4	Mixed Vegetation	0.08
5	Non-coniferous Woodland	0.07
6	Rock	0.05
7	Roads and Tracks and Paths	0.015
8	General Manmade Surfaces	0.017
9	Glass Houses	0.2
10	Buildings	0.3
99	Stability patch	0.5

There are a number of different ways to represent buildings within 2D models, ranging from removing them from the floodplain entirely to allowing flow to pass through the building with reduced hydraulic efficiency, represented through Manning's n, and as described above. The baseline approach has been to represent the buildings using Manning's n, but the implications of the approach to building representation have been reviewed as part of the sensitivity analysis in Section 7.2.

2.3.4 Stability patches

Instabilities in TUFLOW are generally a result of irregularities in the underlying LIDAR ground model resulting in the transfer of large volumes of water between cells with each time step. This can result in rapidly fluctuating water levels and hence instability in the model results. There are a number of methods for overcoming TUFLOW model instabilities. One of these is to use stability patches, which are polygons that have been assigned a high (0.5) Manning's n value. The patches slow the flow of water across the local surface, thus reducing the volume of water transferred between cells with each time step. Use of these patches should be used with caution as they can potentially constrain flow and result in an under prediction of flood risk, however they may be appropriate where there are low lying dips in the topography or where elevations in the LIDAR DTM vary greatly between adjacent cells (this may be a reflection of the natural topography or processing relics). The specific use of the stability patches is explained and justified in the relevant Volume 3 Hydraulic Model Report.

2.3.5 Overtopping of structures in the main channel

Section 2.2.3.1 details those instances where overtopping of hydraulic structures will have been represented in the 2D domain. In these instances, the crest levels of the structures have been incorporated into the 2D model by modifying the floodplain grid. This is a two stage process whereby the surveyed deck level is first applied across the width of the channel over the length of the structure and then the surveyed parapet levels applied along the upstream and downstream face of the structure. The modelling of parapets at specific structures is detailed in the individual AFA hydraulic modelling reports, but in general the spill is positioned at the top of a solid parapet, but if there are railings / open parapet face the spill is at deck level. Including the top of the parapet level in the model does not indicate the structure functions as a flood defence, as they are often bypassed. Instead, this method of representing the structure ensures that the flow routes around the bridge are included. This is particularly important if the overtopping water flows away from the channel, rather than re-entering the watercourse immediately downstream of the structure. Although all bridges have been assessed structurally in the Defence Asset

Database, where the parapet has been included in the model, it has been assumed the structure is capable of withstanding the water build up.

As part of the initial model construction, all bridges and culverts were included in the model with an overtopping spill unit. This allowed water to overtop the structure and return to the 1D channel downstream where deck levels were low enough. When the 1D model was linked to the 2D domain, this bypass route could either be retained in the 1D element of the model, or represented in the 2D domain. In the case of short structures where flows would simply flow over the deck and back into the channel, the structure remained in the 1D domain, and an appropriate spill coefficient applied (Table 2-5). These coefficients are indicative only and the actual selected coefficient will represent the conditions at the site and be site specific.

If the flow paths over the structure are more complex the bridge deck has been included in the 2D domain. In this case, the ISIS spill unit is not required. In some cases the spill will have been deleted from the model, and in other cases will have been deactivated by setting the coefficient to 0.

Table 2-5: Overtopping spill coefficients

Structure description	Overtopping spill coefficient
Spill deactivated	0
Structure deck is a road/foot path less than 20m in width	Within the range 1.5 to 1.7
Structure deck is a road/foot path greater than 20m in width	Within the range 1.2 to 1.5

2.3.6 Representation of structures in the floodplain

Raised structures such as motorways and railway embankments often bisect floodplains and so dictate the extent of flooding. The exclusion of drains or bypass routes beneath these structures can result in the underestimation of flood risk on the side furthest from a watercourse and can potentially overestimate flood risk on the side nearest to the watercourse. Two alternative approaches to including such structures have been used in this modelling study.

- If the bypass routes exert no real vertical constraint on flow, for example in the case of a large underpass which will provide a flow route but will not become surcharged, then modifying the DTM with a cut line through the embankment is sufficient to simulate the flow path.
- For smaller culverts where capacity may limit flow, or where the culvert is smaller than the cell size, the structure has been incorporated into the 2D domain as a 1D ESTRY element. In such cases culvert is specified in a similar manner to the culverts described in Section 2.2.3.2.

The choice of approach in specific situations is detailed in the relevant AFA reports.

2.3.7 2D model boundaries

The 2D model domain has been extended to incorporate the full width of the active floodplain (as determined by draft model runs for extreme flood extents, and through examination of the topography). This means water generally only flows into the 2D domain from the 1D domain, across the boundary discussed in Section 2.3.1.

However, there are a number of instances where modelled flows reach the edge of the 2D domain and a standalone 2D model boundary is required; the extent of the floodplain in these areas will be dictated by this boundary, for example, where the floodplain discharges directly to the sea. In these cases, the tidal curve was applied along the length of the coastline where the floodplain is active and water discharged into the sea only when tidal levels fell below the water level in the floodplain.

3 Coastal hydraulic modelling

3.1 Modelling Approach

Coastal models have been constructed as 2D only models using TUFLOW. The model consists of the floodplain beyond the coastline and has been developed from the LIDAR data, which forms a grid of floodplain levels. Where coastal defences and walls were identified, crest level survey provided greater detail at the 2D boundary and was incorporated into the model.

3.2 Coastal model development

3.2.1 2D model boundaries

EWLs have been taken from the ICPSS Phase IV - West Coast, Predicted Extreme Water Levels Associated with Combined Tide and Surge and were used to develop design tide curves; the details of which are provided in the UoM 30-31 Hydrology Report.

The water levels were determined at intervals around the coastline, generally some distance offshore, however, in accordance with the project specification these were deemed suitable for application at the foreshore. These water levels were generally applied along the coastline as shown in the OSi 1:5,000 mapping. Exceptions to this occurred where there was a complex headland which was not fully represented in the LIDAR. In these cases, the boundary was moved out to sea by a short distance, smoothing the transition of flows across the 2D domain.

There are a number of uncertainties inherent in the translation of the offshore tide curves to the near shore, particularly where this naturally involves the propagation of the tide along an estuary, around islands or through headlands; such limitations are discussed in the relevant Hydraulic Modelling Reports, but it was outside the scope of the CFRAM to attempt to address these issues.

3.2.2 Cell size

As with the fluvial 2D model domains, the coastal floodplain is represented as a ground level grid and has been constructed from the filtered LIDAR data. An appropriate grid resolution has been determined considering the floodplain complexity and model run times; complex overland flow paths (such as around buildings and alleyways) are better represented with a small grid resolution, for example 2m, but model run times will increase accordingly.

Sensitivity testing has been carried out where cell sizes greater than 2m have been used, and there is the potential for complex flow paths to develop, (see Section 7.2).

3.2.3 Floodplain roughness

The same approach to defining Manning's n values has been applied as in the fluvial models. See Section 2.2.2 for more details.

3.2.4 Representation of structures

Flood defences and walls which occur on the coastal boundary (such as quay walls) have been included in the models as a single raised line of cells along the alignment of the defence. Where crest level survey was not available an allowance for the structure has been estimated from site photos. This is of particular importance for the wave overtopping analysis where the presence of a raised structure may prevent flows from returning to the sea, see Section 3.3. Structures in the floodplain - As with the fluvial models, floodplain structures, such as culverts below embankments, can play a significant role in causing or preventing inundation of land. Such features have been represented as ESTRUCT units in the models.

3.3 Wave overtopping

In certain locations, the coastal flooding models were developed further such that they were able to simulate flooding from wave overtopping of defences as well as from tide and surge events.

The locations were driven by the availability of wave data, which in turn was a function of the assessment undertaken through the ICWWS. Of relevance to this UoM is Galway City, which has a history of flooding caused by overtopping in the city⁹. It should be noted that other locations may be vulnerable to wave overtopping, but they were screened out of the ICWWS and have therefore not been included in the CFRAM.

Wave overtopping is a complex process controlled by the state of the sea (depth, wave properties) and the geometry of local flood defences. Wave overtopping has been calculated using EurOtop¹⁰ methods. The EurOtop manual includes different methods and associated guidelines for the prediction of wave overtopping for different structure types. For this study the Neural Network methodology was utilised, requiring the following input parameters for the calculations: still water level at the toe of the structure (coastal defence), the incident wave conditions at the toe of the defences to be overtopped, and the defence profile shape. There are some uncertainties inherent in this process such as the manual schematisation of flood defences, the initial wave heights, the storm duration and the output results being estimates of the mean overtopping discharge rather than the exact values.

The methodology for assessing wave overtopping risks followed the steps laid out below:

- Receipt of still water and wave data from the ICPSS and ICWWS
- Compilation of environmental and topographic data available
- Generate schematisations of the coastal defences
- Estimate overtopping potential for the 2, 5, 10, 20, 50, 100, 200 and 1000-year wave return period events
- Estimate overtopping potential for the present day, MRFS and HEFS
- Provide the mean overtopping discharge for a matrix of scenarios
- Apply the overtopping volumes to the coastal hydraulic model.

3.3.1 Schematisations

Each site was individually investigated for the availability of appropriate environmental and topographic data. Wave, tide, LIDAR and flood defence survey data was collated for the AFA. The suitability of these data was assessed and where necessary additional site survey was procured through the infill contracts.

Due to the history of flooding due to wave overtopping, in order to accurately perform overtopping analysis on the structures in Galway a topographic survey was conducted to determine the changes in structure types. The profiles from this survey provided greater accuracy compared to the available LIDAR data and allowed the changes in roughness and berm level to be accurately represented.

3.3.2 Wave data

Wave data was provided by OPW from the Irish Coastal Wave and Water Level Modelling Study (ICWWS). The wave data was calculated at specific depths using both wind and swell waves. For the analysis these wave heights are used in a depth limited calculation relative to the individual profiles of the coastal defences. For each return period the wave data was provided with six joint probability combinations of water level and wave height for both the wind wave and swell wave components. To find the worst case wave overtopping at each location the wave overtopping calculations were performed for all joint probability scenarios for each return period.

Overtopping calculations were performed for the following scenarios;

- 2 x Sea states: wind and swell waves;
- 6 x Joint probability combinations of water level and wave height;
- 3 x Climate change scenarios (as defined by the ICWWS): Current, Mid-range Future Scenario (MRFS) and High End Future Scenario (HEFS);
- 8 x Return periods: 2, 5, 10, 20, 50, 100, 200 and 1000-year wave events, and

⁹ Galway City Council (2012) "SEA Environmental Report of Galway City Development Plan," 2011- 2017

¹⁰ EurOtop (2010) "Wave Overtopping of Sea Defence and Related Structures: Assessment Manual", Overtopping Course Edition, November 2010. HR Wallingford.

- A design storm surge shape.

3.3.3 Worse case overtopping scenario

Overtopping calculations were performed for all joint probability combinations of water level and wave height. This combined the scenarios described in Section 3.3.2 (i.e. $8 \times 3 \times 6 \times 2 = 288$ scenarios), and the worst case i.e. the highest overtopping volume was selected for each return period at each of the schematised overtopping profiles, was extracted. For input into the flood inundation models the wave overtopping was therefore a composite of the worst case overtopping at each individual defence which may be made up of a number of different combinations of water level and wave height within each return period. Overtopping volumes of less than 0.03l/s/m were considered negligible and so were not included in the flood inundation models.

For each profile through the defence overtopping volumes were generated for the present day, MRFS and HEFS. The wave overtopping conditions were provided every 15 minutes for the duration of overtopping over the peak of one tidal cycle, for each of the different return periods.

3.3.4 Modelling overtopping

Flood risk in the wave overtopping models is driven by both the still water level and the wave overtopping volume associated with the worst case joint probability scenario. These scenarios are represented in the model using two boundary lines, the existing boundary line from the coastal models, which is located a short distance off shore, is used for the still water level, and an additional series of boundary lines, located on the landward side of the coastal defences, are used to add the wave overtopping volumes for each defence profile assessed.

For most return periods in Galway City and Roundstone, the still water level from the joint probability scenario is lower than the typical crest level of the coastal defences. Where this is the case, no tidal boundary has been applied in the model as no overtopping due to the still water levels would occur, and the modelled flood risk extents are attributable to wave overtopping only. In the limited number of scenarios where the still water levels do exceed the defence crest it has generally been possible to apply a single tidal profile as the still water level and wave height combination resulting in the worst case scenario has been the same.

It is important to note that the still water level in these model runs is not equivalent to the extreme sea water levels calculated as part of the ICPSS. Therefore the 0.5% AEP wave overtopping extent will not inherently incorporate the 0.5% AEP tidal flood risk extent.

4 Defence modelling

4.1 Classification and modelling of raised structures adjacent to watercourses

Raised structures adjacent to watercourses and coastlines will play a significant part in determining if the land behind these structures is shown as at flood risk in the final flood maps. Removing these structures when, in reality, they prevent flooding would overestimate flood risk and reduce public confidence in the quality of the flood maps produced. Conversely, including structures when they are not constructed to a sufficient standard to withstand elevated water levels would result in a false sense of security amongst residents, and result in them being underprepared and at greater risk should the structure fail.

All raised structures identified adjacent to watercourses and coastlines, whether OPW defences or other structures identified on site, have been reviewed in some detail and classified as effective or ineffective depending on their ability to provide a flood defence function. The classification is based on visual inspections carried out as part of work on the defence asset database. This classification then dictates how the structure has been represented in the hydraulic model.

Some defences are classed as 'formal' flood defences; these are engineered schemes which have been constructed specifically to provide a flood defence function, but may be effective or ineffective depending on the specific maintenance regime. They are usually the responsibility of the OPW, but may fall under Local Authority or private control. Informal defences are those structures which are not designed specifically for flood defence purposes, but serve to provide such benefits. These may include railway and road embankments and other walls and embankments which would be effective in flood conditions.

The classifications are recorded in the Defence Asset Database, which includes a condition assessment of each structure.

4.1.1 Effective

'Effective' structures are continuous and tie into high ground or other defences. Failure of these structures occurs via overtopping or in the event of a breach. Within the hydraulic model these structures have been represented as surveyed, i.e. the crest level of the defence has been included in the model.

These structures have been considered for the breach scenarios and have been removed for the purposes of the defended area and flood zone mapping.

4.1.2 Ineffective

Ineffective structures can be assumed to fail in a number of different ways, and the way they are likely to fail has dictated the way in which they have been represented in the hydraulic models. To help explain the different modes of failure a further three sub-classifications have been developed.

Table 4-1: Ineffective structure classifications

Ineffective structure	Description	Treatment for modelling purposes
Type 1a - structures with gaps	In their existing condition there is a route for the structure to be bypassed, either through gaps or low points in the structure or because the structure does not tie in to high ground at one or both ends. Such features are included in the topographic survey. The structure may be adequate in its design and materials to resist flood water causing it to overturn or breach.	Modelled as surveyed
Type 1b - walls less than 0.6m	These will not normally have been designed as flood defences but, due to the limited depth of water able to pond against them before overtopping occurs, they are expected to remain intact during a flood event. The effect of these structures would be expected to quickly become negligible in larger flood events as they become drowned out or bypassed.	Modelled as surveyed

Ineffective structure	Description	Treatment for modelling purposes
	They are important in low order events as they push flood waters in particular directions.	
Type 2	Form coherent barriers adjacent to the watercourse and most probably tie into high ground at either end. However the structures themselves were not designed as flood defences and would be expected to fail in the event of a flood. The depth at which the hydraulic pressure on these structures will result in failure has been modelled at 0.6m. This is the height that results in the load that can be withstood by a single skin brick wall where the failure mode is through tensile cracking of mortar bond in brick. Structures where this depth is not exceeded in the 1% AEP event have been classified as Type 1a.	Excluded from the model
Type 3	Structures which could in the future form part of a flood defence but are either currently bypassed, as described in the Type 1 classification, or the base is above the current 1% AEP flood level. They are considered to be structurally sound and capable of retaining flood waters, should the gaps (often walkways) be filled.	Modelled as surveyed

4.1.3 Defences in UoM30-31

Following the classification of defences detailed above, formal defences were identified in Galway, and Gort AFAs. Formal defences, in the form of embankments, were identified by OPW in Corrib Clare, Corrib Mask and Cregadare. No informal effective defences were identified. All the defences protect against fluvial flooding, and in Galway City also protect against coastal flooding.

4.2 Undefended modelling

4.2.1 Defended areas

A suite of undefended flood outlines have been produced. These defended area polygons show areas benefiting from the effective defences identified within AFAs. For each defence the model results have been reviewed and the most severe (lowest AEP) of the 8 predefined AEP events that does NOT overtop the defence / defence group along its length has been identified and termed the effective standard of protection (SOP). For example, if the 10% AEP is fully contained within the defence, but the 5% AEP overtops the defence at any point, then the effective standard of protection of the defence can be considered to be the 10% AEP event.

To generate the defended area polygons, the defences were removed from the model and an undefended run at the effective SOP was completed. The difference between the defended and undefended outline for this SOP is the defended area.

In Galway, where there were multiple defence lines within an AFA, all defences were first removed from the model and the effective SOP event corresponding to each defence line was run in turn, thereby producing a defended area polygon specific to each defence line.

4.2.2 Flood Zones

In addition to modelling the effective SOP of the defences, Flood Zone maps have been produced, meeting the requirements of the Planning System and Flood Risk Management¹¹. Flood Zone A is the 1% AEP fluvial or 0.5% AEP tidal (1 in 100 year fluvial / 1 in 200 year tidal) extent and Flood Zone B is the 0.1%AEP (1 in 1000 year) event; both scenarios are undefended. In order to generate these maps, the 1% and 0.1% AEP flows were run through the undefended models, regardless of the effective SOP of the defences. It should be noted that where there are no defences, the flood extents and flood zone maps are the same.

¹¹ OPW (2009), The Planning System and Flood Risk Management: Guidelines for Planning Authorities 2011s5232 WCFRAM UoM 30-31 Volume 1a Hydraulic Modelling Method Statement v3.0.docx

4.3 Breach analysis

4.3.1 Screening

In the event of a failure of flood defence assets (such as the collapse of a section of wall or breach of an embankment), areas that would otherwise be defended against flooding during a given event severity might become subject to flooding.

The modelling of the defence failure scenarios was required where the defence provided protection to receptors within an AFA and the depth of water retained by the defence exceeds 1.0m during a flood event of a probability equal to the standard of protection provided by the defence.. Where these conditions are met, two failure scenarios were assessed.

A screening was carried out to determine which defences were greater than 1m from crest to toe, and therefore capable of retaining more than 1m of water. If the defences were in excess of this height, the depth of water retained behind the defences in the existing risk scenarios was reviewed. The results of this screening is provided in Table 4-2.

In addition, the requirement for breach analysis where there was a risk from coastal erosion under a current or future scenario was assessed, but was not identified. There are no coastal defences in Galway City, where the only coastal model was developed, and there is in general little risk associated with coastal erosion.

Table 4-2: Breach analysis screening results

AFA	Defence ID	Defence height greater than 1m	Retained depth of water greater than 1m
Galway	A30GLW_146	Yes	No
Galway	A30GLW_150	Yes	No

4.3.2 Breach modelling

The screening assessment did not identify any defences, either formal or informal effective which required breach modelling within UoM 30-31.

5 Model calibration and sensibility checking

5.1 Objectives and categorisation

The objective of the calibration process is to provide confidence in the outputs from the hydraulic model (either fluvial or coastal) by demonstrating that the models produce a suitable representation of past events, and are therefore likely to predict the output of design events well. This process is heavily dependent on the availability of data from past events, both from gauge records and evidence of historical events. Three levels of checking have been identified for use in this study:

- Calibration - where gauge data and evidence of one or more events is available
- Partial calibration - where there is gauge data but limited / no evidence of flooding, or no gauge data but evidence of flooding
- Sensibility check - where there is no gauge and no evidence of flooding.

The availability of gauge data is discussed in the UoM Hydrology Report, and in the specific AFA report, where the historical events are also summarised. Data relating to historical events are likely to be evidence or anecdotal records from a given flood collected by local authorities or residents. A large data collection exercise was completed at the start of the WCFRAM project and historical flood evidence collected is presented in the Flood Risk Review (FRR) Reports and summarised in the AFA modelling reports. Where additional flood records have come to light since the FRR stage, they are discussed in the relevant AFA model report.

In the absence of flow data, it is also theoretically possible to calibrate the model using recorded rainfall data and a rainfall run-off model. However, there are very few sub-daily rainfall gauges across the catchment which has prevented the development of hydrological models to represent the response to catchment wide rainfall events. This issue was identified within the Inception Reports and recommendations were made for the installation of additional rainfall gauges to support the calibration process. As a result, rainfall runoff models have not been used for model calibration exercises.

5.1.1 Calibration

Where flood records, including recorded flows or levels and records of the impacts of flooding at a number of locations (either flood extents, or spot levels) are available, it is possible to calibrate the model. This process would involve running the recorded flows through the model and changing model parameters, such as Manning's n , to match the flood extents or levels that were observed. Where possible a second (or more) event is then run through the amended model and the outputs compared with flood records to confirm the validation work.

5.1.2 Partial calibration

If there is only limited information available, either gauge data but no accompanying historical flood records or vice versa, an informed sensibility check, or partial calibration, has been carried out. This involved checking that the model is producing an expected outcome (such as matching a wrack mark at a suitable return period or producing a reasonable flood extent for the largest recorded event), but without a high degree of confidence in the overall outputs. This level of checking is unlikely to have resulted in changes to the model parameters being made but will flag up where there are obvious inconsistencies between the model and reality. For example if a site has flooded twice in the last 10 years then the site would be expected to be shown as inundated in the 10% AEP design flood extent and dry in the 50% AEP flood extent. Conversely if a site has only flooded once in the last 50 years it would not be expected to be shown as inundated in the 10% AEP design flood extent.

5.1.3 Sensibility check

If there is no gauge data, and / or no record of flooding, model checking is limited to a sensibility check on model outputs based on topography and local knowledge. This is the approach most commonly taken on tributaries which are all ungauged.

6 Application of hydrology

6.1 Hydrological estimation points

Design flows have been provided at a series of HEPs along the length of all watercourses to be modelled. These HEPs are located at the upstream limits of models, upstream and downstream of tributaries, at gauging stations and also at sufficient frequency between these locations to pick up the progressive increases in the catchment drainage area moving downstream.

The hydrology for all catchments has been calculated and reported within the Hydrology Report for UoM 30 and 31. No further details are provided on the development of the design flows and the reader is referred to the Hydrology Report for further information. The design flows for each model have been reported in the relevant AFA modelling report.

6.2 Application of design flow estimates

The approach to applying the design hydrology to the hydraulic models has been set out in the Hydrology Report. It has been reiterated and summarised here as it forms an integral part of the development of the hydraulic models and the approach adopted for each model has been reported on in the specific AFA modelling reports.

As part of the work completed for the Hydrology Report a detailed review of the best way to develop design hydrographs was completed. The findings were that hydrographs developed from catchment descriptors using the Flood Studies Report (FSR) rainfall runoff method for hydrograph generation provided the best fit against a suite of observed data.

The objective of the application of hydrology process is to match the design flows at each of the HEPs within the model. For the purposes of the study a modelled flow within 5% of the design flow is considered to have adequately achieved this aim. It is noted that due to the changing data sets and methodologies when calculating design flows across a large catchment, HEPs along the length of a watercourse are not always consistent, for example where a donor site changes from an upstream gauging station to a downstream gauging station. In these instances a generally conservative approach has been adopted and the HEPs with the higher flows used as the basis for the design events.

6.2.1 Hydrograph shapes

Inflow hydrograph shapes for each watercourse have been developed from the Flood Studies Report (FSR) rainfall runoff method. This approach has been tested (with results detailed in the Hydrology Report) and with the exception of a few gauges, which are detailed in Section 6.2.3, this found the FSR approach to provide the best fit against gauge data. Model inflow hydrographs are located at the upstream limit of each watercourse.

6.2.2 Standard methodology

The specific approach taken in developing the hydrology to match HEPs within the model is detailed in each of the AFA modelling reports, but included the following steps:

1. Production of FSR hydrographs from catchment descriptors for all HEPs within the AFA. FSR hydrographs at the upstream limit of the modelled watercourses or at tributaries have been used directly. FSR hydrographs at intermediate sites along the watercourse have been used to develop lateral inflows.
2. Identification of a suitable critical storm duration (based on catchment descriptors) for the main watercourse. The critical storm duration is assumed to be representative of the whole reach of the AFA but the focus is on the main area of interest, i.e. between the critical duration of the upstream and downstream limits of the reach of interest. The storm duration for all FSR boundary units has been set equal to this to represent a consistent event across the AFA.

Applying the FSR method with a uniform design storm for all sub-catchments within a model imposes a structure on the model inflows with realistic relative timings of the hydrographs. This avoids the need to apply the FSU regression model for relative timings of hydrographs at a confluence, which is associated with a large standard error.

Because the FSR method is being used only to control the shape of the hydrographs rather than the magnitude of the peak flows (which are based on the HEPs), there is no benefit to identifying a critical storm duration, i.e. one that results in the highest peak flow or water level. However, in order to ensure a realistic flood duration, the duration of the design storm has been related to the time to peak for the principal watercourse in the model, using the FSR formula that evaluates storm duration from time to peak and SAAR. The potential impact of the critical storm duration on tributary flood extents has been reviewed and is discussed in Section 7.2.8.

3. Adjustment of time to peak (Tp(0)) values if required. Tp(0) values have been adjusted where a lag analysis has been completed as part of the hydrological analysis and included in the Hydrology Report. Sites where this has been done and the associated Tp(0) adjustment are Ballyhaunis (2.0) and Corrofin (2.49). The Tp coefficient in the FSR units have been set to the relevant values for the gauged watercourse only, unless a tributary is of a sufficient scale to be considered comparable to the gauged watercourse, in which case the same scaling factor has been applied.
4. Scaling of the peak for all FSR units to reflect the design event peaks provided in the HEP file and application of upstream inflows. Where the difference in flows between the upstream and downstream limits of a watercourse, or between confluences, is within 10% then an intermediate value has been selected as representative of the whole reach and applied at the upstream of the model. Where this is not the case, the upstream inflow, as indicated in the HEP, has been used and additional inflows added using lateral inflow units, as detailed in step 5.
5. Application of lateral inflows where required. Lateral inflows have been developed from the FSR units at the upstream and downstream limit of the reach of interest, which will likely include the intermediate HEPs. The lateral inflow hydrograph shape is the upstream hydrograph subtracted from the downstream hydrograph, typically for the 1% AEP event. These have then been scaled to match the flows at the HEPs as required, and input as a lateral inflow over the reach of interest. Where the upstream and downstream hydrographs do not overlap, and so prevent the development of a lateral hydrograph, a suitable hydrograph for the watercourse has been selected from the available HEPs.
6. Confirmation that flows at confluences are consistent. Flows upstream and downstream of tributaries have been reviewed to confirm if HEPs are providing a reasonable estimation.

Whilst the above approach is also appropriate for most MPW models, the Clare River has been split as the catchment changes along its length and the critical storm duration changes significantly between upstream and downstream limits.

6.2.3 Hydrograph width analysis methodology

Rather than applying the FSR method detailed above, inflow hydrographs for Galway City have been developed from observed events; this is due to the availability of suitable gauges in proximity to the AFA. The application of the hydrology in these instances has included the following steps:

1. Setting up the HWA hydrograph for the AFA. The hydrographs from the HWA have been scaled to reflect the design event peaks detailed in the HEPs.
2. Identification of all FSR boundaries required along tributaries for the hydraulic model. Hydrographs on the main watercourse have been based on the HWA hydrograph.
3. Identification of the critical storm duration for tributaries and set the storm duration for all FSR boundary units on tributaries equal to this. Each tributary has been treated separately and the identified critical duration set for all intermediate points on the tributary.
4. Scaling of the peak for all inflow units to reflect the design event peaks at the upstream HEPs.
5. Determination of the timing of the events on the tributaries. The difference between the timing of the peak flow on the main watercourse and the tributary has been determined using the following regression equation from the FSU report Work Package 3.4:

$$\text{time.diff} = 32.1 \times \text{BFI.diff} - 103 \times \text{FARL.diff} + 1.62 \times \text{SQRT.AREA.diff} - 1.94 \times \text{TAYSLO.diff} - 46.4 \times \text{ARTDRAIN.diff} - 0.0272 \times \text{NETLEN.diff}$$

where:

- time.diff is the time difference (hours) between the inflow and the modelled reach. A positive value of time difference means that the inflow peaks before the modelled reach (which it normally will).
- BFI.diff is the BFI of the modelled reach (upstream of the confluence) minus that of the tributary.
- FARL.diff is the FARL of the modelled reach (upstream of the confluence) minus that of the tributary.
- SQRT.AREA.diff is the square root of the AREA (km²) of the modelled reach (upstream of the confluence) minus that of the tributary.
- TAYSLO.diff is the Taylor-Schwartz slope of the modelled reach (upstream of the confluence) minus that of the tributary.
- ARTDRAIN.diff is the arterial drainage index of the modelled reach (upstream of the confluence) minus that of the tributary.
- NETLEN.diff is the network length (km) of the modelled reach (upstream of the confluence) minus that of the tributary.

Timings should be relative to the time of the event peak on the main watercourse at the confluence with the tributary.

1. Application of inflows at the upstream limit of each watercourse
2. Application of lateral inflows where required to match HEPs. Lateral inflows have been developed from the inflow hydrographs, i.e. scaled HWA hydrographs if required. The method used to match design flows at HEPs is as described in Section 6.2.2, except that the HWA hydrographs have been used instead of FSR hydrographs.
3. Confirmation that flows at confluences are consistent

7 Sensitivity testing

7.1 Screening

To support the understanding of the uncertainties associated with the hydraulic modelling process, a suite of sensitivity tests has been carried out. These tests investigate in further detail the implications of the assumptions in the development of the hydraulic model and the production of the design flood extents.

The nature of the sensitivity analysis and the model parameters assessed means that any analysis has been based on engineering judgement only, however by maximising the hydraulic modellers' knowledge of the site, sensitivity assessments are representative of the limitations of the data availability for the site.

Rather than adopting a generic approach to the sensitivity analysis, a screening judgement has been made as to those tests that are applicable and required for each AFA.

The following sections discuss the range of the sensitivity tests required and provide examples of how parameters have been adjusted to reflect known uncertainties. The sensitivity tests and the situations in which they apply are laid out in Table 7-1.

Table 7-1: Sensitivity tests

Sensitivity test	HPW/ MPW applicable	Other watercourse characteristics
Peak flow	HPW and MPW	In all watercourses
Roughness	HPW and MPW	In all watercourses
Water level boundaries	HPW and MPW	Watercourses which discharge into the sea or a lake
Building representation	HPW	Where buildings are within the flood extents
Flow volume	HPW and MPW	Where the hydrograph is generated from catchment descriptors
Afflux / headloss at key structures	HPW and MPW	Where headloss has been noted in the long section, and the structure may cause flood risk
Timing of tributaries	HPW	Where tributary is in the same model as the main river
Timing of fluvial and tidal peaks	HPW and MPW	Where the river has a tidal boundary
Critical storm duration	HPW	Where tributary is in the same model as the main river
Cell size	HPW	Where cell size is greater than 2m and there are complex flow routes across the floodplain.

Where site specific assumptions are identified within the AFA modelling report, further sensitivity tests may be completed to examine these, and may include testing specific control structures.

Sensitivity tests to flow, roughness and water level boundaries have been carried out on all models for the 1% AEP event. Sensitivity tests to building representation, flow volume, afflux at key structures and timing of tributaries for the 1% AEP event, and for all tests related to the 10% AEP event have only been carried out where a screening exercise has identified that there remains significant uncertainty and models may be underestimating flood risk. Details and results of this screening exercise have been provided in the individual AFA modelling reports. Sensitivity testing of the 0.1% AEP events has not occurred.

In all cases it is important to consider the sensitivity tests as a sensible shift within the bounds of reasonableness. Therefore, if through the calibration process, parameters have been increased towards the upper limits of reasonableness for a given parameter then the additional shift for a review of sensitivity will be less than if no calibration/validation process has been carried out and default parameters have been applied.

The mapping of uncertainty bounds is described in Section 0.

7.2 Sensitivity analyses

7.2.1 Flow

Table 7-2 provides a scoring mechanism through which each watercourse has been attributed a score from each row of the table reflecting the level of confidence in the hydrology. The resulting scores have been summed to provide an overall indication of uncertainty and used to look up in Table 7-3 the uncertainty weighting to apply for the sensitivity test.

The uncertainty in QMED was assessed using the equations for SE and FSE provided in the FSU WP2.2 report. These were applied to estimates derived from catchment descriptors, which will give a scaling factor of 1.37, or at gauge sites which will typically give a lower scaling factor. This reflects the uncertainty in the index flood but does not reflect the uncertainty in the growth curve, for this reason an additional multiplication factor is included for the 1% AEP event.

Table 7-2: Flow sensitivity test scoring mechanism

Scoring parameter	Score of 1	Score of 3	Score of 5	Score of 7
Is there a local recording gauge that has been used as a donor for the hydrology?	Within 5km of the AFA and on the same watercourse with no significant other inflows between the gauge and the AFA OR Upstream and downstream of the AFA with no significant other inflows between and routing of flows supports the hydrology	Within 5km of the AFA but not on the same watercourse or with significant other inflows between the gauge and the AFA	Beyond 5km or with significant other inflows between the gauge and the AFA	No useable gauge
What is the length of record of the local gauge?	Greater than 40 years	Between 20 and 40 years	Between 2 and 20 years.	No useable gauge
What quality is the record from the gauge?	Rating review carried out, high confidence	Rating review carried out, moderate confidence or no rating review carried out but gauge is FSU class A	All other sites.	N/A.
What unusual features are there in the catchment hydrology?	None – a rural catchment typical of many in the gauged datasets	Some lakes ($0.99 > \text{FARL} > 0.9$) or urbanisation ($0.05 < \text{URBEXT} < 0.15$)	Some karst or extensive lakes ($\text{FARL} < 0.9$) or urbanisation ($\text{URBEXT} > 0.15$) or arterial drainage	N/A
What is the size of the catchment?	N/A	N/A	<25km	N/A

Table 7-3: Flow sensitivity scaling factors

Return period of event	Score up to 6	Score of between 7 and 14	Score of between 15 and 22	Score above 23
10%	No sensitivity test required.	Use QMED uncertainty	Use QMED uncertainty	Use QMED uncertainty
1%*	Use QMED uncertainty then apply adjustment factor of 1.1	Use QMED uncertainty then apply adjustment factor of 1.2	Use QMED uncertainty then apply adjustment factor of 1.3	Use QMED uncertainty then apply adjustment factor of 1.5.
* Where extensive areas of karst with connections to the surface water system is present then use QMED uncertainty then multiply flows by 2.0 to reflect the uncertainty in the 1% event flow.				

7.2.2 Roughness

Based on the assessment of typical vegetation cover completed as part of the hydraulic modelling, and an understanding of the maintenance regime carried out by the local authorities and OPW, high and low end roughness values have been determined for each channel.

If one or more large events have been observed and sufficient data is available with which to calibrate the roughness within the channel then the uncertainty in channel roughness is assumed to be reduced and a variation of Manning's n to the full extent suggested has not been applied. It is also noted that in large events with greater depths the influence of channel roughness is often reduced; in these instances a variation to the maximum upper bound may not have been applied.

Floodplain Manning's n values have also been adjusted for the 1% AEP roughness sensitivity test only.

Table 7-4 to Table 7-6 build on the quoted values detailed in Section 2.2.2 and 3.2.3 and provide upper and lower bound values for a variety of surfaces. These have been used as a guide; actual values used for the sensitivity analysis are presented in the individual AFA modelling reports and take into consideration local factors as described above.

Table 7-4: Roughness bounds for river channels

Channel substrate	Roughness values (Manning's n)		
	Lower Bound Value	Typical Value	Upper Bound Value
Bedrock	0.023	0.025	0.028
Cobbles (64-256mm)	0.04	0.055	0.07
Coarse Gravel	0.022	0.035	0.04
Gravel (2-64mm)	0.028	0.03	0.035
Sands	0.023	0.025	0.032
Silt	0.02	0.022	0.025
Clay	0.018	0.02	0.023
Concrete	0.018	0.02	0.022

Table 7-5: Roughness bounds for river banks

Bank material	Roughness values (Manning's n)		
	Lower Bound Value	Typical Value	Upper Bound Value
Scrub/Long Grass	0.03*	0.04	0.06*
Bushes	0.04*	0.06	0.08*
Trees – flood level not reaching branches	0.05	0.07	0.13
Trees – flood level reaching branches	0.1	0.15	0.2
* these values are not from published literature, but represent a reasonable uncertainty bound to guide the modeller.			

Table 7-6: Roughness bounds for floodplain surfaces

Floodplain material	Roughness values (Manning's n)		
	Lower Bound Value	Typical Value	Upper Bound Value
General Natural Surfaces	0.030	0.040	0.050
Buildings	0.100	0.300	1.000
Inland Water	0.025	0.035	0.045
Roads, Tracks and Paths	0.013	0.015	0.017
Non-coniferous Woodland	0.060	0.070	0.100
Coniferous Trees	0.080	0.100	0.120
General Manmade Surfaces	0.015	0.017	0.020
Glasshouses	0.100	0.200	0.300
Rock	0.040	0.050	0.070
Mixed Vegetation	0.060	0.080	0.110

7.2.3 Building representation

Buildings in the floodplain can dictate flow paths, and some consideration of how the chosen representation of buildings influences the flood extent is required.

The preferred method for floodplain and building representation has been to apply an increased roughness for building footprints.

The sensitivity test has identified whether the selected approach is appropriate by increasing the threshold level of buildings by 300mm, or an appropriate (higher) level based on local site information.

The results of the sensitivity test have determined if the alternate approach needs to be adopted for all model runs for the AFA or if it can remain as a demonstration of sensitivity only.

This sensitivity test has only been applied where properties are shown to be located within the flood extents.

7.2.4 Water level boundaries

In cases where the downstream boundary of the model has been linked to a model downstream, there is no requirement to test the boundary, which will be based on a rating relationship rather than a water level boundary.

The effect of rising sea levels has been investigated through the future event scenarios detailed in Section 8.1. The increases in levels highlighted for the MRFS have been applied for this sensitivity test.

Further consideration has been given to the initial conditions in lakes within hydraulic models. Where long term level data is available this has been reviewed to determine levels in a typical year and in an extreme year during winter months to determine a suitable shift. Where no long term data is available, an estimate of appropriate changes in water levels is required and an increase in water levels of 1m has generally been adopted.

7.2.5 Flow volume

The sensitivity to the hydrograph duration has been assessed where design storm hydrographs have been developed from limited data. Where observed data from significant flood events is available, it is considered a reasonable approximation of the flood duration has been made and no sensitivity test has been required. Table 7-7 details a range of flood duration multipliers reflecting the basis for the development of the design event hydrographs.

Where the 1% AEP flow remains in bank, sensitivity to flow volume will not be investigated because the peak flow, and therefore corresponding peak water level would remain unchanged, and would stay within bank.

Table 7-7: Flood duration multipliers for flow volume sensitivity test

Description of site	Sensitivity multiplier applied to flood duration
Flood duration has been developed from a single observed event data or multiple events below the 10% AEP.	1.2
Flood duration has been developed from catchment descriptors and there are few or no lakes in the upstream catchment (FARL>0.9)	2
Flood duration has been developed from catchment descriptors and there are extensive lakes in the upstream catchment (FARL<0.9)	9

7.2.6 Afflux/head loss at key structures

General modelling parameters often fail to fully represent the head loss that can occur at complex structures. Whilst it is not realistic to model these structures exactly as observed on site, it is feasible to investigate the effect of greater head losses resulting from this known complexity.

In some cases, such as where pipe crossings exist which have the potential to alter the flow regime within or upstream of a culvert, or there is unusual skew apparent on the approaches or exits, then specific modelling approaches to reflect these observed constriction, such as partially blocking the culvert, have been adopted.

Where there is complexity that is less easily quantified, such as changes in culvert shape through the length, sensitivity tests incorporating additional contraction and expansion losses to account for these complexities have been completed to determine the effect of incorporating these in the model. These losses are relative to the velocity head and can be applied simply through a general head loss unit and the multiplier K, Table 7-8.

Table 7-8: Coefficients for contraction and expansion head losses

K Value	
Calculated expansion loss	$K = \left(1 - \frac{W_1}{W_2}\right)$ where W1 and W2 are the upstream and downstream widths
Typical bridge expansion	K = 0.5
Abrupt expansion	K = 0.8
Square edged contraction	K = 0.3 (lower bound 0.23, upper bound 0.35)
Round edged contraction	K = 0.15 (lower bound 0.1, upper bound 0.2)
Typical bridge contraction	K = 0.3
Abrupt contraction	K = 0.6

Based on Table 5-2 in the HEC-RAS manual

This analysis has been completed for hydraulically significant structures only, i.e. those that are likely to have an impact on either scale of flood risk or future flood risk management measures.

Hydraulically significant structures have been identified in the AFA modelling report. They are those structures that show a hydraulic jump in the long section plot, and are also situated near a receptor that could be at risk of flooding. If the structure is in a low risk area it is not deemed to be hydraulically significant.

Table 7-9: Roughness bounds for culverts

Culvert material	Roughness values (Manning's n)		
	Lower Bound Value	Typical Value	Upper Bound Value
Precast concrete	0.009	0.008	0.013
Monolithic concrete construction	0.011	0.012	0.014
Brickwork (well pointed)	0.014	0.016	0.017
Brickwork (in need of pointing)	0.018	0.021	0.023

7.2.7 Timing of tributaries

Coincidence of flood peaks on tributaries needs to be considered in the context of the catchment and the potential impact. A shift in the timing of tributaries to coincide the peaks will give higher peak flows downstream of the confluences. Therefore, this test was only required if the resulting increase in flows was greater than those tested in the flow sensitivity (as detailed in Section 7.2.1). To be truly useful, this test would also require good confidence in the model hydrology.

A simple alignment of flood peaks on tributaries is considered too conservative, and instead a shift in the timing of flood event hydrographs has been limited to a maximum of +/-10% of the tributary event duration.

7.2.8 Critical storm duration

The critical storm duration used in model was based on timing for the main watercourse. As the peak flow is matched to the HEPs on the watercourse, the impact of changing the storm duration is to increase or decrease the volume of the hydrograph. This is unlike the standard approach to varying storm duration where a longer storm tends to result in a longer, but lower hydrograph than a shorter duration storm. In cases where tributaries drain considerably smaller catchments than the main river, it is likely the storm duration has been overestimated, and therefore flood volumes are too large. This may result in over estimation of flood extents.

In each case where a main river and a tributary share a common storm duration the flood extent for the tributary was examined. If the extent was limited (i.e. largely in-bank) testing alternative storm durations was not required as the only adjustment would be to reduce the volume of the hydrograph, thereby giving even less out of bank flooding. However, if there was considerable floodplain inundation arising from the tributary, an additional model run was carried out using the critical storm duration for the tributary, matched to the 1% AEP flows at the HEPs. If this run resulted in smaller flood event, the base model was adjusted accordingly.

7.2.9 Cell size

Where a cell size greater than 2m has been used, and there are complex flow routes (such as around buildings), the model will be run with a 2m grid resolution. This will allow the potential for development of additional flow paths to be identified. It is not proposed to test sensitivity to cell size at a resolution less than 2m, or where flood extents are limited, or are simple (i.e. across open floodplain).

7.3 Joint probability analysis

7.3.1 Fluvial and tidal

The potential for a joint probability event has been considered in relation to the impact of a fluvial event in conjunction with extreme still water tidal levels only, for example the joint probability of waves and tidal levels in conjunction with fluvial flows has not been investigated.

A staged approach has been adopted to determine those locations where a detailed joint probability analysis is required. This has consisted of using the hydraulic models to screen out those sites where the flood risk associated with a joint probability event in excess of the 1% AEP event is limited or of no significant consequence. This scoping event has combined the 2% AEP fluvial event with the 2% AEP tidal event. This has been compared against the extents produced by a 1% AEP fluvial event in conjunction with a 50% AEP tidal event, and a 1% AEP tidal event in conjunction with a 50% AEP fluvial event.

Should flood risk from the screening event have been found to impact receptors beyond the fluvial only or tidal only flood risk extents, a more detailed joint probability analysis would have been carried out. However, in all cases there was little, if any, increase in the extent of flooding and no new receptors at risk.

The results of the joint probability sensitivity testing are detailed in the water level boundary section of the relevant AFA hydraulic modelling report.

7.3.2 Main river and tributaries

No joint probability testing has been carried out at the confluence of tributaries and the main river. This is because the CFRAM approach is intended to model the same design event on all watercourses at the same time. As the flows are scaled to the HEPs it is not possible to run alternative combinations; the default position is to match the 1% AEP on the main river with the 1% AEP event on the tributary. In addition, as the flows have all been calculated using FSU catchment descriptor methodology at HEPs along the watercourse, the contribution of tributaries is automatically taken into account at the downstream HEPs.

7.4 Sensitivity testing results

As discussed in the introduction to this section, a screening assessment was undertaken in each AFA to determine which sensitivity tests would be undertaken. The result of the screening assessment is detailed in the relevant AFA hydraulics report, and is summarised in the UoM30-31 Hydraulic Report.

8 Model outputs and mapping

8.1 Model run scenarios and design events

There are a suite of model scenarios and associated design events for which the hydraulic models have been run to fulfil the requirements for the WCFRAM study.

The full suite of design events include the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP events.

The three scenarios represent different time periods; a present day scenario, a Mid-Range Future Scenario (MRFS) and a High End Future Scenario (HEFS). The objective of the future scenarios is to understand the implications of climate change and land use change on flood risk over the period to 2100. The MRFS is intended to represent a 'likely' future scenario, whereas the HEFS is intended to represent a more extreme, but still possible, future scenario. Full details of the development of the hydrology for the future scenarios are recorded in the Hydrology Report but the general changes are summarised in Table 8-1. The future scenarios have been run for the 10%, 1% and 0.1% AEP design events.

Table 8-1: Allowances for future scenarios

	MRFS	HEFS
Flood Flows	+20%	+30%
Mean Sea Level Rise	+500mm	+1000mm
Urbanisation	+20% to URBEXT	+30% to URBEXT
Land movement	-0.5mm/year for Kinvarra. Nothing elsewhere	-0.5mm/year for Kinvarra. Nothing elsewhere

8.2 Flood hazard mapping

The flood mapping deliverables include flood extent maps, Flood Zone maps, flood depth maps, flood velocity maps and risk to life maps. The Flood Zone maps are primarily used for development planning and management, and represent an undefended situation as discussed in Section 4.2.2. The maps have been produced for all modelled watercourses.

For each of these mapping deliverables there are different combinations of scenario and design event model runs required. Table 8-2 details the flood mapping requirements for the WCFRAM.

Table 8-2: Flood mapping requirements

Map type	Flood event probabilities to be mapped for each scenario		
	Present day	MRFS	HEFS
Flood extent	All probabilities	All probabilities	10%, 1%, 0.1%
Flood Zone	1%, 0.1%	1%, 0.1%	Not required
Flood depth	All probabilities	10%, 1%, 0.1%	Not required
Flood velocity	All probabilities	Not required	Not required
Risk to life	10%, 1%, 0.1%	Not required	Not required
Wave overtopping	10%, 1%, 0.1%	Not required	Not required

8.2.1 HPWs

One of the advantages of using a 2d modelling package, such as Tuflow, is that the outputs from the model can be used directly to generate hazard maps, with little or no post-processing required. The only exceptions are the flood extent and Flood Zone maps, which are polygons generated from the outer extents of the depth grid. The main advantage of this approach to generating maps is that the outputs are consistent. However, the resulting grid (depth, hazard etc.) is linked to the resolution of the DTM used in the model. This means the resolution of the maps may vary from AFA to AFA and between models within an AFA. This is particularly true where varying cell sizes have been used in the models.

Where an HPW flows to or from an MPW, this is indicated on the map with a note directing the viewer to the appropriate adjacent map.

8.2.2 MPWs

The maps for the MPWs have been produced by interpolation of water levels between cross sections, and projection of those levels across the DTM. Owing to the coarse resolution of the section spacing this means the accuracy of the outlines is lower than for the HPW maps. A relic of this process is wet and dry islands; these are disconnected areas of floodplain which are shown to have flooded, or areas within the floodplain which are raised above water levels and are shown to be dry. In general, wet islands have been removed where there is not obvious link to the watercourse network, however in some instances where there are turloughs present these have been left in as they are representative of likely flood risk. Dry islands have been removed where they are less than 750m² in area, islands larger than this have been left in the maps.

8.3 Long section plots

For each modelled watercourse a long section plot has been produced. These plots show the design water levels against a corresponding area of mapping. The plots have been produced to show water flowing downstream across the page from left to right. To ensure consistency across the images it has, in some cases, been necessary to rotate the mapping orientation to align with the long section plot. This is indicated by the north arrow on the mapping.

In some cases it is possible that the long section plot indicates water levels are lower than the bank crests while the plan indicates out of bank flooding. The reason for this anomaly is that at a particular cross section the bank is raised, or includes a wall, but flows have bypasses the structure from upstream. It does not indicate an error in the modelling.

The long section plots are useful for indicating where bridges and culverts cause constrictions and result in afflux (headloss) which may result in flood risk locally.

8.4 Presentation of uncertainty

Uncertainty bounds have been developed by extracting the largest flood extents produced by the sensitivity tests discussed in Section 7 in all locations along a river or coastal reach. The final uncertainty bound is therefore the result of all sensitivity tests overlain using a GIS package to produce a final merged uncertainty bound.

In general, the hydrology sensitivity test has produced the greatest uncertainty extents, reflecting the fact that hydrology is usually the greatest source of uncertainty in modelling. However, in the particular locations where multiple sensitivity tests produce equivalent extents, a review has been undertaken, and where necessary additional model runs completed, to incorporate a greater worst case scenario by modelling a combination of uncertainties for that specific location.

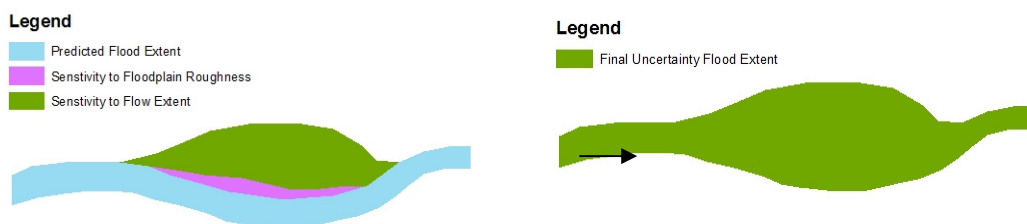
In summary, the approach is as follows:

1. Complete hydraulic modeller led sensitivity assessments and document findings
2. Map 2D model results and review extents to identify where multiple sensitivity tests produce similarly extreme outlines.
3. Run extreme sensitivity model run if required.
4. Overlay and merge in GIS to develop a final uncertainty bound.

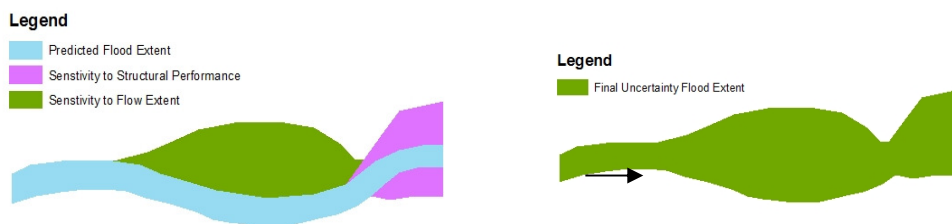
Examples of the output of this process is shown in Figure 8-1 which illustrates sensitivity to one or a number of parameters. It also provides an example of one model location which may be sensitive to a number of different parameters. In such a case, an additional, worst case sensitivity run will be carried out.

Figure 8-1: Sensitivity run example outputs

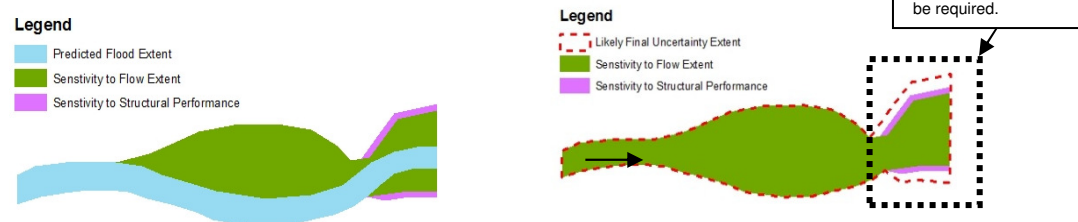
Example 1 – A single sensitivity test produces the greatest bound



Example 2 – Different locations are sensitive to different sensitivity tests



Example 3 – Different sensitivity tests produce similarly large extents in critical areas



8.5 Flood risk maps

Maps have been produced showing flood risk to a number of receptors within AFAs. These maps are based on a GIS interrogation of the receptor data against the 10%, 1%/0.5% and 0.1% flood extents for fluvial and coastal scenarios. All the risk maps have been produced for the existing risk (present day), and MRFS. A summary of the receptor data examined in each case is provided in Table 8-3.

Table 8-3: Risk map receptors

Map type	Receptors mapped
Specific risk - No. inhabitants	Gridded density of inhabitants at flood risk
Specific risk - Type of activity	Presence or absence of property, infrastructure, rural activities or economic activities at flood risk within the AFA.
Specific risk - Risk density	Annual average damages (AAD (€))
General risk - Social	Residential Properties
	Residential Homes - Children
	Residential Homes - Disabled
	Residential Homes - Elderly
	Primary Schools
	Post-primary Schools
	Third Level Education
	Health Centres
	Prisons
	Fire Stations

Map type	Receptors mapped
	Garda Stations
	Civil Defence
	Ambulance Stations
	Hospitals
	OPW Buildings
	Government Buildings
	Local Authority Buildings
General Risk - Environmental	Pollution Sources
	Groundwater abstraction for Drinking water
	Recreational water including Bathing water
	Special Area of Conservation
	Special Protected Area
	S4 and S16 licences
	Shellfish waters including fresh water pearl mussel areas, surface drinking water, and nutrient sensitive areas.
General risk - Cultural heritage	Architectural Heritage
	National Monuments
	National Heritage Area
	Proposed National Heritage Area
General risk - Economic	Commercial Properties
	Airports
	Road Networks
	Rail networks & Stations
	Ports & Harbours
	Infrastructure: ESB Power Stations, ESB HV Substations, Bord Gais Assets, Eircom Assets
	Water Supply
	Oil infrastructure



Registered Office

**24 Grove Island
Corbally
Limerick
Ireland**

T: +353 (0) 61 345463
e: **info@jbaconsulting.com**

**JBA Consulting Engineers and
Scientists Limited**

Registration number 444752

Visit our website
www.jbaconsulting.com

Western CFRAM Unit of Management 30 and 31 - Corrib and Owengowla

**Final Hydraulic Model Report
Volume 1b: Hydromorphology and Coastal
Erosion Assessment**

September 2016

**Office of Public Works
Trim
Co. Meath**



 **WESTERN
CFRAM
STUDY**
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

JBA Consulting

24 Grove Island
Corbally
Limerick
Ireland

JBA Project Manager

Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM

Revision History

Revision ref / Date issued	Amendments	Issued to
April 2015 version 1.0	First Issue	Rosemarie Lawlor
July 2016 version 2.0	Minor typos/format	Clare Butler
September 2016 version 3.0	Final issue including coastal erosion	Clare Butler

Contract

This report describes work commissioned by The Office of Public Works, by a letter dated (28/07/11). The Office of Public Works' representative for the contract was Rosemarie Lawlor. Clare Bithell and Sam Willis of JBA Consulting carried out this work.

Prepared by Claire Bithell BSc
..... Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM

Reviewed by Jonathan Cooper BEng MSc DipCD CEng MICE
MCIWEM C.WEM MIOd

Purpose

This document has been prepared as a draft report for The Office of Public Works. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to the Office of Public Works.

Copyright

Copyright – Copyright is with Office of Public Works. All rights reserved. No part of this report may be copied or reproduced by any means without the prior written permission of the Office of Public works.

Legal Disclaimer

This report is subject to the limitations and warranties contained in the contract between the commissioning party (Office of Public Works) and JBA.

Carbon Footprint

A printed copy of the main text in this document will result in a carbon footprint of 157g if 100% post-consumer recycled paper is used and 157g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to reduce its per capita carbon emissions.

Contents

1	Introduction.....	1
1.1	Background and project scope	1
1.2	Report overview.....	1
1.3	Study background.....	2
2	Hydromorphology assessment methodology	3
2.1	Overview	3
3	Hydromorphology results.....	7
3.1	Mapping Outputs	7
3.2	Structures identified from hydromorphology appraisal	8
4	Hydromorphology summary	14
5	Coastal erosion assessment background	17
5.1	Scope.....	17
5.2	Available Data.....	17
6	Coastal erosion assessment analysis.....	18
6.1	Review of existing ICPSS data.....	18
6.2	Methodology	19
6.3	Findings	19
6.4	Recommendations	20
7	Coastal erosion assessment summary	21
	Appendices.....	22
A	Hydromorphology maps	23
B	Coastal erosion maps	24

List of Figures

Figure 1-1: AFAs within UoM 30 and 31	1
Figure 2-1: Flow chart of hydromorphology appraisal process	4

List of Tables

Table 3-1: Mapping output legend and explanation	8
Table 3-2: Structures at risk of sedimentation from reach scale sedimentation processes	9
Table 3-3: Structures at risk of sedimentation from local scale sedimentation processes.....	11
Table 3-4: Structures to be monitored for sedimentation from reach scale sedimentation processes	13
Table 4-1: Summary of findings and recommendations for each AFA.....	15
Table 6-1: Review of ICPSS outputs against latest aerial imagery	18
Table 6-2: Summary of coastal erosion risk categorisation.....	19

Abbreviations

AEP	Annual exceedence probability
AFA	Area for further assessment
AP	Aerial photography
CFRAM	Catchment flood risk assessment and management
HEFS	High end future scenario
HPW	High priority watercourse
ICPSS	Irish Coastal Protection Strategy Study
MIKE21	Hydraulic modelling software package
MPW	Medium priority watercourse
MRFS	Medium range future scenario
NHA	National Heritage Area
pNHA	proposed National Heritage Area
SAC	Special Area of Conservation
SPA	Special Protection Area
UoM	Unit of management
WCFRAM	Western Catchment flood risk assessment and management

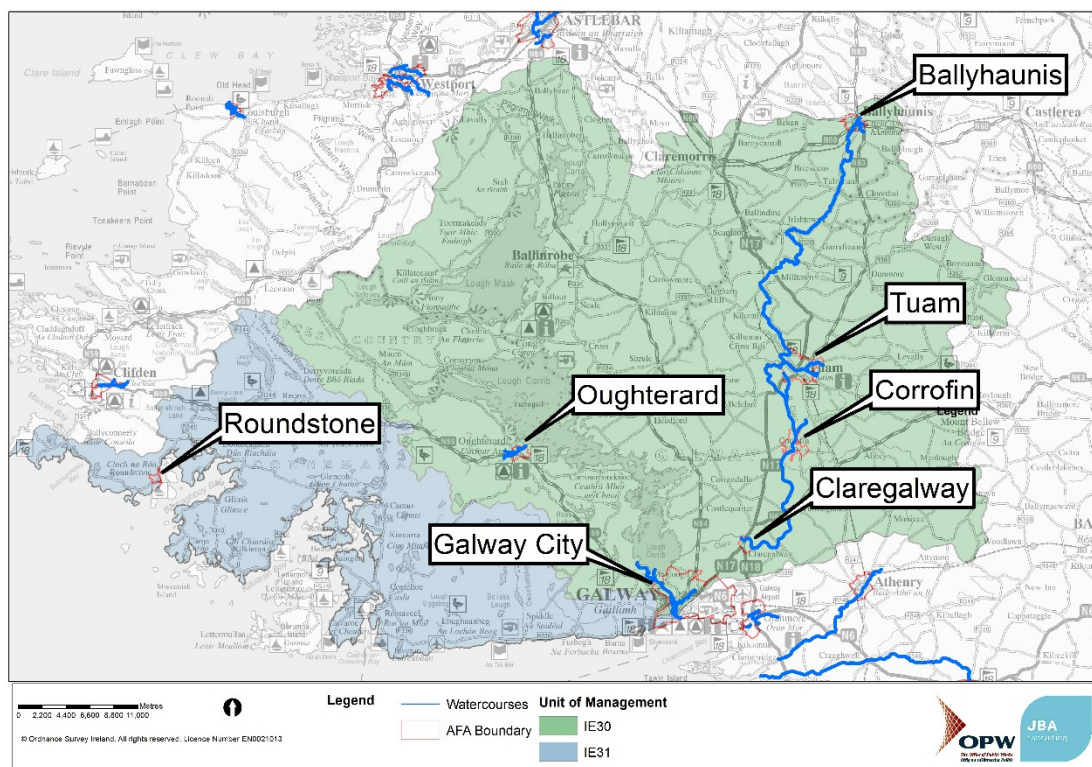
1 Introduction

1.1 Background and project scope

This report describes the investigations into hydromorphology and sediment transport issues, and coastal erosion across the Western Catchment-based Flood Risk Assessment and Management (WCFRAM) study area. The report covers the Areas for Further Assessment (AFAs) within Unit of Management (UoM) 30 and 31, Corrib and Owengowla, as shown in Figure 1-1:

- Ballyhaunis
- Claregalway
- Corrofin
- Galway City
- Oughterard
- Tuam
- Roundstone

Figure 1-1: AFAs within UoM 30 and 31



The focus of the study is on channel hydromorphology and coastal erosion in so far as it may affect flood risk within the AFAs being investigated within the CFRAM. This report is one element of the hydraulic modelling investigations and presented within this suite of reports provides valuable context for understanding the implications of the historical management of watercourses across the WCFRAM and the best approaches for continued management into the future. Results reported in the hydraulic modelling investigations can therefore be considered in light of the findings from this study.

This work will also feed into the Preliminary Options Investigation Phase of the study supporting the discussion within the Strategic Environmental Appraisal on the implications of proposed measures and options.

1.2 Report overview

This report is one of a series which describe the work undertaken as part of the CFRAM, and together they provide a description of the approach taken to identifying flood risk, and a discussion

of the results of the analysis and potential flood management measures, where they are appropriate.

This report should be read in conjunction with the following supporting documents:

- Western CFRAM UoM30-31 Hydrology Report¹
- Western CFRAM UoM30-31 Inception Report²
- Western CFRAM Flood Risk Review Report³
- Western CFRAM SEA Scoping Report⁴
- Western CFRAM Defence Asset Database: Handover Report and accompanying database files⁵

The reports in the suite for the Hydraulic Modelling are:

- Western CFRAM UoM30-31 - Hydraulic Modelling Report
- Western CFRAM UoM30-31 - Hydraulic Modelling Report: Volume 1a – Hydraulic Modelling Method Statement
- Western CFRAM UoM30-31 - Hydraulic Modelling Report: Volume 1b – Hydromorphology and Coastal Erosion Assessment (this report)
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 2 – AFA Modelling Reports
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 3 – Flood Risk Maps
- Western CFRAM UoM30-31- Hydraulic Modelling Report: Volume 4a – Hydraulic Model and Check Files

1.3 Study background

The Inception Report for UoM 30 and 31 was delivered in October 2012. This report consisted of a baseline review of available data and the development of the proposed methodology for the hydrological and hydraulic modelling investigations to be completed within this phase.

The method statement for the hydrological analysis detailed in the Inception Report has been developed and finalised in the UoM 30 and 31 Hydrology Report. This work has developed design flows at a series of Hydrological Estimation Points (HEPs) along all watercourses to be modelled. The detailed development of the hydrology has not been reiterated here and the reader is referred to the Hydrology Report for full details of the hydrological analysis. Design flows have been extracted directly from the Hydrology report and are summarised in the AFA modelling reports.

The Hydrology Report also provides guidance on the development of appropriate design storm hydrographs for each AFA for the purposes of the hydraulic modelling. These methods are summarised in this report to provide clarity on the application of the design event hydrology as this work has been undertaken in the hydraulic modelling phase.

The Inception Report identified all High Priority and Medium Priority Watercourses (HPWs and MPWs) to be modelled. HPWs are those watercourses that dictate flood risk within an AFA boundary as originally delineated within the Flood Risk Review (FRR) Report. HPWs therefore extend a short distance upstream and downstream of an AFA but do not include watercourses with catchments less than 1km². HPWs have been modelled to a greater level of detail than MPWs. MPWs are the watercourses which link two AFAs together and the watercourses that extend downstream of an AFA to the sea. Coastal AFAs do not have a downstream MPW associated with them.

In total, 56 km of HPW and 102 km of MPW have been modelled within UoM 30 and 31, along with the coastline in Roundstone and Galway City.

¹ JBA Consulting (2014), Western CFRAM Unit of Management 30 and 31 - Corrib and Owengowla Hydrology Report, Final Report, Office of Public Works

² JBA Consulting (2012), Western CFRAM Unit of Management 30 and 31 - Corrib and Owengowla Inception Report, Final Report, Office of Public Works.

³ JBA Consulting (2012), Western CFRAM Flood Risk Review, Final Report, Office of Public Works.

⁴ JBA Consulting (2013), Western River Basin District Catchment-based Flood Risk Assessment and Management (CFRAM) Strategic Environmental Assessment, Scoping Report, Office of Public Works.

⁵ JBA Consulting (2013), Western CFRAM Defence Asset Database, Handover Report, Office of Public Works.
WCFRAM UoM 30-31 Volume 1b Hydromorphology and Coastal Erosion Assessment_v3.0.docx

2 Hydromorphology assessment methodology

2.1 Overview

The study has approached the investigation in two stages:

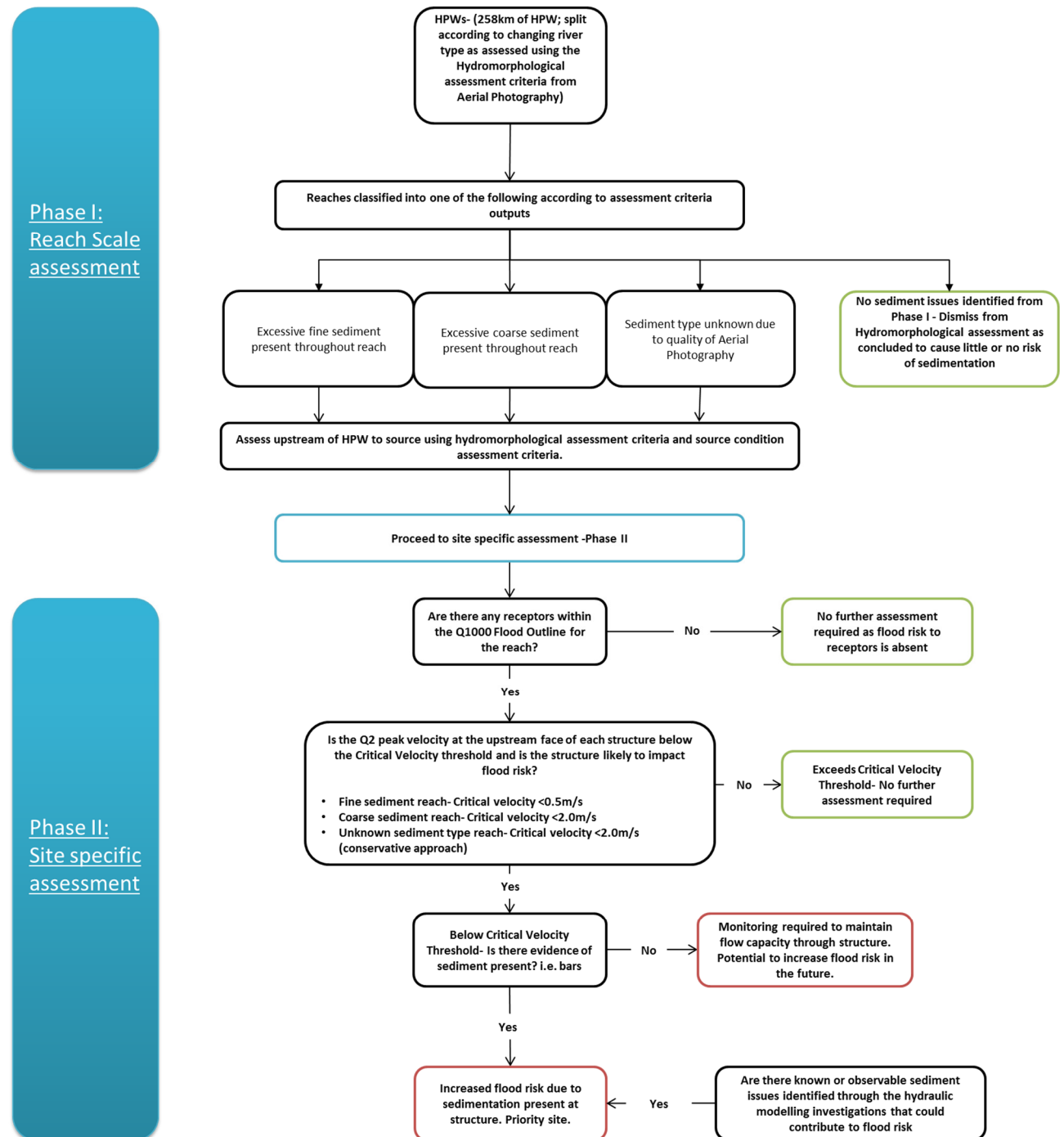
1. A catchment based hydromorphology audit which has identified key watercourses susceptible to sedimentation at a catchment scale.
2. A site specific assessment using the outputs from the hydromorphology audit along with the knowledge of the sites developed as part of the hydraulic modelling investigations to identify key structures along these watercourses where sedimentation could be relevant to flood risk.

This information has been supplemented by the finding of the asset inspection work, which highlighted where scour was present beneath structures.

The aim of this process therefore is to produce a hydromorphological assessment of the key watercourses informed by both available catchment data and local knowledge from across the project team.

The flow chart in Figure 2-1 summarises the stages of the hydromorphology appraisal and each stage is discussed in further detail in the following sections.

Figure 2-1: Flow chart of hydromorphology appraisal process



2.1.1 Phase I: Reach Scale assessment

All HPWs throughout Western CFRAM have been assessed using a Hydromorphological Assessment Criteria developed for this study allowing for categorisation of river types and sediment behaviour to be made about the rivers throughout WCFRAM. The assessment utilises readily available information including Aerial Photography (APs), soils maps and site photos. The Western River Basin Management Plan was reviewed for suitable information to support the analysis.

The Hydromorphological Assessment Criteria included the River Type, the activity of the channel, the vegetation in the floodplain and along the banks, the sediments in and outside of the channel and the evidence of historical activity.

HPWs have been taken forward to Phase II where the catchment assessment indicated the presence of excessive fine or coarse sediment. Where no sediment issues were determined by the desk based assessment no further analysis of the HPW has taken place. Where it was not possible to identify the sediment type due to poor quality data, a conservative approach has been adopted and the HPWs have been taken forward to the Phase II part of the assessment.

All watercourses, with the exception of those where no sediment issues were observed, have then been traced to their source to determine the source condition and to provide a wider understanding of system processes.

The Phase I assessment has identified 41 HPWs to take forward to the Phase II assessment.

2.1.2 Phase II: Site specific assessments

All HPWs identified from Phase I have been taken forward to Phase II in order to provide a deeper understanding of local issues associated with the watercourses susceptible to sedimentation. This identified those structures or channels where sedimentation could lead to increased flood risk to surrounding receptors with flood risk being the primary driver.

Initially the 0.1% AEP flood extent developed as part of the hydraulic modelling work has been used as a screening tool to identify sites where there are local flood risk receptors. Whilst high levels of blockage at a structure from sedimentation could result in a larger flood risk extent, a detailed blockage analysis is outside the scope of the study and so has not been undertaken as part of the hydraulic modelling investigations. The 0.1% AEP extent is therefore considered to be a reasonable proxy for the implications of increased sedimentation in lower order events and allows the study to focus only on those sites where sedimentation is most likely to affect flood risk.

Where there are receptors within the 0.1% AEP flood extent along the watercourse, then the peak velocity from the 50% AEP event (seen as a typical 'bankful' or geomorphologically effective flow) has been extracted from the hydraulic modelling results at the upstream face of key structures. This has been compared against the critical velocities extracted from the Hjulström curve for the dominant sediment type on each watercourse. Where the modelled velocities do not exceed the critical velocity, deposition of sediment is assumed to be likely to occur leading to a reduction in the capacity of the structure over time and hence an increase in flood risk.

A final stage has reviewed photos of the key structures identified to confirm if sedimentation is apparent at the site at the date taken. Where there is visible evidence at the identified structure the structures have been flagged as priority sites, (and are shown on the associated maps in red) where there is not visible evidence at the identified structure, it has been flagged for monitoring only (these sites are shown on the associated maps in green).

This review of the catchment audit HPWs has been supplemented using the local knowledge available within the project team developed through the study to date and in particular as part of the hydraulic modelling investigations, or from other sources such as Local Authorities. This allows for problems that may not have been picked up in the catchment scale approach to be identified. For example, sedimentation may have been observed at key structures from survey data or site photographs which is causing flooding or concerns may have been raised by local authorities which were screened out in the broad catchment scale approach.

In each case, the structures identified have been assessed using the same Phase II assessment criteria to determine if they are likely to increase flood risk to surrounding receptors. That is if a

structure has been observed with sedimentation issues in an entirely rural area, based on the 0.1% AEP flood extent, then it will not have been flagged as a priority structure in the associated maps.

3 Hydromorphology results

3.1 Mapping Outputs

The results of the hydromorphology assessment, including the findings of the Phase 1 assessment are presented in the AFA specific maps presented in Appendix A at the end of this report.

The findings of Phase 1 are presented in the line colour and line style of each watercourse assessed. The line colour dictates the sediment characteristic of the reach of interest, i.e. the reach within the AFA boundary. The line style dictates the source condition of the reach of interest, i.e. the likelihood of the upstream watercourse to carry sediments to the AFA.

The OPW arterial drainage schemes are also presented on these maps. This information can be important to understand where, despite the hydromorphology audit highlighting a watercourse with high risk of sedimentation, there are no observed sedimentation effects or high priority structures identified. Such watercourses could have had problems historically which are now resolved as a result of the maintenance regime. Examples of such findings are Swinford and Athenry.

The findings of the assessment are presented on the accompanying maps as "high priority" or "for monitoring only". High priority structures are those where there are known and observed sedimentation issues. Structures for monitoring are those where there are no observed problems but the catchment audits and flow velocities suggest sediment build up is likely to be a problem in the long term.

Furthermore receptors that are likely to be susceptible to high levels of sediments in the water have been overlain on the maps. These include Special Areas of Conservation (SACs), Special Protection Areas (SPAs), existing and proposed National Heritage Areas (NHAs and pNHAs) and relevant Annex IV sites. The Annex IV sites have excluded groundwater zones but include shellfish and salmonid watercourses.

Table 3-1 below shows the legend used in the mapping outputs and provides an explanation of how the results of the hydromorphology assessment have been translated into a graphical format.

Table 3-1: Mapping output legend and explanation




Map Legend	Explanation
<p>Legend</p> <p>Phase 1 Findings User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.</p> <p>Sediment characteristic</p> <ul style="list-style-type: none"> Coarse sediment reach Fine sediment reach Sediment type unknown- Proceed to Phase II No sediment issues identified from Phase I <p>Source Condition</p> <ul style="list-style-type: none"> Stable or no upstream watercourse Incised but stable Incised valley bottom with reworking Extensive valley side inputs <p>Phase 2 Findings</p> <ul style="list-style-type: none"> High Priority Structures Structures for Monitoring Only <p>Supporting Information</p> <ul style="list-style-type: none"> Structures with Scour AFA Boundary WFD Annex IV Sites WFD Annex IV Sites WFD Annex IV Sites National Heritage Areas Proposed Natural Heritage Areas Special Area of Conservation Special Protection Areas Arterial drainage schemes 0.1% AEP Fluvial Extent 	<p>Phase 1 Findings The findings of Phase 1 are presented in the dashed box and are presented in the map itself as a single line of varying colour and style. The sediment characteristic of the watercourse within the AFA boundary is represented by the colour of the line. The source condition, associated with the relevant watercourse upstream of the AFA boundary, is represented by the style of the line. Therefore a fine sediment reach with an incised but stable source condition will be represented on the map as a yellow dashed line.</p> <p>Phase 2 Findings The findings of Phase 2 of the assessment are represented in the coloured points (red or light orange) situated at relevant structures. The red points are high priority structures and have been identified as having observable sedimentation issues. The light orange structures are for monitoring only; these have been identified through the hydromorphology assessment as being susceptible to sedimentation but for various reasons, such as arterial drainage schemes, no sedimentation issues have been observed at the site.</p> <p>Supporting Information Further data to support the analysis is shown in the supporting information. This includes details of OPW's arterial drainage schemes, the findings of the asset condition assessment and a range of environmental receptors.</p>

3.2 Structures identified from hydromorphology appraisal

The results of the overall assessment have led to individual structures being identified as being high risk of sedimentation which increase flood risk to receptors. The results and node locations of individual structures from the hydromorphology assessment are presented in Table 3-2 and Table 3-3.

Table 3-2 details those high priority structures identified from the catchment based hydromorphology audit. Table 3-3 details those high priority structures where sedimentation is not associated with reach scale deposition but is rather a local risk associated with the channel in the vicinity of the structure. Table 3-4 shows those structures along the reaches identified from the catchment based hydromorphology audit where velocities suggested sedimentation may be an issue but no evidence of sedimentation was observed at the structures.

Table 3-2: Structures at risk of sedimentation from reach scale sedimentation processes

Location	Comments from site specific assessment	Photo
Galway, Castlegar watercourse, 30CAST00397	Fine sediment reach supported by extracted velocity data and local evidence of sedimentation.	
Galway, Castlegar watercourse, 30CAST00302	Fine sediment reach supported by extracted velocity data and local evidence of sedimentation.	
Galway, Castlegar watercourse, 30CAST00115	Fine sediment reach supported by extracted velocity data and local evidence of sedimentation.	








Location	Comments from site specific assessment	Photo
Galway, Castlegar watercourse, 30CAST00085	Fine sediment reach supported by extracted velocity data and local evidence of sedimentation and vegetation	
Galway, Castlegar watercourse, 30CAST00034	Fine sediment reach supported by extracted velocity data and local evidence of sedimentation.	
Galway, Castlegar watercourse, 30CAST00017	Fine sediment reach supported by extracted velocity data and local evidence of sedimentation and vegetation.	

Table 3-3: Structures at risk of sedimentation from local scale sedimentation processes

Location	Comments from site specific assessment	Photo
Ballyhaunis, Curraghfore watercourse, 30CURR00033	Vegetated bar in channel. Channel through this reach has been artificially straightened and over-widened. Shallower depths resulting in build-up of sediment in centre of channel.	
Ballyhaunis, Dalgan, 30DALG02200	Heavily vegetated channel. Channel downstream of this reach has been artificially straightened and over-widened. Lower velocities resulting in build-up of sediment encouraging plant and weed growth in the channel.	
Ballyhaunis, Devils, 30DEVL00011	Heavily vegetated channel. Channel upstream of this reach has been artificially straightened and over-widened. Lower velocities upstream resulting in build-up of sediment encouraging plant and weed growth in the channel.	
Galway, Nun's Island, 30NUNS00043	Vegetated bar on right bank of channel. The whole channel is heavily modified and over-widened. Lower velocities downstream of Bergers Bridge have resulted in build-up of sediment encouraging plant and weed growth in the channel.	


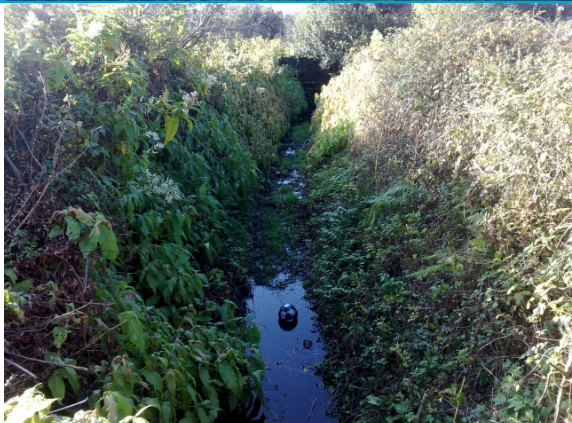
Location	Comments from site specific assessment	Photo
Oughterard, Owenriff, 30ORIF00203	Gravel bar on right bank. Widened channel upstream of bridge resulting in reduced flows. Downstream structure appears reasonably clear so minor issue only.	
Oughterard, Tonweeroe, 30TONW00016	Heavily vegetated channel. Channel upstream of this reach has been artificially straightened and is heavily incised. Structure at downstream of this reach likely reducing flows resulting in sediment deposition and plant growth in the channel.	

Table 3-4: Structures to be monitored for sedimentation from reach scale sedimentation processes

Location	Comments from site specific assessment	Photo
No structures identified in UoM 30 and 31		

4 Hydromorphology summary

This study has completed a preliminary assessment of the hydromorphological issues with respect to flood risk only.

A catchment wide analysis using geomorphic auditing principles, analysing the watercourses for sediment loading, has identified sources and pathways of sedimentation. The findings of this analysis, presented in the accompanying maps, are river systems susceptible to sedimentation. In addition to supporting the identification of key structures where sedimentation is critical to flood risk, this work has also been beneficial to flag those sites where there may be a problem in the future but for reasons, such as the OPW arterial drainage programme, there is currently no risk of flooding resulting from sedimentation.

Using the knowledge from the catchment wide analysis and built up through the hydraulic modelling work completed as part of the WCFRAM, all AFAs have been ground truthed. The findings of the ground truthing have identified those structures where flood risk from sedimentation is a current issue, either as a result of catchment wide sedimentation processes or as a result of local conditions in the vicinity of the structure. Sites where, based on velocity data and the reach scale assessment, there may be a problem in the future have also been flagged for monitoring.

The approach to the assessment reflects the different causes of sediment build up. The hydromorphology audit has identified sensitive watercourses, such as the Castlegar in Galway. The ground truthing builds on the hydromorphology audit but also allows local reach processes, such as straightening and widening, to be flagged.

Table 4-1 summarises the findings of the assessment for each AFA and provides recommendations for the incorporation of the findings into future analysis, within and without the WCFRAM, based on an understanding of the broader flood risk management issues.

Table 4-1: Summary of findings and recommendations for each AFA

AFA	Reach in potential deposition from audit	Historic issues with sediments and erosion	Arterial Drainage active in this AFA	Sites where modelling and site surveys have identified sediment and erosion	Conclusion	Recommendations for further consideration		
						Structures where modelling of long term impacts of sedimentation should be considered in the FRMP	Watercourses /sites to include in maintenance regime	Watercourses where scheme works may be required and would need to seek hydromorphic support in SEA
Ballyhaunis	Yes	No known issues.	Yes	30CURR00033, 30DALG02200, 30DEVL00011.	Sediment is a reach level issue but is generally managed through the arterial drainage schemes. Flood risk is exacerbated by sedimentation at a limited number of key structures.	30CURR00033, 30DALG02200, 30DEVL00011.	Curries, Devlis and Dalgan.	Curries, Devlis and Dalgan in the vicinity of the structures identified.
Corrofin	Yes	No known issues.	No	No key structures identified with current sediment issues.	No evidence of sediment issues associated with flood risk.	None	None	None
Galway City	Yes	Canals in Galway, especially in the Western Canal System, are heavily silted. The Castlegar is silted to some degree. The Upper Corrib has some evidence of shoaling. The loop around Jordans Island is thought to have deteriorated over recent years.	Yes	30NUNS00043, 30CAST00397, 30CAST00302, 30CAST00115, 30CAST00085, 30CAST00034, 30CAST00017.	Sediment is a reach level issue within the AFA with flood risk particularly a concern along the Castlegar watercourse. The canals do not show significant flood risk at the moment but the current situation could worsen.	None	Castlegar River	Upper Corrib, Castlegar River, Canal systems.
Oughterard	No	The Owenriff was reported to be heavily modified in the 1970s by blasting of the bedrock deepening the river	Yes	30ORIF00203, 30TOWN00016.	Sediment is a local issue only with flood risk exacerbated by local channel modifications encouraging deposition.	30TOWN00016	Tonwee	Locally around structures only.

AFA	Reach in potential deposition from audit	Historic issues with sediments and erosion	Arterial Drainage active in this AFA	Sites where modelling and site surveys have identified sediment and erosion	Conclusion	Recommendations for further consideration		
						Structures where modelling of long term impacts of sedimentation should be considered in the FRMP	Watercourses /sites to include in maintenance regime	Watercourses where scheme works may be required and would need to seek hydromorphic support in SEA
		channel.						
Tuam	No	No known issues.	Yes	No key structures identified with current sediment issues.	Sediment is a reach level issue but is generally managed through the arterial drainage schemes. Flood risk is potentially exacerbated by sediment at multiple locations along the watercourse but risk has been managed so will need to be monitored over time.	None	Suileen	None
Roundstone	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

5 Coastal erosion assessment background

5.1 Scope

The project brief requires the assessment to build on the work completed as part of the Irish Coastal Protection Strategy Study (ICPSS) and develop erosion prediction lines within AFAs at risk from coastal flooding for the Medium Range Future Scenario (MRFS) and High End Future Scenario (HEFS) for 2050 and 2100.

5.2 Available Data

5.2.1 ICPSS

The coastline of the WCFRAM is covered by Phase 4⁶, the West Coast, and Phase 5⁷, the North West Coast, of the ICPSS. The hazard maps for these areas were completed in January 2014.

The ICPSS presents two predictive erosion maps representing the future location of the coastline in 2030 and 2050. These maps have been generated from observed erosion rates extracted from historical mapping and aerial photography. The position of the coastline was compared in two time periods reflecting the data available, the starting point for the analysis was aerial photography from between 1973 and 1975 and the end point for the analysis was aerial photography from 2000 for the west of Ireland. The calculated erosion rates therefore represent the change over a period of approximately 25 years.

A baseline for the existing coastline has been derived from data for the year 2000. Predictive erosion lines have then been extrapolated from the baseline inland using the historically observed erosion rates with no additional allowance for climatic factors such as sea level rise.

Historic erosion rates have been developed typically for reaches of approximately 25m in length for the entire WCFRAM coastline. This dataset has been supplied for use in the development of future scenario erosion lines.

The ICPSS maps were produced at a strategic level only and the report recommends that these lines should not be used in place of detailed local erosion hazard and risk assessment. Furthermore, the study assumes that defences currently in situ will continue to be maintained into the future.

5.2.2 Climate Change Scenarios

The climate change scenarios, the MRFS and HEFS, have been specified in the project brief and are considered to be applicable for future changes to 2100.

- The MRFS is intended to represent a 'likely' future scenario, based on the wide range of predictions available with the allowances for sea level rise etc. within the bounds of widely accepted projections.
- The HEFS is intended to represent a more extreme potential future scenario, but one that is nonetheless not significantly outside the range of accepted predictions available, and with the allowances for sea level rise, etc. at the upper bounds of widely accepted projections.

Two elements of the climate change scenarios are applicable to the analysis of coastal erosion, sea level rise and land movement. The sea level rise climate change scenarios are an increase in levels of 0.5m and 1m in the MRFS and HEFS respectively. Land movement changes are only applicable for coastal sites south of the Galway to Dublin line; therefore, this does not apply to any AFAs within UoM 30-31.

Increases in sea levels of 0.5m and 1.0m will be applied for the MRFS and HEFS respectively.

⁶ Irish Coastal Protection Strategy Study, Phase 5 – North West Coast, Work Packages 2, 3 & 4A – Appendix 4 – Erosion Mapping, The Office of Public Works, January 2014.

⁷ Irish Coastal Protection Strategy Study, Phase 4 – West Coast, Work Packages 2, 3 & 4A – Appendix 4 – Erosion Mapping, The Office of Public Works, January 2014.

6 Coastal erosion assessment analysis


6.1 Review of existing ICPSS data

The coastal erosion hazard analysis for the WCFRAM is limited to within the boundary of the AFAs only. The coastal AFAs within the UoM 30-31 consist of Galway City and Roundstone.

The baseline date for the ICPSS existing coastline is set to the year 2000 and appears to have been based on data from that period. Digital aerial photography is now available online from as late as 2012 for the West of Ireland. A review of the ICPSS existing coastline was made against this latest dataset, paying particular attention to those sites where historical erosion had been observed, Table 6-1.

Without the aerial imagery from 2000 it has been difficult to validate the observed changes in locations. The following table summarises the findings of this review. The findings indicate confirmed observable erosion risk since the year 2000 is present in the rural areas around Galway only.

Table 6-1: Review of ICPSS outputs against latest aerial imagery

Site	Description of change
Galway City	<p>The two sites highlighted to the west coincide with predicted sites of erosion in the ICPSS study. The changes are relatively minor and it is possible coastal protection works have been undertaken on the eastern of the two sites. Minor erosion is shown in the eastern location although this could be a result of image quality in this location. It is also noted that the coastline of the harbour area has changed as a result of recent development, this is not shown on the map below.</p> 
Roundstone	No change.

The hazard prediction lines for 2030 and 2050 from the ICPSS have been extracted and plotted in the maps at the end of the report.

The outputs from the ICPSS study show Galway City has seen some coastal retreat over this period.

UK Climate Projections⁸ reports that average sea levels around the UK are rising on average approximately 1mm/yr although this rate has increased since 1990. This suggests the observed changes already incorporate a degree of sea level rise.

⁸ Jenkins, G.J., Perry, M.C., and Prior, M.J. (2008). The climate of the United Kingdom and recent trends. Met Office Hadley Centre, Exeter, UK.

6.2 Methodology

It is accepted that coastal erosion into the future will be impacted by rising sea levels. The increase in water depth offshore will support increased wave heights which, in conjunction with increasing storm frequency will increase erosion rates at any given site. Erosion rates at a given site however will not be dictated by sea level rise alone. Of equal or greater importance are the sediment transport processes in action along the shoreline both on a local and on a regional basis. It is quite possible that along with sea level rise, the shoreline continues to progress seawards where sediment deposition is sufficient. Furthermore the existing sediment transport processes cannot be assumed to continue to operate as understood currently; the effect of the local topography as well as changes in storminess and hence wave heights and wave direction resulting from climate change will also contribute to the equation. In this context therefore the coastline in any given location must be understood as a site specific dynamic system that will redistribute sediments in response to its own range of influences. A search undertaken to identify literary sources for sediment transport in coastal waters, and thereby the influence of sediments at the sites of interest, across the west of Ireland found no available information. This would appear to be an area where further research is required.

The level of data required to determine coastal erosion lines into the future with any degree of confidence is not available. Simplified approaches as proposed in the project brief, such as the Bruun Rule, are also considered to be unreliable for the reasons stated above and so have not been taken forward.

Instead to fulfil the requirements of the brief the study has focussed on discussion of local landform and its likely influence on coastal erosion. A risk based approach has been applied to determine the likelihood of future erosion based on the historical erosion rates available, the prevalence of the wave climate and the observed shoreline material in each case. As with the ICPSS study it has been assumed that where existing coastal protection works are in place, these will continue to be maintained. Similarly, where there are existing structures which would need to be abandoned or moved for further coastal erosion to occur, it has been assumed that these will continue to be protected.

Four risk categories have been defined for the analysis as follows:

- Low Risk – Active management of the shoreline is likely to protect key assets.
- Moderate Low Risk – Tide dominated environment. No observed erosion over the last 30 years, however no active management of the shoreline is likely.
- Moderate High Risk - Wave dominated environment. No observed erosion over the last 30 years, however no active management of the shoreline is likely.
- High Risk – Erosion has been observed over the last 30 years and there is no active management of the shoreline expected in this location to prevent further erosion.

It is noted that because of the approach adopted there can be no differentiation between the risk associated with the MRFS and the HEFS and the risk boundaries are considered to be applicable to both.

6.3 Findings

Table 6-2 provides a summary of the findings of the analysis and this is also presented in the attached maps in Appendix B.

Table 6-2: Summary of coastal erosion risk categorisation

AFA	Brief Description of Site	Risk Categorisation
Galway City	Manmade coastal frontage to the west of the Corrib and mix of manmade coastal and limestone till shoreline to the east. Some observed erosion has occurred to the east of the Corrib. Erosion protection works are now in place in some of these areas.	Low Risk for manmade reaches, high risk for remaining sites.
Roundstone	Granite till and rock shoreline to the south, manmade coastal frontage to the north. No observed erosion.	Low Risk for manmade reaches, moderate low risk for remaining sites.

6.4 Recommendations

Taking the findings of the screening assessment forward focus for further investigations should be on those areas where there is moderate high or high risk of coastal erosion. Within UoM 30-31 this consists of Galway only. It is noted there is no immediate risk of coastal erosion impacts on property in Galway.

High level methodologies are not appropriate for these further investigations. A more detailed approach is required, which is beyond the scope of the WCFRAM study. It is recommended the modelling undertaken for the ICPSS be used as a base dataset to extend the assessment of coastal erosion potential.

The ICPSS used the MIKE21 SW package and outputs include wave conditions and the associated radiation stresses in the surf zone, from which the wave-induced currents can be derived. The long shore currents and the sediment transport can then be calculated using the MIKE21 flow and sediment transport modules. This approach may not give a direct indication of shoreline regression but will at least provide an indication of changes in shoreline vulnerability, through changes in radiation stresses, during the climate change scenarios; particularly when matched against the previously identified high risk coastal erosion sites.

The ICPSS results include wave conditions at 3 of the 7 moderately high or high risk coastal erosion sites that have been identified in the Western CFRAM study area. The ICPSS also assumed the coastline remains constant and the existing sea defences will be maintained. Further work should be undertaken to extend beyond these assumptions, such as calibration of the wave model against recorded/observed wave data, and concurrent wind data, for an extended period (at minimum including a winter period). This would give some confidence in the models capability to reproduce current conditions, and therefore it's probable capability at predicting future conditions. Without calibration data the best that can be achieved is a reality check on the propagation of the waves applied at the boundaries. It is therefore a priority that the availability of calibration data be reviewed and additional data collection be undertaken if required.

Where there is the potential erosion to impact on an AFA it is recommended that a bespoke model is produced, not tied into the ICPSS licencing restrictions, and a local assessment of nearshore currents and sediment processes undertaken. For example, the model could be developed using the suite of modules such as the Deltares Delft3D software package, namely FLOW⁹ (hydrodynamics and sediment) and SWAN¹⁰ (Waves). Delft3D is a leading computer package that is used to numerically simulate tides, storm surges, currents, waves, transport, morphology, water quality, ecology and biology in natural waters such as harbours, rivers, lakes, reservoirs and coastal seas. It has been developed, calibrated and validated for a large number of applications for marine waters, estuaries and rivers and combinations of these.

⁹ FLOW is a multi-dimensional (2D or 3D) hydrodynamic (and transport) simulation program which calculates non-steady flow and transport phenomena that result from tidal and meteorological forcing on a rectilinear or a curvilinear, boundary fitted grid.

¹⁰ SWAN computes the evolution of random, short-crested waves in coastal regions with deep, intermediate and shallow water and ambient currents. The model accounts for propagation due to current and depth and represents the processes of wave generation by wind, dissipation due to whitecapping, bottom friction and depth-induced wave breaking and non-linear wave-wave interactions. Wave blocking by currents is also explicitly represented in the model.

7 Coastal erosion assessment summary

The Irish Coastal Protection Strategy Study (ICPSS) includes an assessment of predicted coastal erosion rates, which is based on observed shorelines, from either mapping or anecdotal sources. A suite of maps have been prepared and they can be referenced from <http://www.opw.ie/en/floodriskmanagement/floodanderosionmapping/icpss/>

A generic methodology to assess the potential for future coastal erosion was considered for the Western CFRAM study. Extending the observed rates as was undertaken for the ICPSS does not consider underlying geology and coastal processes. The Bruun Rule is typically applied in these situations, which is the first and best known model relating shoreline retreat to an increase in local sea level is that proposed by Per Bruun (1962). The Intergovernmental Panel on Climate Change reports that 1 cm rise in sea level erodes beaches about 1 m horizontally. This becomes a large issue for developed beaches that are less than 5 m from the ocean (IPCC, 1998).

There are acknowledged limitations in the application of the Bruun Rule, and it is considered applicable to small scale local sites. Over long stretches of coast, the Bruun rule and associated cross-shore transport models become complex. There has been a number of critiques e.g. Cooper and Pilkey (2004). It is limited to a sediment based cross shore process. For the coastal AFAs included in the Western CFRAM study area the Bruun rule is not applicable.

Instead a risk based approach has been applied to determine the likelihood of future erosion based on the historical erosion rates available, the prevalence of the wave climate and the observed shoreline material in each case.

Four risk categories have been defined for the analysis as follows:

- Low Risk – Active management of the shoreline is likely to protect key assets.
- Moderate Low Risk – Tide dominated environment. No observed erosion over the last 30 years, however no active management of the shoreline is likely.
- Moderate High Risk - Wave dominated environment. No observed erosion over the last 30 years, however no active management of the shoreline is likely.
- High Risk – Erosion has been observed over the last 30 years and there is no active management of the shoreline expected in this location to prevent further erosion.

Within UoM 30-31 Galway was classified as low risk for manmade reaches and, high risk for remaining sites and Roundstone was classified as low risk for manmade reaches and moderate low risk for remaining sites..

The findings of the screening assessment should be used to inform further investigations, with the focus on those AFAs where there is moderate high or high risk of coastal erosion. Within UoM 30-31 this consists of Galway only. It is noted there is no immediate risk of coastal erosion impacts on property in Galway.

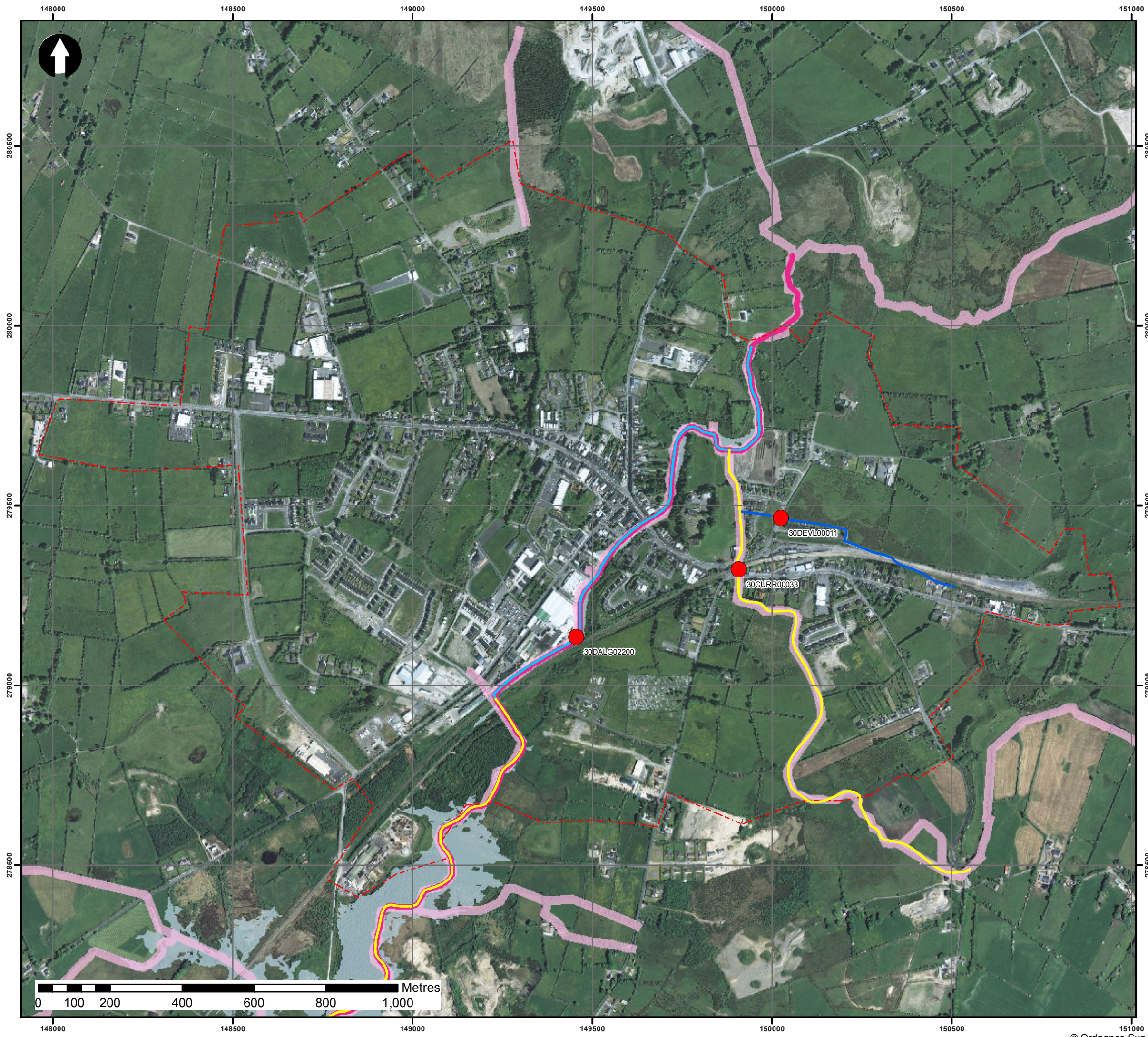
The level of detail required in these further investigations is beyond the scope of the WCFRAM study. It is recommended the modelling undertaken for the ICPSS be used as a base dataset to extend the assessment of coastal erosion potential. The long shore currents and the sediment transport can be calculated using the ICPSS flow and sediment transport modules. This will provide an indication of changes in shoreline vulnerability, through changes in radiation stresses, during the climate change scenarios; particularly when matched against the previously identified high risk coastal erosion sites.

Further work should be undertaken to extend beyond the ICPSS assumptions, such as calibration of the wave model against recorded/observed wave data, and concurrent wind data, for an extended period (at minimum including a winter period). Without calibration data the best that can be achieved is a reality check on the propagation of the waves applied at the boundaries. It is therefore a priority that the availability of calibration data be reviewed and additional data collection be undertaken if required.

Where there is the potential erosion to impact on an AFA it is recommended that a bespoke model is produced, not tied into the ICPSS licencing restrictions, and a local assessment of nearshore currents and sediment processes undertaken.

Appendices

A Hydromorphology maps



Legend
Phase 1 Findings
User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.
Sediment characteristic
Coarse sediment reach
Fine sediment reach
Sediment type unknown- Proceed to Phase II
No sediment issues identified from Phase I
Source Condition
Stable or no upstream watercourse
Incised but stable
Incised valley bottom with reworking
Extensive valley side inputs
Phase 2 Findings
High Priority Structures
Structures for Monitoring Only
Supporting Information
Structures with Scour
AFA Boundary
WFD Annex IV Sites
WFD Annex IV Sites
WFD Annex IV Sites
National Heritage Areas
Proposed Natural Heritage Areas
Special Area of Conservation
Special Protection Areas
Arterial drainage schemes
0.1% AEP Fluvial Extent

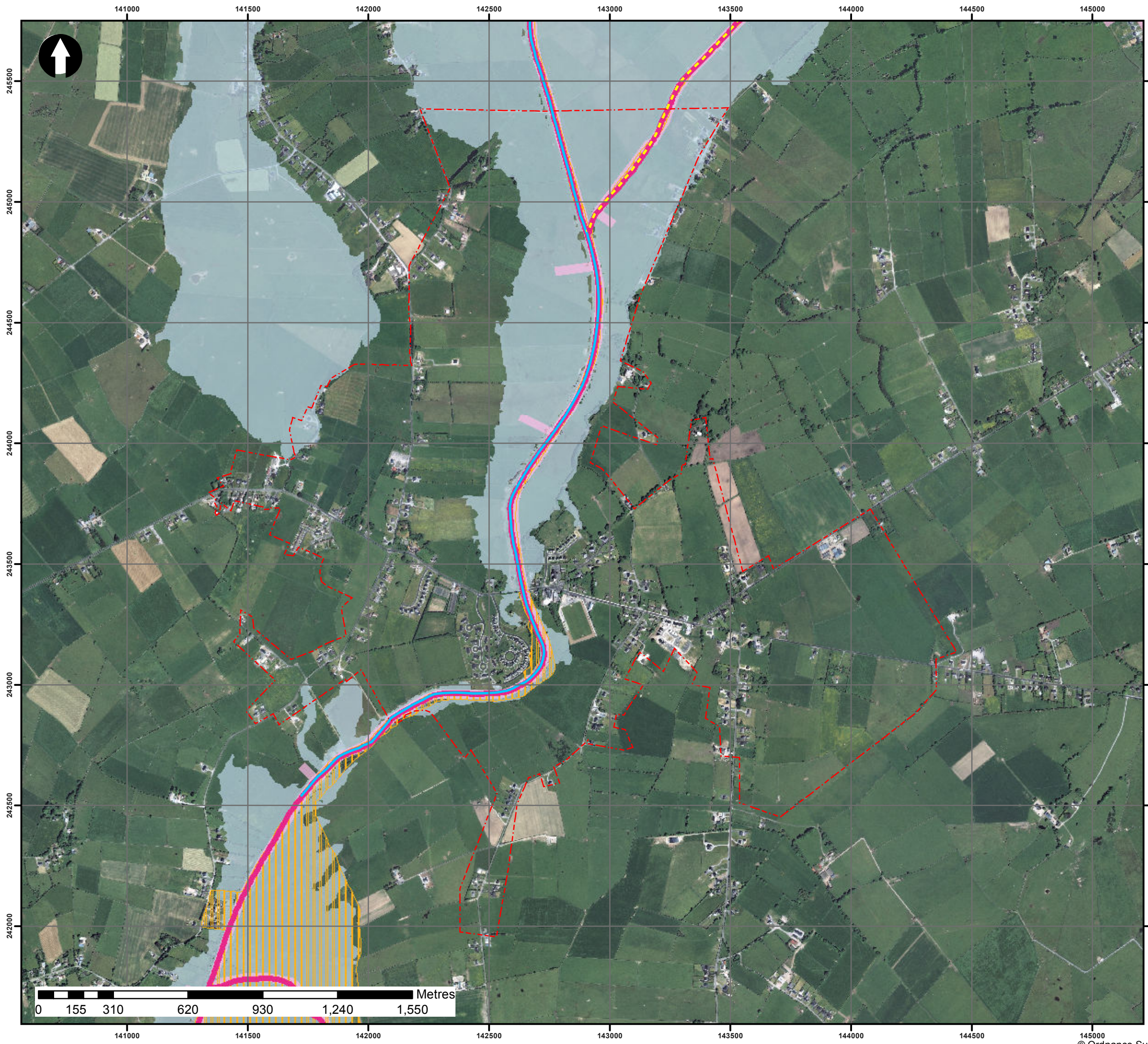
Important user note:
The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.

OPW
The Office of Public Works
Oifig na nOibreacha Poiblí
The Office of Public Works
Jonathan Swift Street
Trim
Co. Meath

JBA consulting
JBA Consulting
24 Grove Island
Corbally
Limerick, Ireland

WESTERN CFram STUDY
CATCHMENT FLOOD RISK ASSESSMENT AND MANAGEMENT

Map: Ballyhaunis	
Map Type: Hydromorphology Assement Results	
Map Area: HPW	
Drawn by: SW Date: Apr 2015	Scale: 1:10,000
Checked by: JNC Date: Apr 2015	
Approved by: SW Date: Apr 2015	Original @ A3
Map No: W30BLH_Hydromorphology_D_001	



Legend
Phase 1 Findings
User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.
Sediment characteristic
Coarse sediment reach
Fine sediment reach
Sediment type unknown- Proceed to Phase II
No sediment issues identified from Phase I
Source Condition
Stable or no upstream watercourse
Incised but stable
Incised valley bottom with reworking
Extensive valley side inputs
Phase 2 Findings
High Priority Structures
Structures for Monitoring Only
Supporting Information
Structures with Scour
AFA Boundary
WFD Annex IV Sites
WFD Annex IV Sites
WFD Annex IV Sites
National Heritage Areas
Proposed Natural Heritage Areas
Special Area of Conservation
Special Protection Areas
Arterial drainage schemes
0.1% AEP Fluvial Extent

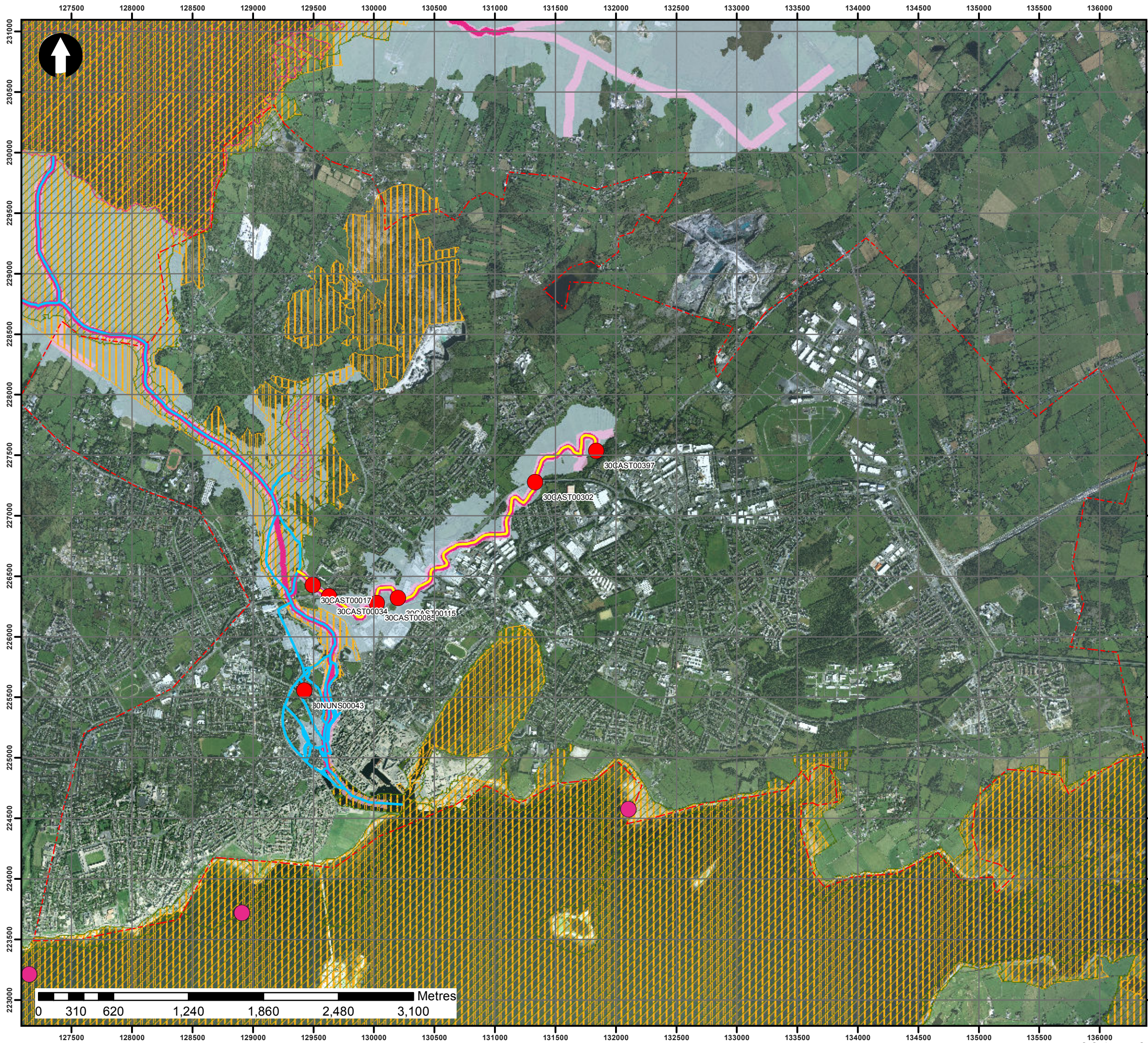
Important user note:
The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.

OPW
The Office of Public Works
Oifig na nOibreacha Poiblí

JBA
consulting

WESTERN
CFRAM
STUDY
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

Map: Corrofin	
Map Type: Hydromorphology Assement Results	
Map Area: HPW	
Drawn by: SW Date: Apr 2015	Scale: 1:15,000 <small>Original @ A3</small>
Checked by: JNC Date: Apr 2015	
Approved by: SW Date: Apr 2015	
Map No: W30CRF_Hydromorphology_D_001	



Legend

Phase 1 Findings

User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.

Sediment characteristic

- Coarse sediment reach
- Fine sediment reach
- Sediment type unknown- Proceed to Phase II
- No sediment issues identified from Phase I

Source Condition

- Stable or no upstream watercourse
- Incised but stable
- Incised valley bottom with reworking
- Extensive valley side inputs

Phase 2 Findings

- High Priority Structures
- Structures for Monitoring Only

Supporting Information

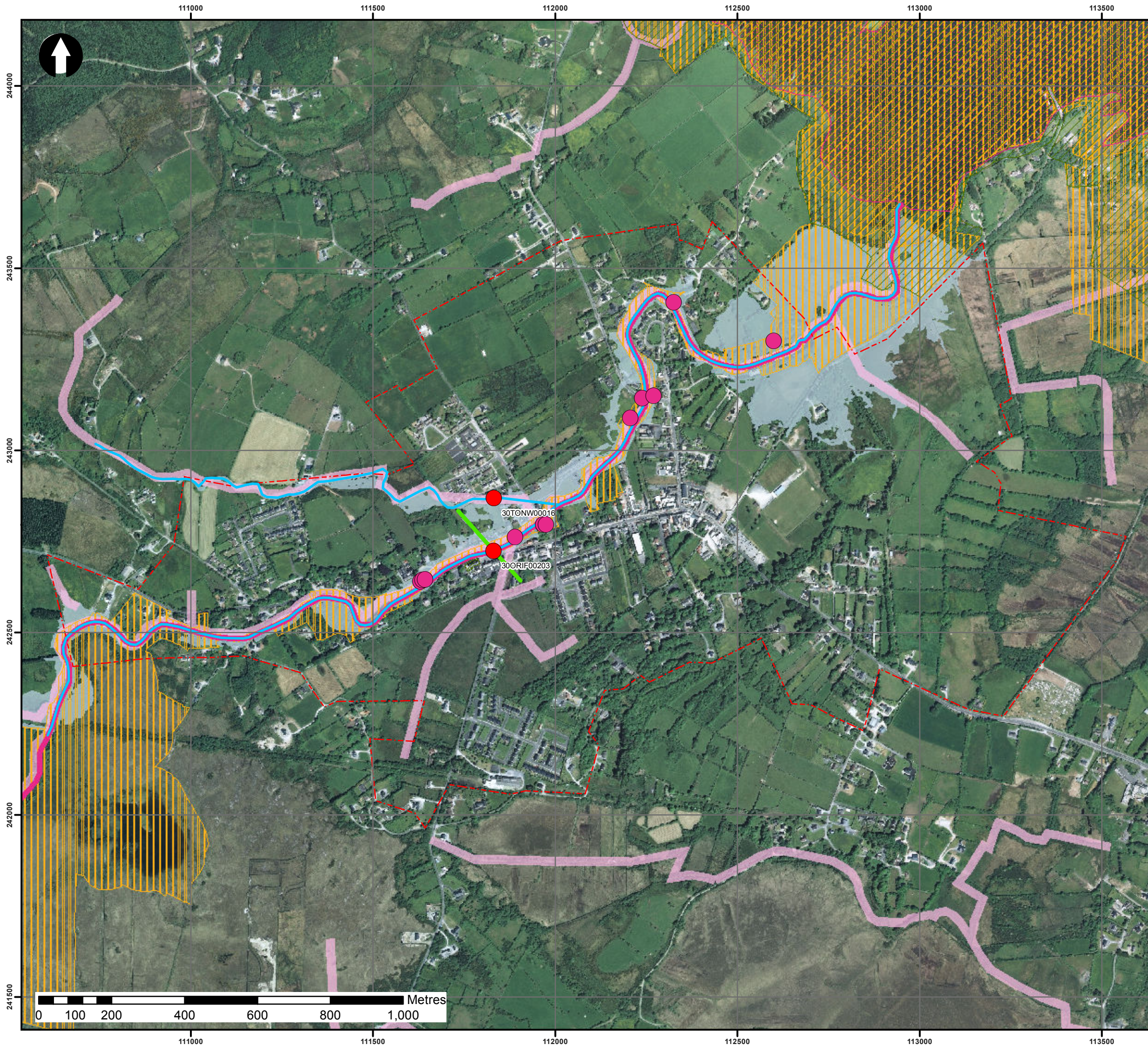
- Structures with Scour
- AFA Boundary
- WFD Annex IV Sites
- WFD Annex IV Sites
- WFD Annex IV Sites
- National Heritage Areas
- Proposed Natural Heritage Areas
- Special Area of Conservation
- Special Protection Areas
- Arterial drainage schemes
- 0.1% AEP Fluvial Extent

Important user note:

The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.



Map: Galway City	
Map Type: Hydromorphology Assement Results	
Map Area: HPW	Scale: 1:30,000 Original @ A3
Drawn by: SW Date: Apr 2015	
Checked by: JNC Date: Apr 2015	
Approved by: SW Date: Apr 2015	
Map No: W30GLW_Hydromorphology_D_001	



Legend
Phase 1 Findings
User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.
Sediment characteristic
Coarse sediment reach
Fine sediment reach
Sediment type unknown- Proceed to Phase II
No sediment issues identified from Phase I
Source Condition
Stable or no upstream watercourse
Incised but stable
Incised valley bottom with reworking
Extensive valley side inputs
Phase 2 Findings
High Priority Structures
Structures for Monitoring Only
Supporting Information
Structures with Scour
AFA Boundary
WFD Annex IV Sites
WFD Annex IV Sites
WFD Annex IV Sites
National Heritage Areas
Proposed Natural Heritage Areas
Special Area of Conservation
Special Protection Areas
Arterial drainage schemes
0.1% AEP Fluvial Extent

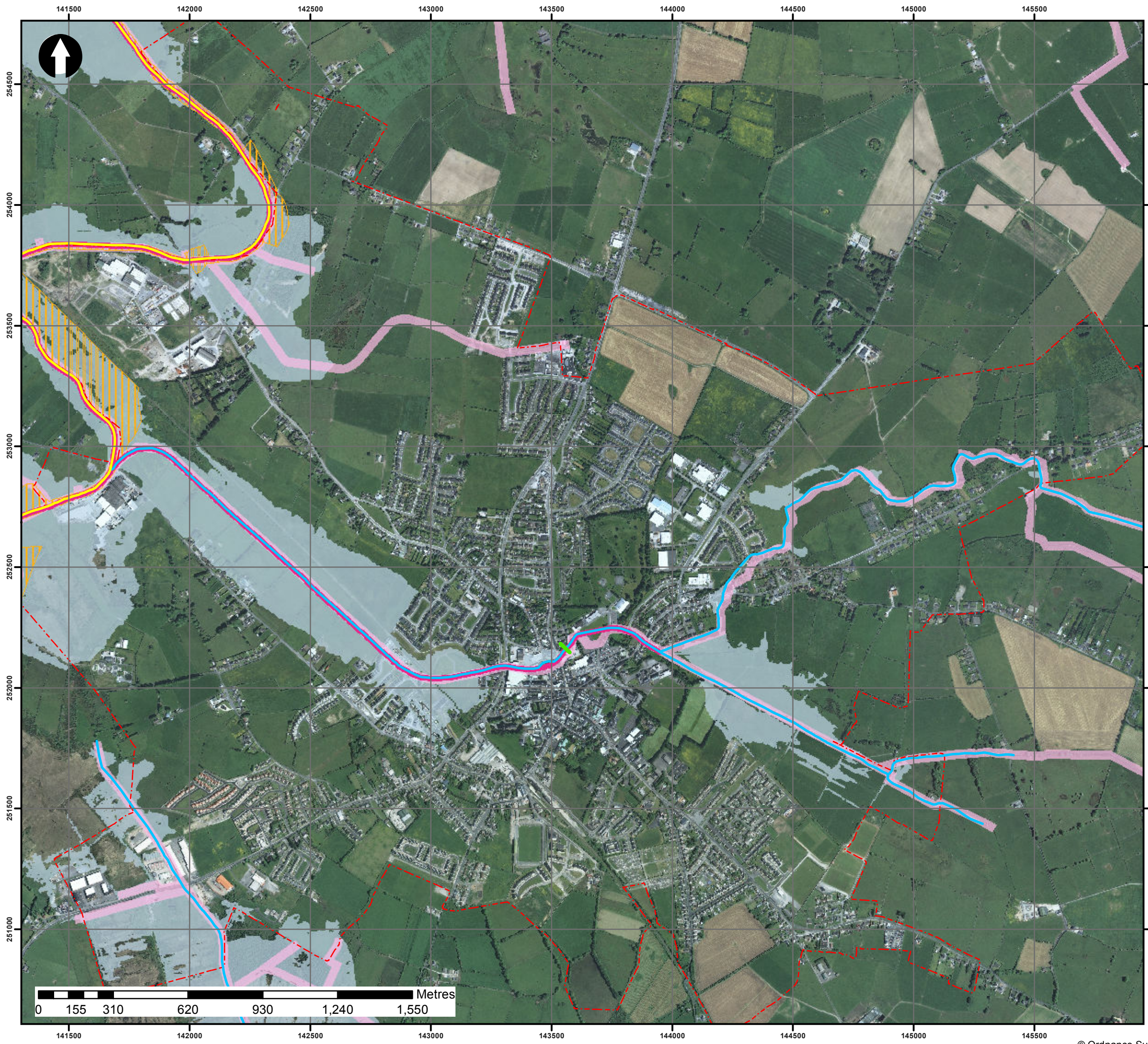
Important user note:
The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.

OPW
The Office of Public Works
Oifig na nOibreacha Poiblí
The Office of Public Works
Jonathan Swift Street
Trim
Co. Meath

JBA
consulting
JBA Consulting
24 Grove Island
Corbally
Limerick, Ireland

WESTERN CFram STUDY
CATCHMENT FLOOD RISK ASSESSMENT AND MANAGEMENT

Map: Oughterard	
Map Type: Hydromorphology Assement Results	
Map Area: HPW	
Drawn by: SW Date: Apr 2015	Scale: 1:10,000 Original @ A3
Checked by: JNC Date: Apr 2015	
Approved by: SW Date: Apr 2015	
Map No: W30OTD_Hydromorphology_D_001	



Legend
Phase 1 Findings
User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.

Sediment characteristic
Coarse sediment reach
Fine sediment reach
Sediment type unknown- Proceed to Phase II
No sediment issues identified from Phase I

Source Condition
Stable or no upstream watercourse
Incised but stable
Incised valley bottom with reworking
Extensive valley side inputs

Phase 2 Findings
High Priority Structures
Structures for Monitoring Only

Supporting Information
Structures with Scour
AFA Boundary
WFD Annex IV Sites
WFD Annex IV Sites
WFD Annex IV Sites
National Heritage Areas
Proposed Natural Heritage Areas
Special Area of Conservation
Special Protection Areas
Arterial drainage schemes
0.1% AEP Fluvial Extent


Important user note:
The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.



The Office of Public Works
Jonathan Swift Street
Trim
Co. Meath

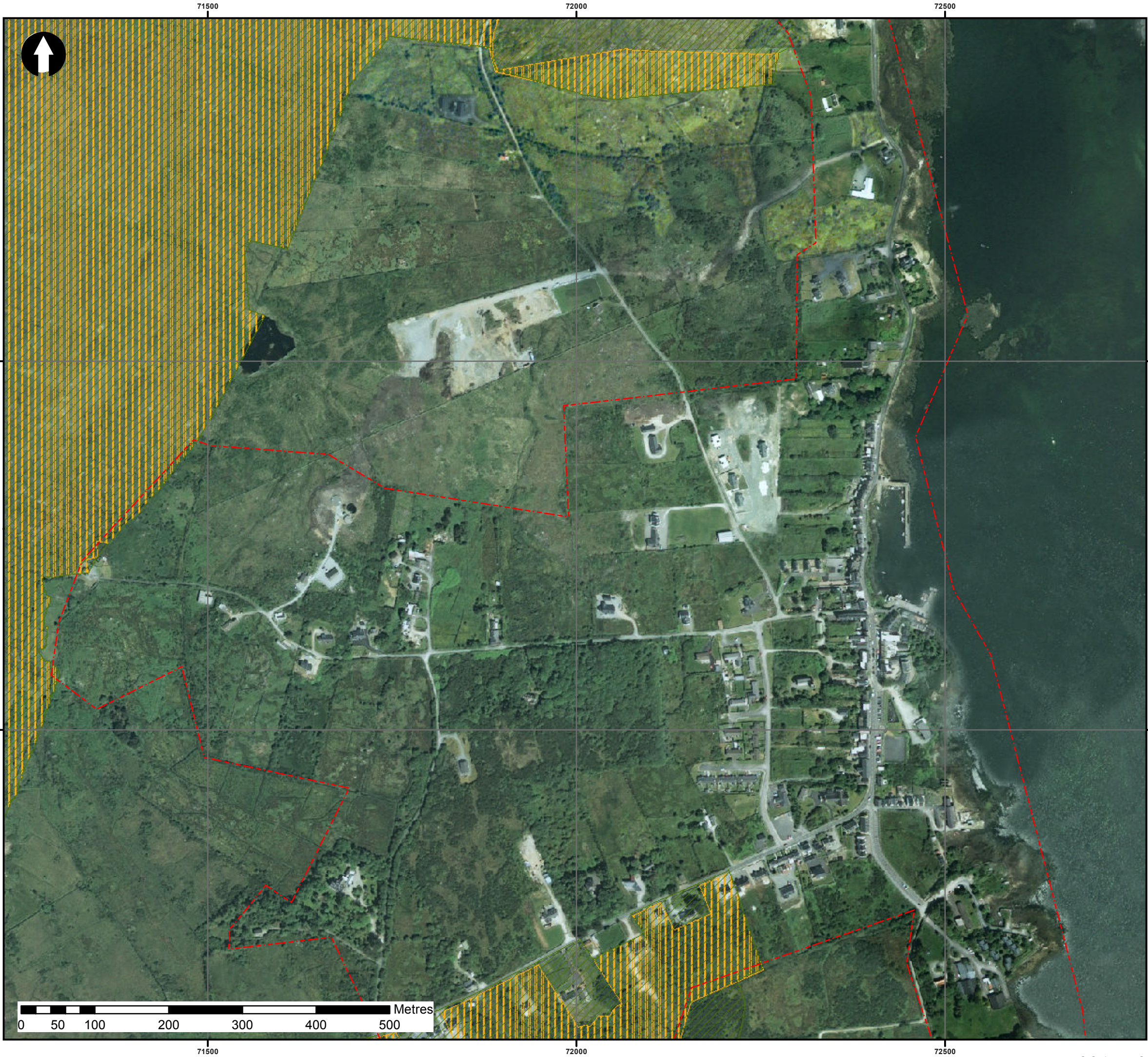


JBA consulting
24 Grove Island
Corbally
Limerick, Ireland



WESTERN
CFRAM
STUDY
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

Map: Tuam	
Map Type: Hydromorphology Assement Results	
Map Area: HPW	Scale: 1:15,000 Original @ A3
Drawn by: SW Date: Apr 2015	
Checked by: JNC Date: Apr 2015	
Approved by: SW Date: Apr 2015	
Map No: W30TUM_Hydromorphology_D_001	



Legend
Phase 1 Findings
User Note: Phase 1 findings include both the Sediment Characteristic and the Source Condition and as such are presented through both the line colour and style.

Sediment characteristic
Coarse sediment reach
Fine sediment reach
Sediment type unknown- Proceed to Phase II
No sediment issues identified from Phase I

Source Condition
Stable or no upstream watercourse
Incised but stable
Incised valley bottom with reworking
Extensive valley side inputs

Phase 2 Findings
High Priority Structures
Structures for Monitoring Only

Supporting Information
Structures with Scour
AFA Boundary
WFD Annex IV Sites
WFD Annex IV Sites
WFD Annex IV Sites
National Heritage Areas
Proposed Natural Heritage Areas
Special Area of Conservation
Special Protection Areas
Arterial drainage schemes
0.1% AEP Fluvial Extent


Important user note:
The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.



The Office of Public Works
Jonathan Swift Street
Trim
Co. Meath



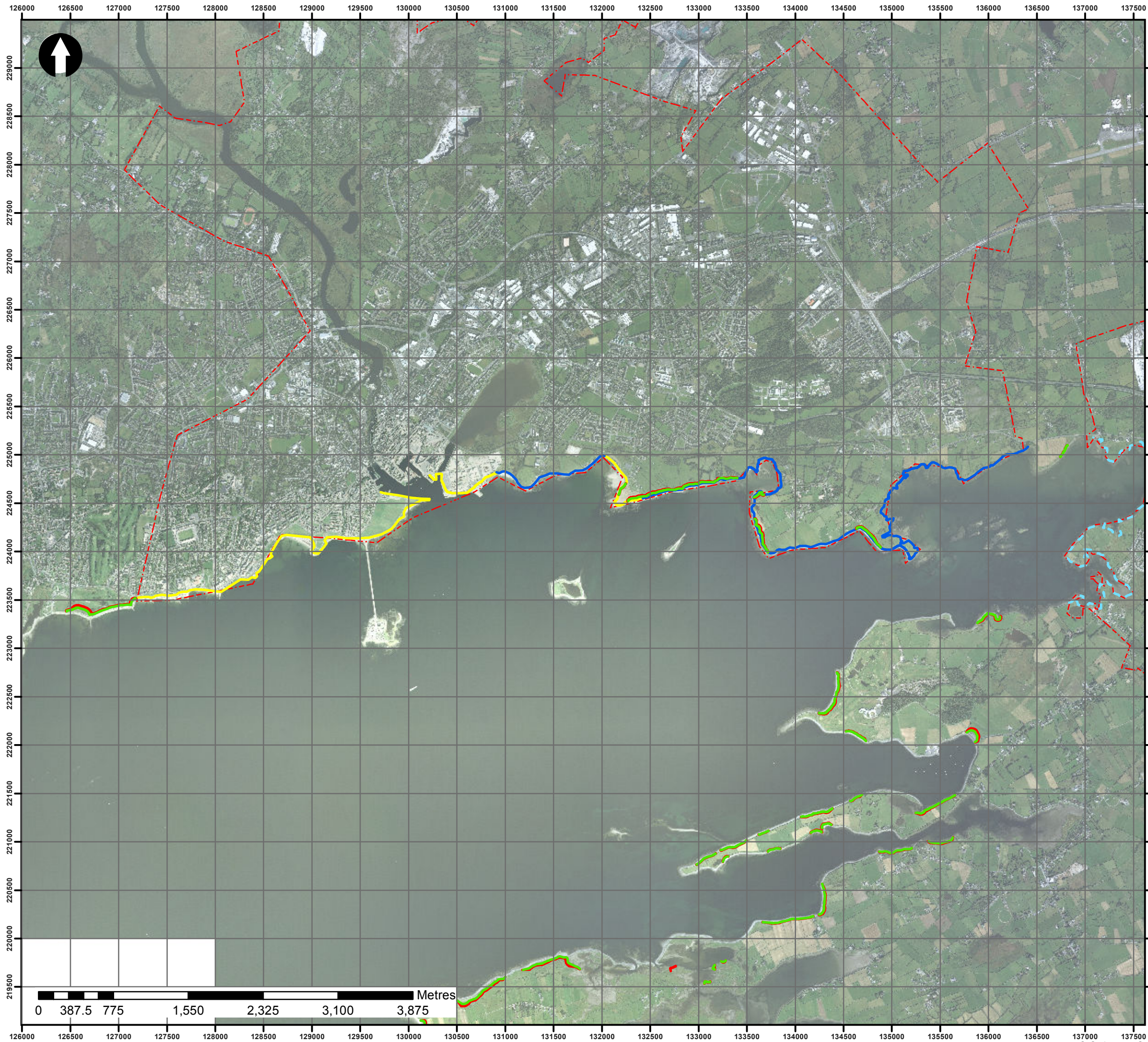
JBA consulting
24 Grove Island
Corbally
Limerick, Ireland



WESTERN
CFRAM
STUDY
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

Map: Roundstone	
Map Type: Hydromorphology Assement Results	
Map Area: HPW	Scale: 1:5,000 Original @ A3
Drawn by: SW Date: Apr 2015	
Checked by: JNC Date: Apr 2015	
Approved by: SW Date: Apr 2015	
Map No: W31RSN_Hydromorphology_D_001	

B Coastal erosion maps



Legend


- ICPSS Erosion Line Yr 2030
- ICPSS Erosion Line Yr 2050

Coastal Erosion Risk Classification


- Low Risk
- Moderate Low Risk
- Moderate High Risk
- High Risk
- AFA_Boundary

Important user note:


The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.



The Office of Public Works
Jonathan Swift Street
Trim
Co. Meath

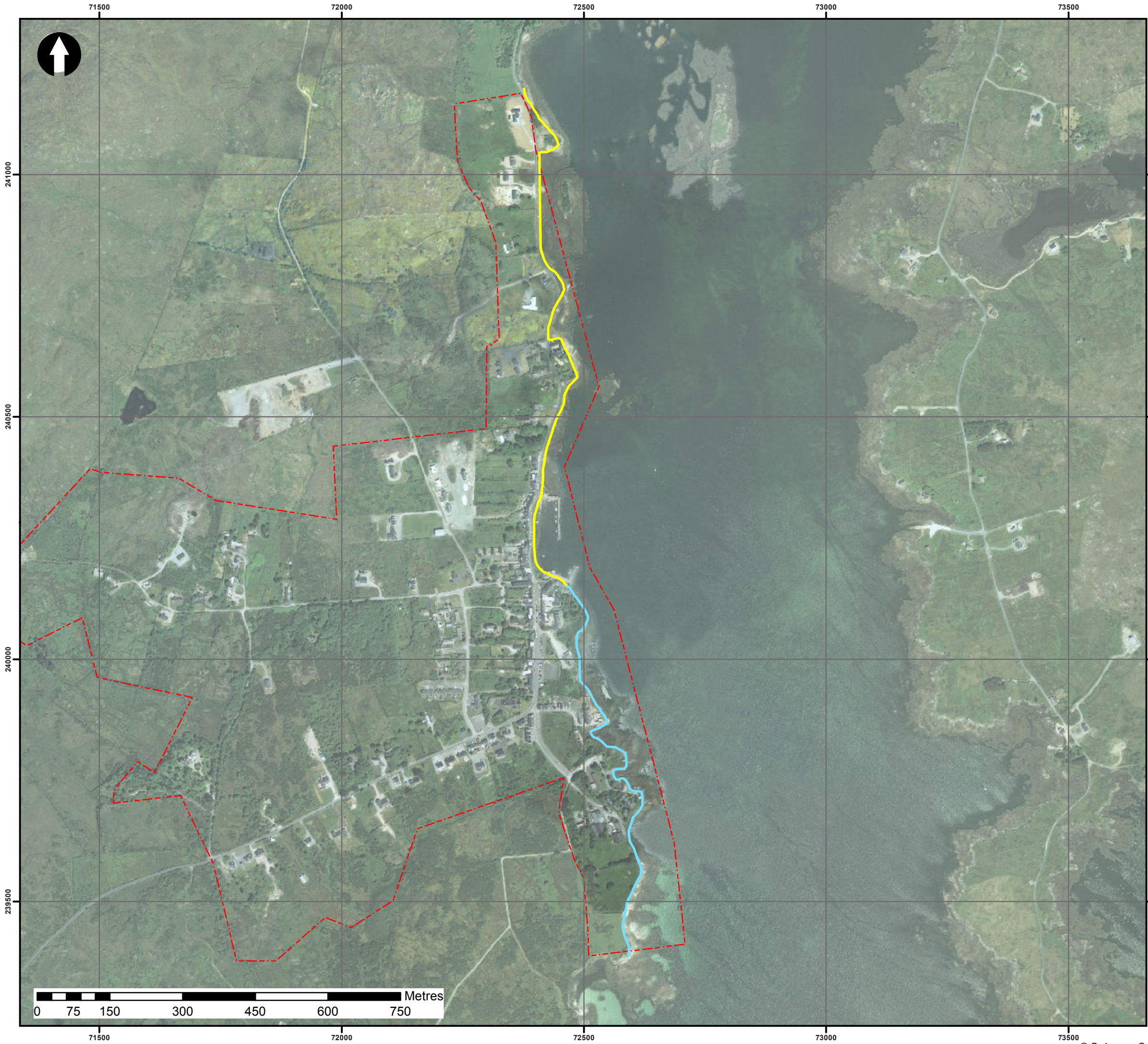


JBA consulting
JBA Consulting
24 Grove Island
Corbally
Limerick, Ireland



WESTERN
CFRAM
STUDY
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

Map: Galway City	
Map Type: ICPSS Coastal Erosion Results	
Map Area: HPW	Scale: 1:37,500 Original @ A3
Drawn by: SW Date: Mar 2015	
Checked by: ER Date: Mar 2015	
Approved by: JNC Date: Mar 2015	
Map No: W30GLY_Coastal_Erosion_D_001	



Legend

- ICPSS Erosion Line Yr 2030
- ICPSS Erosion Line Yr 2050

Coastal Erosion Risk Classification

- Low Risk
- Moderate Low Risk
- Moderate High Risk
- High Risk
- AFA_Boundary

Important user note:
The viewer of this map should refer to the disclaimer guidance notes and conditions of use that accompany this map.
This draft map is for consultation purpose only and should not be used for any other purpose.



The Office of Public Works
Jonathan Swift Street
Trim
Co. Meath



JBA consulting
JBA Consulting
24 Grove Island
Corbally
Limerick, Ireland



WESTERN
CFRAM
STUDY
CATCHMENT FLOOD RISK
ASSESSMENT AND MANAGEMENT

Map: Roundstone	
Map Type: ICPSS Coastal Erosion Results	
Map Area: HPW	Scale: 1:7,500 Original @ A3
Drawn by: SW Date: Mar 2015	
Checked by: ER Date: Mar 2015	
Approved by: JNC Date: Mar 2015	
Map No: W31RSN_Coastal_Erosion_D_001	



Registered Office

**24 Grove Island
Corbally
Limerick
Ireland**

T: +353 (0) 61 345463
e: info@jbaconsulting.com

**JBA Consulting Engineers and
Scientists Limited**

Registration number 444752

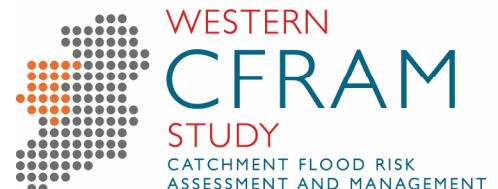
Visit our website
www.jbaconsulting.com

Western CFRAM UoM 31 - Owengowla Hydraulic Modelling Report: Volume 2a - Roundstone

Final

September 2016

**Office of Public Works
Trim
Co. Meath**



JBA Consulting

24 Grove Island
Corbally
Limerick
Ireland

JBA Project Manager

Sam Willis BSc MSc CEnv CSci MCIWEM C.WEM

Revision History

Revision ref / Date issued	Amendments	Issued to
Version 1.0 / January 2014	Initial Issue	Rosemarie Lawlor, OPW
V2.0 / January 2015	Updated to reflect model updates following review and public consultation. Report updated to reflect client and TAS review.	Rosemarie Lawlor, OPW
V3.0 / September 2016	Final updates	Clare Butler, OPW

Contract

This report describes work commissioned by The Office of Public Works, by a letter dated (28/07/11). The Office of Public Works' representative for the contract was Rosemarie Lawlor. Sam Willis, Chris Smith and Elizabeth Russell of JBA Consulting carried out this work.

Prepared by David Forde BEng MEng MIEI

Reviewed by Chris Smith BSc PhD CEnv MCIWEM C.WEM
MCMI

Purpose

This document has been prepared as a draft report for the Office of Public Works. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to the Office of Public Works.

Copyright

Copyright – Copyright is with Office of Public Works. All rights reserved. No part of this report may be copied or reproduced by any means without the prior written permission of the Office of Public works.

Legal Disclaimer

This report is subject to the limitations and warranties contained in the contract between the commissioning party (Office of Public Works) and JBA.

Carbon Footprint

A printed copy of the main text in this document will result in a carbon footprint of 124g if 100% post-consumer recycled paper is used and 157g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to achieve carbon neutrality.

Contents

1	Introduction	1
1.1	Scope of report	1
1.2	Model and report overview	1
1.3	Coastal domain overview.....	1
1.4	Available data	4
2	Hydraulic modelling	6
2.1	Overview	6
2.2	Defences and walls.....	6
2.3	Floodplain	7
2.4	Wave overtopping.....	8
3	Flood history, model calibration and sensibility checking	9
3.1	Calibration versus sensibility checking	9
3.2	Flood history	9
3.3	Calibration outcomes.....	10
4	Tidal boundaries	13
4.1	Model scenarios.....	13
5	Model Results	14
5.1	Model runs	14
5.2	Flood risk mapping	14
5.3	Key flood risk mechanisms	14
6	Sensitivity testing	15
6.1	Screening of sensitivity tests	15
6.2	Sensitivity testing results and uncertainty bounds.....	17
7	Model limitations	18
7.1	Gap in quay wall	18

List of Figures

Figure 1-1: Properties at risk in Roundstone village.....	2
Figure 1-2: Primary school exposed to extreme tide levels.....	3
Figure 1-3: Roundstone AFA catchment overview	3
Figure 1-4: Defence survey crest levels	4
Figure 2-1: 2D model tidal boundary	6
Figure 2-2: Stability patch locations in Roundstone	8
Figure 3-1: Gap in quay wall adjacent to houses flooded during 3rd January flood event	10
Figure 3-2: January 2014, estimated as design T100, modelled flood extent for northern part of AFA	11
Figure 3-3: January 2014, estimated as design T100, modelled flood extent for southern part of AFA	11
Figure 4-1: Selection of tide levels input into model.....	13
Figure 6-1: 0.5% AEP event uncertainty bounds - Roundstone AFA.....	17

List of Tables

Table 1-1: Extreme Sea Levels (mOD Malin).....	5
Table 2-1: Informal effective defences	7
Table 3-1: Summary of flood history.....	9
Table 4-1: Extreme sea levels (mOD) - current and future scenarios.....	13
Table 6-1: Sensitivity test summary.....	15
Table 6-2: Floodplain roughness range.....	15

Abbreviations

AEP	Annual exceedence probability
AFA	Area for further assessment
AMAX	Annual maximum
CFRAM	Catchment flood risk assessment and management
DAD	Defence asset database
DAS	Defence asset survey
DEM	Digital elevation model (Includes surfaces of structures, vegetation, etc)
DTM	Digital terrain model ('bare earth' model; does not include surfaces of structures, vegetation, etc
ESTRY	One-dimensional model from the TUFLOW suite
FRISM	Flood risk metrics (a flood risk tool developed by JBA)
FRMP	Flood risk management plan
FRR	Flood risk review
FSR	Flood studies report
FSU	Flood studies update
GIS	Geographical information system
HEFS	High-end future scenario
HEP	Hydrological estimation point
HPW	High priority watercourse
HWA	Hydrograph width analysis
IBIDEM	Interactive bridge invoking the design event method
ICPSS	Irish coastal protection strategy study
ISIS	One-dimensional hydraulic modelling software
LA	Local authority
LIDAR	Light detection and ranging
mOD	Metres above Ordnance datum (unless stated this refers to the Malin datum)
MPW	Medium priority watercourse
MRFS	Mid-range future scenario
NDHM	National digital height model (a DTM by Intermap)
OSi	Ordnance Survey Ireland
PFRA	Preliminary flood risk assessment
Q(T)	Flow for a given return period
QMED	Median annual flood, used in FSU methods
SAAR	Standard annual average rainfall
SoP	Standard of protection (in relation to flood defences)
T	Return period, inverse of AEP
Tp	Time to peak
TUFLOW	Two-dimensional hydraulic modelling software

UoM Unit of Management

* Asterisks at the end of a cross section label denotes interpolated
model cross sections

1 Introduction

1.1 Scope of report

This report summarises the hydraulic modelling work for the Roundstone AFA coastal hydraulic model. This document is specific to the AFA itself and should be read in conjunction with the various reports detailed in Section 1.2 for details on the modelling approaches and wider context of the study.

The report covers the overall hydraulic modelling process from model build through to the development of design runs with the aim of providing a detailed understanding of the hydraulic controls and flood mechanisms identified throughout the study.

The report is not a user manual for the hydraulic model itself, full details of which are provided in the model handover check files accompanying the hydraulic model.

The hydraulic modelling work summarised in this and the Unit of Management (UoM) 31 Hydraulic Modelling Report, of which this report is an Annex, forms one element of the Western Catchment-based Flood Risk Assessment and Management (WCFRAM) study process. The process to date has included amongst other tasks a Flood Risk Review (FRR)¹, a project inception stage², a Strategic Environmental Assessment (SEA)³ and the development of the catchment hydrology⁴. Where the work completed in these tasks contains information relevant to the analysis discussed in this document, references have been included directing the reader to the relevant report for further background information.

1.2 Model and report overview

There is one hydraulic model within the Roundstone suite; this is the coastal model. A wave overtopping assessment was not undertaken, as per the conclusions of the Irish Coastal Wave and Water Level Modelling Study (ICWWS)⁵. Instead, an assessment of the impact of still water levels with a surge component has been carried out.

The model code relevant to this river is:

- Roundstone Coastal - C4

Reports which are relevant to this AFA are:

- Western CFRAM Flood Risk Review Report
- Western CFRAM UoM 31 Inception Report
- Western CFRAM UoM 31 Hydrology Report
- Western CFRAM UoM 31 Hydraulic Modelling Report
- Western CFRAM UoM 31 Hydraulic Modelling Report: Volume 1 - Hydraulic Modelling Method Statement
- Western CFRAM UoM 31 Hydraulic Modelling Report: Volume 3 - Flood Risk maps
- Roundstone AFA Hydraulic Model Check File

1.3 Coastal domain overview

Roundstone has a coastline of approximately 1.8km within the boundaries of the AFA. The key areas of flood risk within the AFA are properties in the centre of the village, particularly adjacent to the pier. Houses to the north of the main village pier, including the local library tend to be

¹ JBA Consulting (2012), Western CFRAM Flood Risk Review, Final Report, Office of Public Works

² JBA Consulting (2012), Western CFRAM Unit of Management 29 - Galway Bay South East Inception Report, Final Report, Office of Public Works

³ JBA Consulting (2013), Western River Basin District Catchment-based Flood Risk Assessment and Management (CFRAM) Strategic Environmental Assessment, Scoping Report, Office of Public Works

⁴ JBA Consulting (2014), Western CFRAM Unit of Management 29 - Galway Bay South East Hydrology Report, Final Report, Office of Public Works

⁵ RPS Consulting Engineers. (2012) Irish Coastal Wave and Water Level Modelling Study (ICWWS)

particularly affected during extreme events. The figure below shows some of the properties that are potentially at risk of tidal ingress.

Figure 1-1: Properties at risk in Roundstone village



Another area of potential risk is a primary school in the southern extents of the village. There is a wall at the front of the school but it may not form a sufficient barrier to flood water entering the site, especially when wave action compounds extreme water levels.

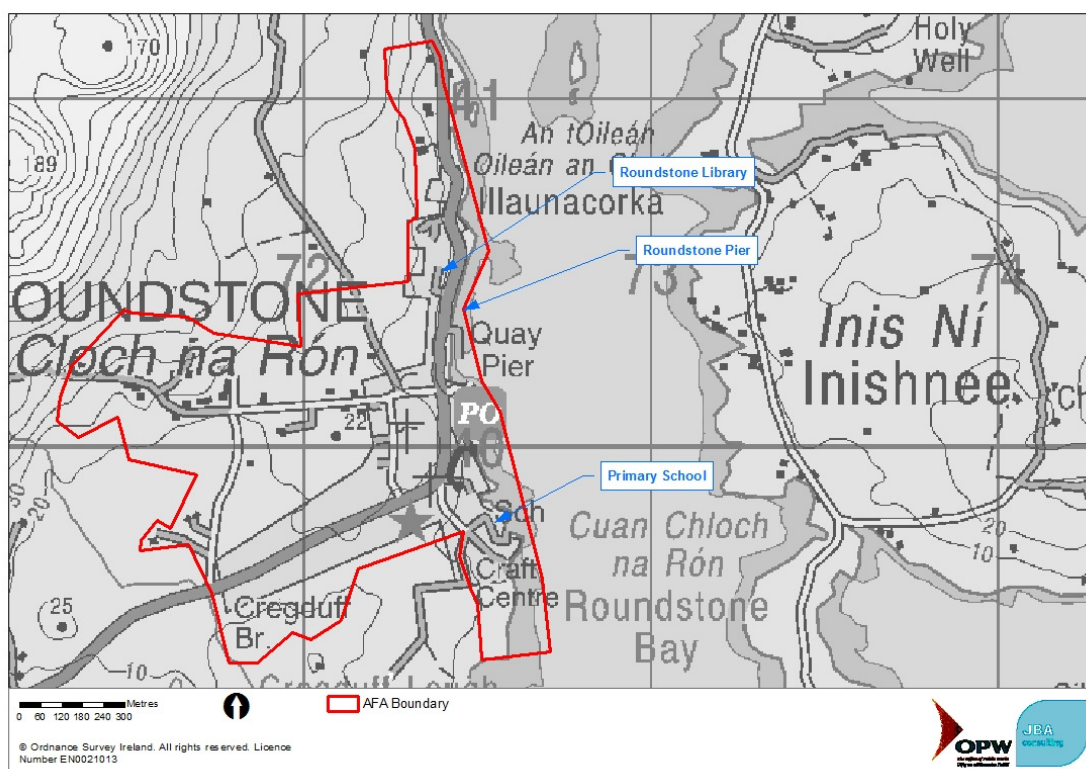
There is a quay wall that extends north from the pier along the R341 to the outskirts of the village. The ground elevation along the pier is approximately 3.00mOD Malin. The elevation increases to approximately 5.00mOD on the R341 behind the quay wall. The quay wall provides some protection to seaward properties. However, the wall is of old, masonry construction and has some gaps in it. Its structural integrity could be compromised if subjected to extreme water levels. The wall's maximum crest level is 5.04mOD Malin, at the corner of the pier, before falling to a minimum of 3.19mOD Malin approximately 100m to the north. There are no bridges or culverts within the AFA that will impact upon the tidal flooding extents.

Figure 1-2: Primary school exposed to extreme tide levels



The frequency of flooding is relatively high within Roundstone. It can be as frequent as once or twice a year for the R341 road. Flooding within the village is due to a combination of high tides and storm surge, exacerbated by wave action. Floor levels have been raised in properties adjacent to the southern jetty in the centre of the village. Critically, the same cannot be said of the properties directly behind the quay wall in the north of the village.

Figure 1-3: Roundstone AFA catchment overview

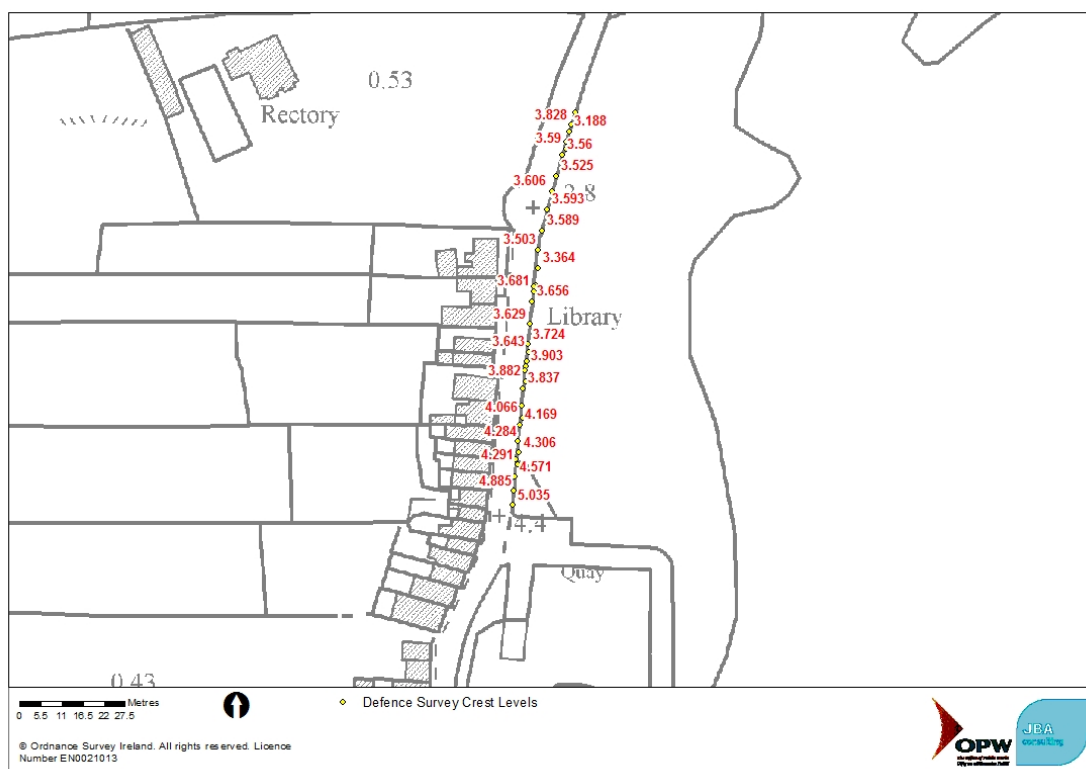


1.4 Available data

1.4.1 Survey data

Topographic survey of the quay wall was undertaken by CCS Surveying in Work Package 3 as part of the National Survey Contract No. 6 and delivered in February 2013. The completed defence survey is shown in Figure 1-4.

Figure 1-4: Defence survey crest levels



LIDAR data has been collected for use in the model. Data has been provided in both filtered and unfiltered formats in a 2m grid resolution. The LIDAR was flown between November 2011 and August 2012. It was not possible to verify the accuracy of the LIDAR as there was no ground survey carried out in this AFA. The LIDAR was, however, inspected for any anomalies.

Changes following completion of the baseline survey

Following the completion of the baseline survey feedback was received from the local authority that following damage incurred during a flood event the quay wall had been rebuilt and road levels raised. Survey of this improved structure was collected by Mayo County Council and supplied in February 2016.

Figure 1-5: New quay wall



The effect of this structure has been to reduce the risk of flooding to properties behind this quay wall from the 5% AEP event to the 0.1% AEP event. Owing to the later date of the survey for this structure, this hydraulic model report has not been revised to reflect this change on the ground. The structure is however incorporated into the final flood maps.

1.4.2 Tide data

The term extreme still water sea-level refers to the level that the sea is expected to reach during a storm event of a particular AEP due to a high tide and the passage of a storm surge. The extreme sea level tidal graphs were developed using Highest Astronomical Tide (HAT) tidal data for Galway Bay. An appropriate surge profile was then applied which increases sea levels above the tidal levels. This was done for a variety of return periods. The extreme sea levels were informed by the Irish Coastal Protection Strategy Study (ICPSS) deliverables. Full details of the procedure are provided in the WCFRAM Hydrology Report for UoM 31. A summary of the tide levels applicable to Roundstone is provided in Table 1-1.

Table 1-1: Extreme Sea Levels (mOD Malin)

AEP	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Sea Level	2.81	2.96	3.07	3.18	3.33	3.43	3.54	3.79

There is no tide gauge in the vicinity of the AFA with which the hydraulic model could be calibrated. The nearest tidal gauges are the Rossaveel Pier gauge in Rossaveel (straight-line distance of approximately 28km to Roundstone) and the Inishmore gauge on the Aran Islands (straight-line distance of approximately 36km to Roundstone). They are operated by the Office of Public Works and Galway County Council respectively. Interpolations were carried out between these gauges to generate levels which could be used for model calibration events.

2 Hydraulic modelling

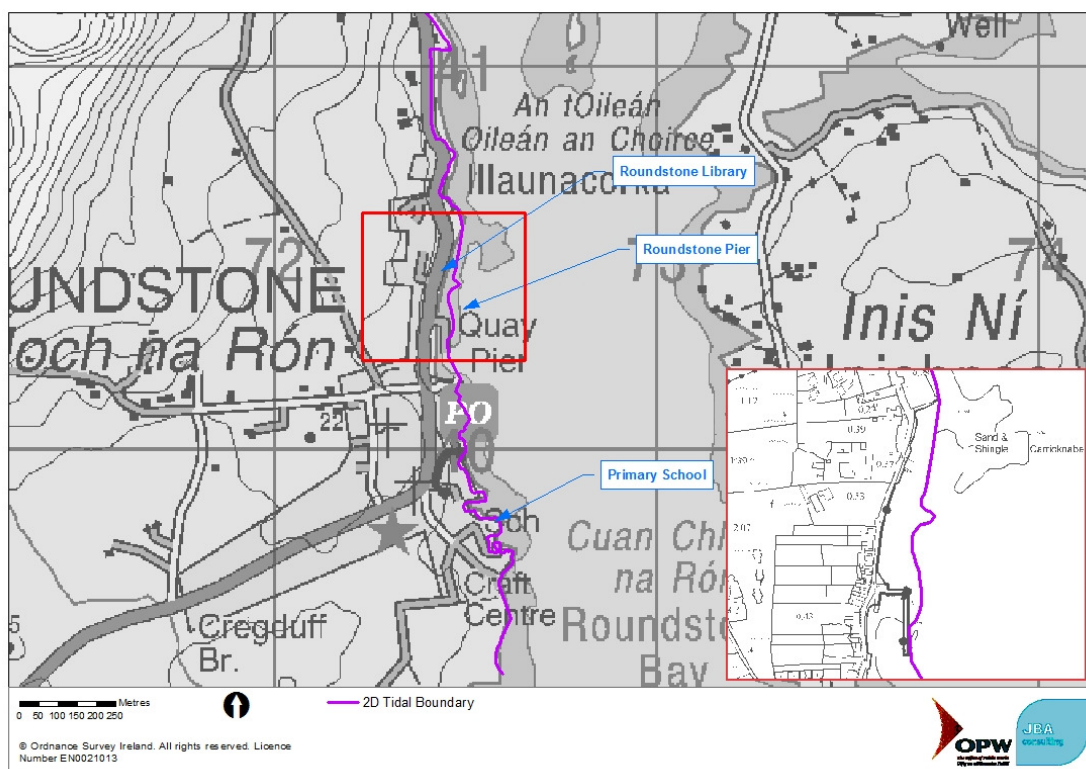
2.1 Overview

This section should be read in conjunction with the Hydraulic Model Report: Volume 1a: Hydraulic Modelling Method Statement and the Roundstone Hydraulic Model Check File. The Method Statement provides an overview of the elements of the 2D model construction and the following section of the report describes how they were applied to the Roundstone AFA.

A 2D TUFLOW model has been constructed, incorporating the coastline identified in Figure 2-1: The model has been given the ID code C4.

The tidal boundary has generally followed the same path as the coastline described in the OSi 5k mapping. However, in some areas the boundary had to be adjusted to maintain stability in the model, particularly where the inlets and bays were complex. This involved placing the boundary further out to sea in some locations; this will not affect model results. An example of where the model boundary does not follow the coastline is shown in Figure 2-1.

Figure 2-1: 2D model tidal boundary




2.2 Defences and walls

2.2.1 Defences

There are no formal (i.e. OPW, local authority or privately maintained defences) in Roundstone. Informal effective defences identified within the AFA are detailed in Table 2-1. These structures have been modelled as surveyed and are assumed to retain flood waters to the crest of the structure.

Table 2-1: Informal effective defences

Description and Location	Photograph
<p>Quay Wall</p> <p>This informal structure runs from the northern village boundary to the pier in the centre of the village. It is an old, masonry wall with an average crest height of 3.85mOD.</p> <p>There is a surveyed crest level approximately every 5m along the wall in the TUFLOW model. As the cell size in the model is 2m, there would be an acceptable confidence in interpolation between these points.</p> <p>There is a 1m wide gap in the wall, just at the northern boundary of the village. The gap was probably produced by the eroding action of breaking waves at some point in the past. This gap reduces the effectiveness of the wall as a defence. The wall will be included in the model as it will alter the path of the water in this portion of the AFA.</p> <p>The elevation at this point is the same as the road levels of 2.9m OD. The gap has been modelled by breaking the defence line on either side and lowering the LIDAR locally to road level.</p>	

2.2.2 Walls

Informal ineffective structures are structures that are not assumed to function as flood defences and are either bypassed in the model or have been removed to allow flooding beyond them, such as garden walls. However, within Roundstone AFA, there are no structures of this type to account for in the model.

2.3 Floodplain

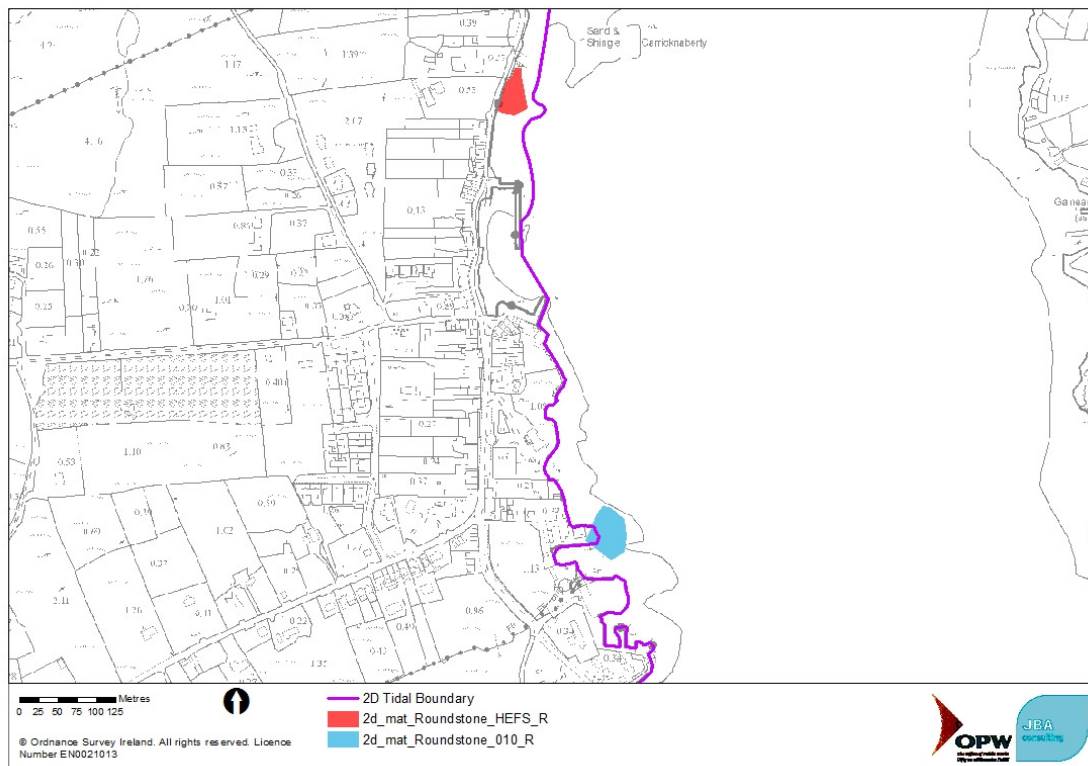
A 2D cell size of 2m has been used for the majority of the model runs, as it was deemed to provide the greatest level of detail, matching the resolution of the LIDAR, but still resulting in manageable model run times. However, to simulate the high end future climate change scenario (HEFS) the model was run with a 4m cell size and the gap in the quay wall was closed. This was to attain stability in the model run at such significant tide levels. The overall flood extent was not affected by such a change as the peak of the HEFS events significantly exceeds the crest level of the quay wall for all return periods.

The active model area was determined using the LIDAR DTM for the AFA. Areas of high ground were deemed 'natural boundaries' and serve well as model extents.

Roughness values have been assigned to the floodplain using the values detailed in the Modelling Method Statement.

In addition to the standard ground cover categories, such as roads and houses, stability patches have been used in two locations. There is a stability patch in the southern portion of the AFA that is used for all model scenarios. This is to ease the passage of water over a relatively high piece of headland. Another stability patch is used in the northern extent of the AFA for the 0.1% AEP MRFS, 0.5% HEFS and 0.1% HEFS model runs. This is necessary due to the velocity of flow overtopping the defence on the R341. The locations of both stability patches are shown in Figure 2-2.

Figure 2-2: Stability patch locations in Roundstone



2.4 Wave overtopping

Roundstone has not been included in the list of AFAs to be considered for wave overtopping analysis as part of the Western CFRAM project. A screening assessment was undertaken as part of the ICWWS⁶ study, and Roundstone was not found to be particularly vulnerable to wave overtopping. Instead, the flood extents modelled using extreme still water levels (tide and surge only) are likely to be representative of the extent and level of wave overtopping.

⁶ RPS Consulting Engineers. (2012) Irish Coastal Wave and Water Level Modelling Study (ICWWS)

3 Flood history, model calibration and sensibility checking

3.1 Calibration versus sensibility checking

Where a recording tide gauge is located in or near the site and this data is accompanied by historical data from a flood event (such as flood extents, or spot levels), then it is possible to undertake calibration of the model. This process would involve running the recorded tide levels through the model and changing model parameters, such as Manning's 'n', to match the flood extents or levels that were observed. Ideally, a second event would then be run through the model and used to validate the outputs. While it is possible to simulate levels recorded at a gauge in the model, without any record of the impact of the event, the model cannot be calibrated and the checking process is limited to a confirmation that predicted extents match expectations based on topography and local knowledge. If there is no gauge data available but there are historical records of flooding, then the predicted extent from an appropriate design event with a similar exceedance probability to the historical flood event can be used as a sensibility check of the predicted flooding frequency.

3.2 Flood history

Key flood risk areas have been identified in the Flood Risk Review and Inception Reports. Table 3-1 shows a summary of historical flood events, and includes a note on whether they have been used to calibrate or validate the model.

Table 3-1: Summary of flood history

Area affected	Main flood mechanisms	Recorded flood event date	Use in model check
Roundstone Quay	High tides	January 2014	Sensibility checking of model
Roundstone Quay	High tides	2012	Background information
Roundstone Quay	High tides	Recurring	Corroborates 2014 validation run
R341 (North AFA)	High tides	Recurring	Corroborates 2014 validation run
Monastery Road	High tides	Recurring	Corroborates 2014 validation run

The largest recorded event on record at the time of modelling occurred on the 15th of December 2012. The event had a peak tide level of 2.98mOD. Based upon the design tide curves derived from the ICPSS deliverables, this peak level is approximately equivalent to a 20% AEP event. This may have been recently surpassed by the event of 3rd January 2014, which had an approximate peak tide level of 3.56mOD at the Galway Port tide gauge. However, this tide level or return period cannot be directly correlated to Roundstone.

3.2.1 January 2014 event

The event in question coincided with the high tide on the morning of the 3rd of January and affected a proportion of the Roundstone AFA. It was a short duration event, with very little evidence of the flood impacts being visible upon JBA's site visit on the 9th of January.

The flooding was from a combination of high tide, storm surge and waves breaking over the quay wall. Water also exploited a significant gap in the quay wall, as shown in Figure 3-1:. It is not known when the sandbags were positioned in the gap or whether they were effective. Flood water receded extremely quickly upon the turning of the tide.

As a result of the site visit and discussion with residents, it was determined that the R341 road and four properties north of the pier in the centre of the village were flooded at approximately 05:30 of the morning in question. The properties consisted of three private residences and the public library. The white house, farthest from the pier, was the most severely impacted with water coming in both the front and back doors of the house. The house owner recalled that the highest water

level was reached simultaneously with the peak of the high tide. The R341 road was reported as being impassable for the duration of the event. The local national school principal described that a large pool of water had collected adjacent to the school, but no water had made its way into the building.

There is no photographic evidence of the flooding but approximate flood extents for the 3rd of January event were produced based on anecdotal evidence and are included in Figure 3-2 and Figure 3-3.

Figure 3-1: Gap in quay wall adjacent to houses flooded during 3rd January flood event



3.3 Calibration outcomes

3.3.1 January 2014 event

As there is no nearby tide gauge, measured tide levels could not be compared with the flood extents observed for the 3rd January event. However, lower return period events were input into the model and compared against the observed extents to validate the overall model performance. Essentially, calibration of this model became a 'ground-truthing' exercise to ensure that no unrealistic flow paths or depths were occurring.

It was found that the 1% AEP event (T100) outline most closely resembled the extent recorded, although given the limitations of the modelling and the recording of the event, it would be incorrect to report the event as having a 1% AEP.

The results of the hydraulic modelling have been combined with the evidence gathered by JBA's 9th of January site visit to construct the composite flood extent maps in Figure 3-2 and Figure 3-3.

It seems clear that the flow paths and flood extents created using the hydraulic model resemble those described to the JBA inspection team for the 3rd of January event. The model allows water to flow through the gap in the quay wall and pond in front of the adjacent properties, as witnessed in reality.

There is a discrepancy between the modelled and observed flood depths. The model is not giving the same magnitude of inundation in the properties in the north of the AFA. However, upon further

reflection it is most likely that this has been caused by the combined actions of storm surge and waves breaking over the quay wall. This wave breaking (or overtopping) was also corroborated by testimony of local residents, but has not been included in the modelling for the reasons given in Section 2.4. It is also likely that the choice of the 1% AEP return period, whilst being a reasonable representation, was a slight underestimation of the actual event.

Figure 3-2: January 2014, estimated as design T100, modelled flood extent for northern part of AFA

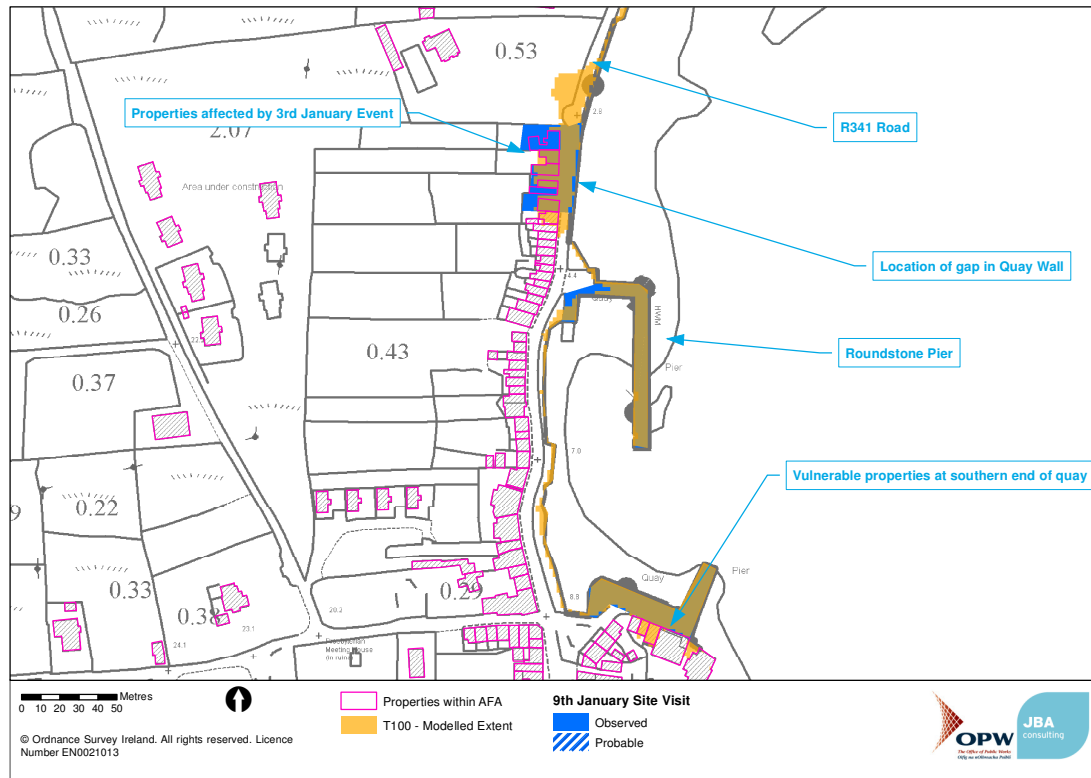
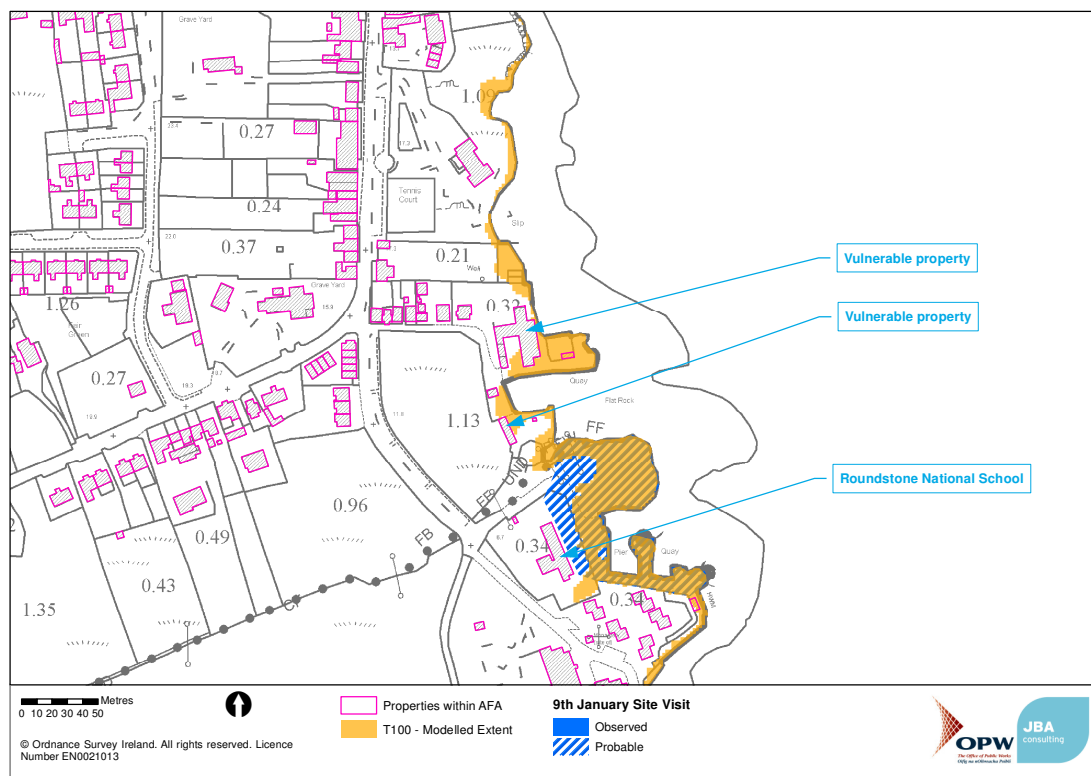


Figure 3-3: January 2014, estimated as design T100, modelled flood extent for southern part of AFA



When comparing the observed and modelled extents, it is evident that flood water ponds in similar locations. It must also be noted that the peak of the event occurred in the early hours of the morning and does not have a reliably recorded peak water level so cannot be compared directly to the 1% AEP model extents shown in the previous figures.

3.4 Stakeholder Engagement

Throughout this process OPW regional engineers and local authority engineers have been consulted in the form of steering group, progress group and engineer meetings and their input has feed into the flood maps.

Public Consultation Day (PCD) - 15th of October 2014

On October 15th 2014 a public consultation was held at the Roundstone Community Hall to present the flood maps for the town and solicit comments and feedback.

This PCD was attended by 11 people. At the PCD attendees were invited to leave feedback, in the form of a questionnaire. The questionnaire sought feedback on resident's knowledge of flooding in the town including the locations of flooding and the frequency of flooding.

Table 3-2 outlines the feedback received at the day relevant to the study and a note regarding how this information has been accommodated by the study.

Table 3-2: PCD Feedback

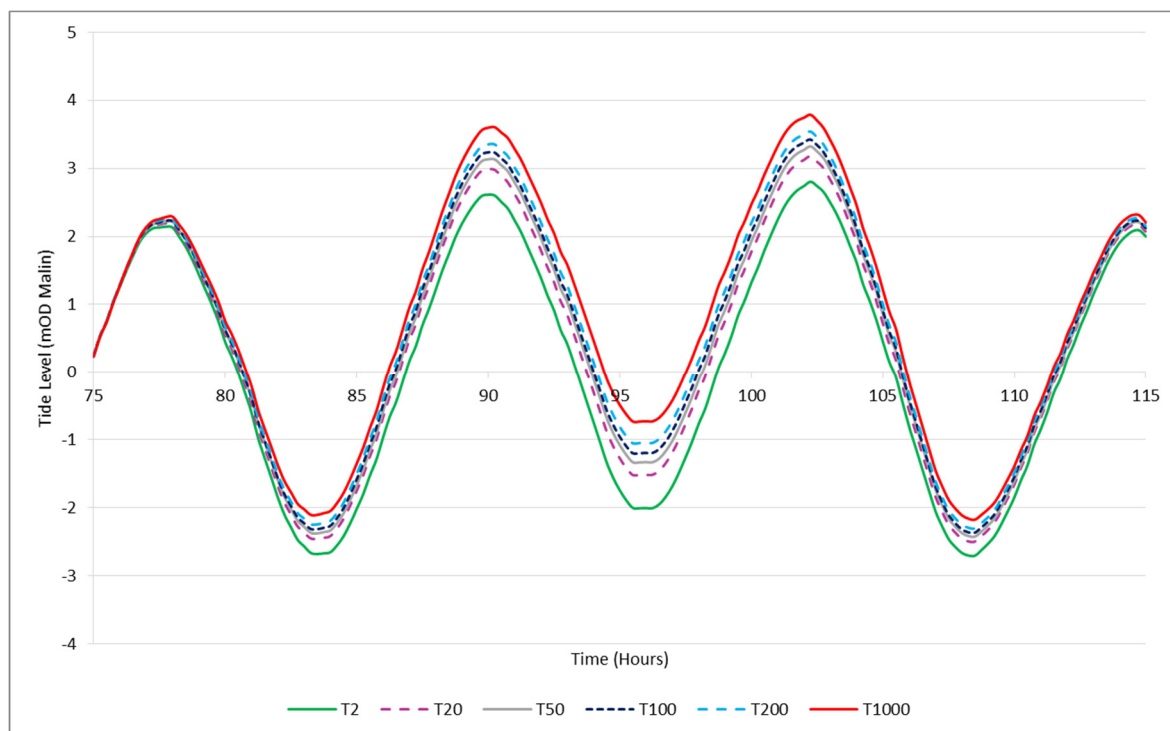
Comments Received	Study Response
A resident living on the quay confirmed depths match likely flood risk. The property has flooded to a depth of approximately 10 inches with 1 ft. of flooding on the quay.	The report provides validation of the predicted flood risk for Roundstone.

4 Tidal boundaries

4.1 Model scenarios

Flood extents have been produced for the 50%, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP design event probabilities. An extract of the tide data used to derive these extents have been illustrated in Figure 4-1.

Figure 4-1: Selection of tide levels input into model



4.1.1 Future scenarios

Future scenarios have been developed as part of the hydrological analysis and are described in detail in the WCFRAM Hydrology Report for UoM 31.

With regard to coastal flooding, accounting for future scenarios requires the addition of 500mm and 1000mm respectively for the Mid-Range Future Scenario (MRFS) and High End Future Scenario (HEFS) on the existing tide profiles for each exceedance probability. The extreme sea levels for future scenarios are detailed in Table 4-1.

Table 4-1: Extreme sea levels (mOD) - current and future scenarios

Return Period (Years)	2	5	10	20	50	100	200	1000
AEP	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Current Scenario	2.81	2.96	3.07	3.18	3.33	3.43	3.54	3.79
MRFS	3.31	3.46	3.57	3.68	3.83	3.93	4.04	4.29
HEFS	-	-	4.07	-	-	-	4.54	4.79

5 Model Results

5.1 Model runs

The model has been run for a present day and two future scenarios, a Mid-Range Future Scenario (MRFS) and a High-End Future Scenario, which consider the potential impact of climate change. Further details of the allowances within the calculations are included in the Hydrology Report, but the increased flows include for the impacts of urbanisation and climate change.

The model has been run for the following present day and MRFS tidal events; 50%, 10%, 5%, 2%, 1%, 0.5% and 0.1% AEP design events. Only the 10%, 1% and 0.1% AEP design events have been run for the HEFS.

5.2 Flood risk mapping

Flood risk extents for the present day and MRFS 10%, 1% and 0.1% AEP design events along with long section profiles for present day 10, 1% and 0.1% AEP design events are presented in Volume 3 of the UoM 29 Hydraulic Modelling Report.

5.3 Key flood risk mechanisms

Further to the information presented in the flood risk maps, a brief description of the key flood risk sites and flooding mechanisms is provided below.

5.3.1 Flooding at Roundstone pier and quay wall

The model shows that flooding within the village begins with flow coming up through the gap in the quay wall onto to the R341 road. The northern extent of the village is at particular risk and this has been confirmed by the flooding on the 3rd of January 2014. In the 0.1% AEP event, there are approximately 11 different properties at risk. The wall in front of these properties is also bypassed from the north.

5.3.2 Flooding at Roundstone National School

Although the Roundstone National School was not seriously affected in January 2014, risk is likely to be more significant in larger events and particularly in the MRFS and HEFS scenarios. Risk in this location is solely dictated by elevated water levels.

6 Sensitivity testing

6.1 Screening of sensitivity tests

The suite of potential sensitivity tests is detailed in Volume 1a: Hydraulic Modelling Method Statement. The application of the sensitivity tests has been an iterative process which allowed certain criteria to be screened out. Table 6-1 summarises the full suite of potential sensitivity tests, and highlights those which are not applicable, and those which have been screened out. Further details of these criteria are provided in the following sections.

The results of testing those criteria which are relevant to Roundstone are detailed in Section 6.2.

Table 6-1: Sensitivity test summary

Sensitivity test	Relevance to Roundstone
Peak flow / tide level	Tested (by means of MRFS and HEFS)
Flow volume	Not applicable
Critical storm duration	Not applicable
Roughness	Tested
Building representation	Tested
Afflux / headloss at key structures	Not applicable
Water level boundaries and joint probability	Not applicable
Timing of tributaries	Not applicable
Timing of fluvial and tidal peaks	Not applicable
Cell size	Not applicable (model is already 2m resolution)
Additional tests	Gap in quay wall

6.1.1 Peak tide level

The effect of rising sea levels is being investigated through the climate change scenarios and the change quoted for the MRFS (i.e. Current +0.5m) has been used to test sensitivity to sea levels so has not been explicitly included in this sensitivity testing.

6.1.2 Roughness

The limited flood extents in the existing-risk design events mean there will be likely little benefit to testing the sensitivity of the model results to a reduction in floodplain roughness (NTF) values. However, sensitivity to both lower and upper bound roughness values for the 10% and 0.5% AEP events, as shown in Table 6-2, has been tested.

Table 6-2: Floodplain roughness range

Floodplain Material	Roughness Values (Manning's n)		
	Typical Value	Lower Bound Value	Upper Bound Value
General Natural Surfaces	0.040	0.030	0.050
Buildings	0.300	0.100	1.000
Roads, Tracks and Paths	0.015	0.013	0.017
Rock	0.050	0.040	0.070

6.1.3 Building representation

The current flood extents for the 0.5% AEP event show approximately sixteen properties at risk of inundation, including those adjacent to the gap in the quay wall. It may be of some benefit to add a threshold level of 300mm to the affected buildings and investigate the resultant flood extents.

6.1.4 Gap in the quay wall

The gap in the key wall has been included in the model although it was photographed as being sandbagged after the January 2014 event. The impact of closing this gap was tested by removing the gap from the model and assuming that the defence along the R341 was an effective structure.

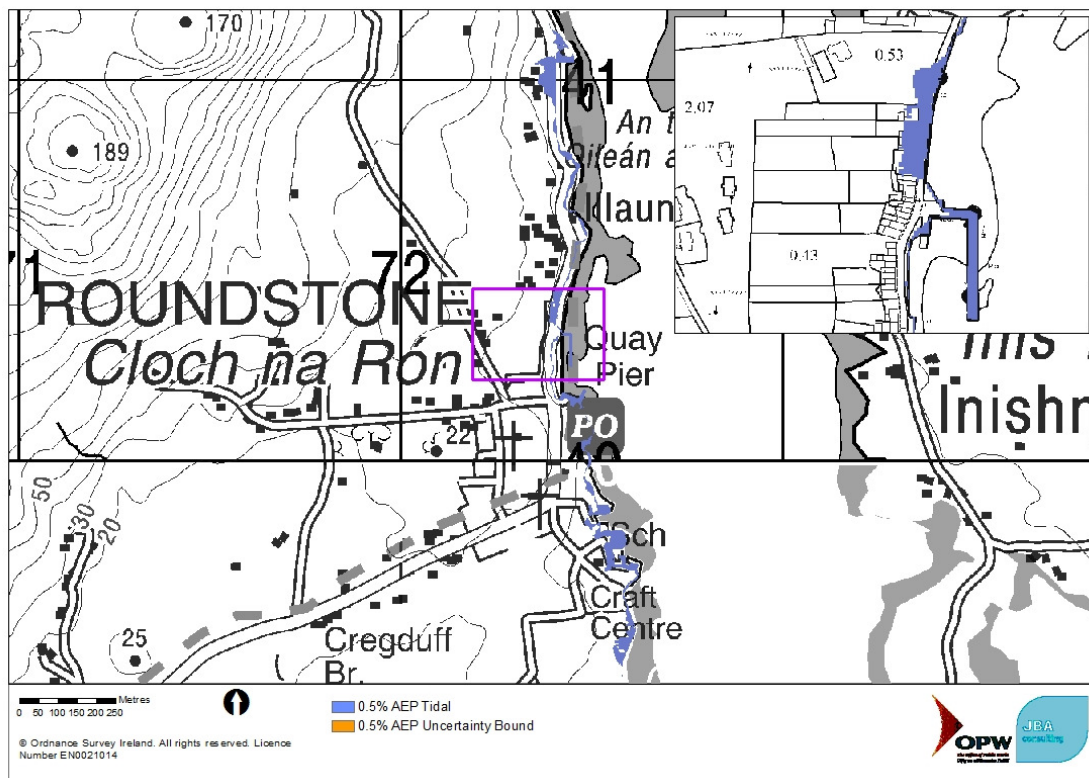
6.2 Sensitivity testing results and uncertainty bounds

The results of the sensitivity tests have been used to inform the uncertainty bounds as detailed in the Hydraulic Modelling Method Statement. The uncertainty bound in effect presents the most sensitive hydraulic parameter/s as assessed within the bounds identified in Section 6.1 at all locations within the AFA (note, this excludes the impact of climate change on the tide levels).

To simplify the presentation of the sensitivity tests, the uncertainty bound for the 0.5% AEP event has been presented only. Where different parameters have contributed to the development of the uncertainty bound, these are highlighted on the map and in the adjoining text.

It can be seen from Figure 6-1 that the model is not sensitive to the adjustment of the various parameters identified in Section 6.1. Indeed, there is no increase documented on the design flood extents. Essentially, the model simply runs the design water level to a particular contour on the DTM. This process does not change no matter what parameter is tested. Closing of the gap in the quay wall simply transfers the point at which flow makes land further north along the R341.

Figure 6-1: 0.5% AEP event uncertainty bounds - Roundstone AFA



7 Model limitations

7.1 Gap in quay wall

The gap in the quay wall, as previously noted, has been modelled as a 2m wide opening in the quay wall with an elevation value of 2.9mOD which matches the adjacent road level. In reality, the opening is approximately 1m wide. Initially, when trying to replicate this size in the hydraulic model, the cell size had to be reduced to 1mx1m. However, this caused the model to become unreasonably slow and resulted in numerous stability issues. Thus, it was decided that using a 2m cell size was a justifiable compromise and still allows the flow path to be represented at lower return period events. Given the small area that fills behind the wall, the water level across the wall will equalise at the event peak making the size of gap less significant. The impact of this change was also investigated and documented through the sensitivity testing.



Registered Office

**24 Grove Island
Corbally
Limerick
Ireland**

T: +353 (0) 61 345463
e: info@jbaconsulting.com

**JBA Consulting Engineers and
Scientists Limited**

Registration number 444752

Visit our website
www.jbaconsulting.com