Western CFRAM
Unit of management 30 - Corrib
Hydrology Report

Final report

March 2015

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Revision History

Revision Ref / Date Issued	Amendments	Issued to
Draft v1.0		OPW
Draft final v2.0 24 July 2014	Updated to reflect comments received on all draft UoM reports	OPW
Final v3.0 12 December 2014	Updated to reflect comments received from OPW	OPW
Final v4.0 25 March 2015	Updated to reflect comments received final UoM reports	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. Duncan Faulkner, Colin Riggs, Paige Garside, Lucy Barker and Kevin Haseldine of JBA Consulting carried out this work.

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Purpose

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Executive Summary

This report describes the hydrological analysis carried out as part of the Catchment-Based Flood Risk Assessment and Management Study (CFRAM) for the Western River Basin. It covers unit of management (UoM) 30, the Corrib catchment.

The brief calls for a comprehensive and detailed hydrological analysis that places particular emphasis on flood flow estimation for the main flood risk areas (termed AFAs, Areas for Further Assessment) and the watercourses that flow through these areas (termed HPWs, High Priority Watercourses). In UoM 30, the AFAs are Ballyhaunis, Tuam, Corrofin, Claregalway, Oughterard and Galway City. Galway is subject to both coastal and fluvial flood risk, while the other five AFAs are vulnerable to fluvial flood risk only. Claregalway has been investigated as part of a separate study and so is not considered any further in this report.

The principal objective of the hydrological study is to derive best estimates of design fluvial flood parameters including peak flows, hydrographs and flood volumes, for all hydrological estimation points. The study also includes derivation of design coastal flood parameters for AFAs subject to significant coastal flood risk. The word “design” here refers to a quantity that is expected to be exceeded with a specified probability or frequency, as opposed to a measured river flow or sea level for any particular date and time. Design flood parameters are estimated by statistical analysis or modelling.

The report includes a review of the hydrological data available in the study area. All AFAs benefit from the presence of nearby river gauging stations. The Hazelhill and Dangan gauges measure only river level and do not have a rating equation that enables conversion of level into flow, however in both these cases alternative flow gauges are available within the AFA. At six gauging stations in UoM 30 the rating equations have been reviewed in detail as part of this study.

A variety of methods are available for estimation of design floods. The approach taken for the Western CFRAM is to base the analysis closely on the recorded flow data, in accordance with the methods developed during the Flood Studies Update research. The implementation of the FSU research project has not yet been completed and so it has been necessary to develop software to apply some of the methods.

Peak flows have been estimated from statistical analysis of annual maximum flows. At locations without flow data, design flows have been estimated indirectly from physical properties of the catchment, combined with transfer of data from representative gauged catchments both locally and further afield throughout Ireland. For the most extreme design floods (annual probabilities below 1%), the statistical analysis has been supplemented with an extended flood growth curve from the Flood Studies Report rainfall-runoff method.

The design flows have been derived by direct analysis of flood data so they will naturally be consistent with that data. However design flows have been checked to identify any results that fall outside expected ranges; these included confirmation that growth factors are within expected ranges, that AEPs for observed events implied in the flood frequency curves are appropriate and that there was spatial consistency between design flows.

Several approaches have been trialled for the estimation of design flood hydrographs, and the results assessed using techniques such as analysis of percentage runoff and flood volumes. The recommended approach for most watercourses is to derive the shape of design hydrographs using the rainfall-runoff method from the Flood Studies Report. For some unusual catchments, particularly those containing large loughs, design hydrograph shapes are derived more directly from averaging of observed flood hydrographs.

The design flood hydrographs will form inflows to the hydraulic models that are being used to predict flood levels, depths and extents. It has been necessary to reconcile flows within the model with hydrological estimates of flow to ensure consistency through the river systems, and consider the main assumptions and sources of uncertainty in the design flows, and how these are translated into the model.

Methods used to estimate design flood hydrographs at each AFA

AFA	Name	QMED method	Growth curve method	Distribution	Hydrograph Shape
Corrofin	Clare – upstream of gauge 30004	Data Transfer – Pivotal 30007 and 30004	Pooled	General Logistic	FSR rainfall-runoff with either Tp(0) adjusted from lag analysis
	Clare – downstream of gauge 30004	Data Transfer – Pivotal 30004	Single Site - 30004	Gumbel	n/a – will be routed by model
	Grange	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
Galway City	Corrib	Data Transfer – Pivotal 30061	Pooled	General Logistic	
Oughterard	Owenriff	Data Transfer – Pivotal 30101	Pooled	General Logistic	FSR rainfall-runoff
	Tonweeroe	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
Tuam	Clare	Data Transfer – Pivotal 30007 and 30004	Pooled	General Logistic	FSR rainfall-runoff with either Tp(0) adjusted from lag analysis or Tp(0) adjusted to match HWA results
	Lough Park, Nanny, Suileen	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
Ballyhaunis	Dalgan, Devlis, Curries	Data Transfer – Pivotal 30020	Pooled	General Logistic	FSR rainfall-runoff with Tp(0) adjusted from lag analysis

As well as design flows for the present-day situation, the study has produced a set of flows for two future scenarios, which have considered climate change impacts on both river flows and sea levels and the impact of increased urbanisation. It is considered that land use change, in the form of changes to forestry practice, will have little impact on flood risk in the UoM, so this has not been accounted for.

To provide a downstream boundary condition for hydraulic models of rivers that enter the sea, design tidal graphs have been created by combining information on extreme sea levels with design surge shapes and design astronomical tide curves.

Detailed records of the calculations are provided in the appendices, along with a table of the design peak flows. The report is accompanied by digital deliverables which provide the design flows for all locations, along with further information on the methods used at each location.

The Hydrology Report for UoM 30 should be read in conjunction with the Hydraulic Modelling Report for UoM 30, and the specific modelling reports for each AFA, which detail the application of the hydrology to the specific river reaches.

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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
ESL .... Extreme sea level
EU .... European Union
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
MPW .... Medium priority watercourse
MRFS .... Mid-range future scenario
OPW .... the Office of Public Works
PFRA .... Preliminary flood risk assessment
Q(T) .... Flow for a given return period
QMED .... Median annual flood, used in FSU methods
RBD .... River basin district
T .... Return period, inverse of AEP
Tp .... Time to peak
TUFLOW .... Two-dimensional hydraulic modelling software
UoM .... Unit of management
WP .... Work package

1 Introduction

1.1 Background

This report describes the hydrological analysis carried out as part of the Catchment-Based Flood Risk Assessment and Management Study (CFRAM) for the Western River Basin. The Inception Report, issued in 2012, presented an initial hydrological analysis including a detailed review of rainfall and flood event data and development of a method statement. This Hydrology Report is intended to be readable with minimal need to refer back to the Inception report. However, not all the hydrological analysis presented in the Inception report is repeated here.

1.2 Objectives of hydrological study

The brief calls for a comprehensive and detailed hydrological analysis that places particular emphasis on flood flow estimation for the main flood risk areas (termed AFAs, Areas for Further Assessment) and the watercourses that flow through these areas (termed HPWs, High Priority Watercourses). It also requires estimation of design flows for watercourses that link the AFAs and connect them to the sea (termed MPWs, Medium Priority Watercourses).

The principal objective of the hydrological study is to derive best estimates of design fluvial flood parameters including peak flows, hydrographs, flood volumes and other design flood parameters, as necessary to deliver the requirements of the CFRAM project, for all Hydrological Estimation Points (HEPs). The study also includes derivation of design coastal flood parameters for AFAs subject to significant coastal flood risk.

1.3 Report structure

Chapter 2 describes the physical characteristics of the study area that are relevant for flood hydrology. Chapter 3 summarises the hydrometric data that have been used in the study and presents the findings of the rating review. The method statement in Chapter 4 sets out an overview of and justification for the choice of analysis method. Chapters 5 and 6 describe the core of the hydrological study, the estimation of design peak flow and design hydrograph shapes. Some of the analysis in Chapters 5 and 6 is described in terms of the entire Western CFRAM study area, since the comparisons of methods were carried out using example sites throughout the Western river basin district. Towards the end of each chapter, the text focuses more specifically on UoM 30. Chapter 7 summarises the approach that has been taken for design flow estimation at each AFA in UoM 30. The remaining chapters deal with application of the flows to the river models, uncertainty and future changes in flood flows.

Detailed results of rating reviews and analysis for individual gauging stations are presented in appendices to keep the main text more readable.

The report is intended principally for readers who understand the basic concepts of flood hydrology and have some familiarity with the methods of the Flood Studies Update.

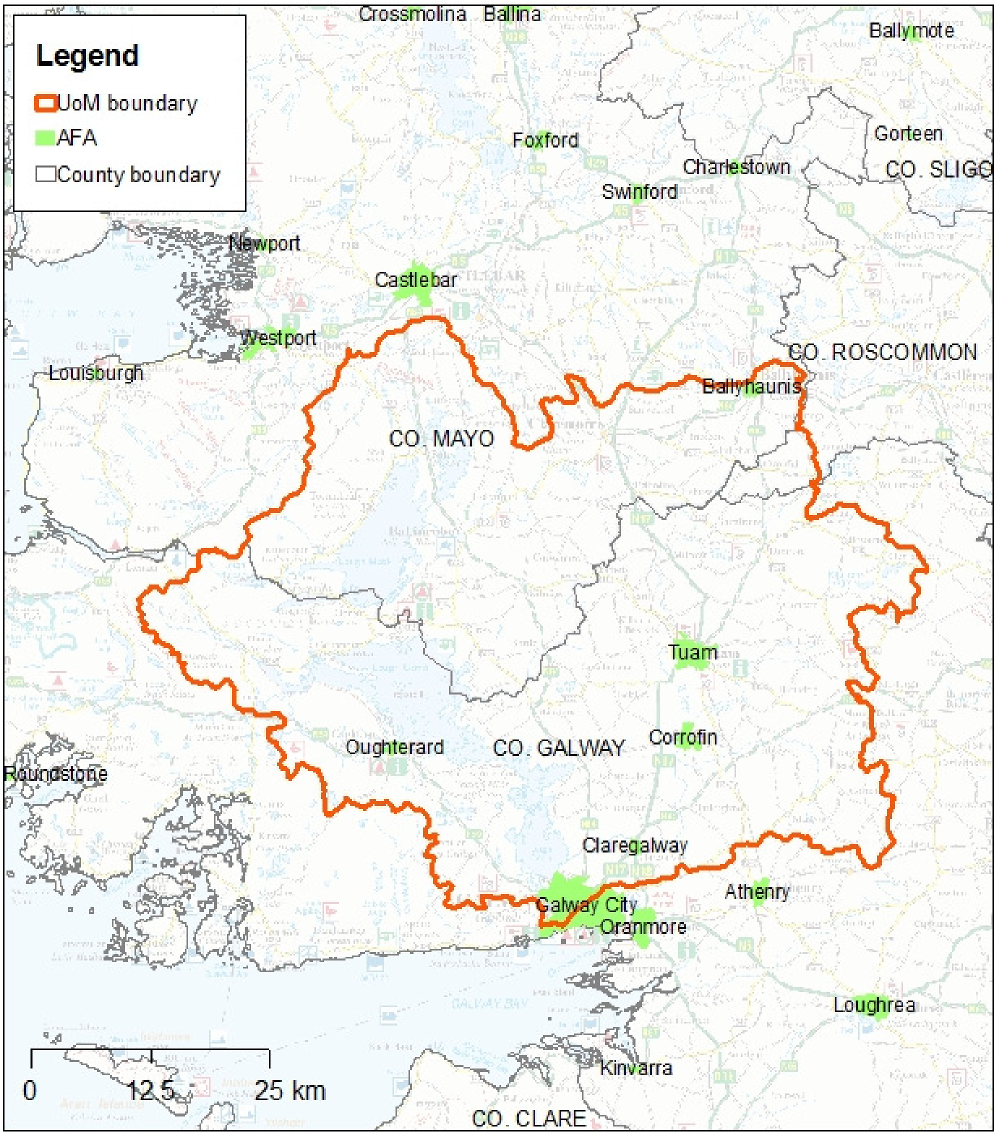
The Hydrology Report for UoM 30 should be read in conjunction with the Hydraulic Modelling Report for UoM 30, and the specific modelling reports for each AFA, which detail the application of the hydrology to the specific river reaches.

Work on the geomorphology study that forms part of the Western CFRAM will be described in the Hydraulic Modelling Report for UoM 30, as will the assessment of the joint probability of fluvial and coastal flooding.

1.4 Unit of management 30 - Corrib

Unit of management 30, shown in Figure 2-1, also referred to as the Corrib catchment, covers an area of 3,113 square kilometres of the Western RBD. The area is predominantly within County Galway but there are also areas of County Mayo and Roscommon included.

Figure 1-1: Unit of management 30: Corrib - overview map



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The Flood Risk Review identified six Areas for Further Assessment (AFAs) in UoM 30. These are:

1. Ballyhaunis
2. Claregalway
3. Corrofin
4. Galway City
5. Oughterard
6. Tuam

The CFRAM for UoM 30 focuses predominantly, but not exclusively, on the six AFAs. Ballyhaunis, Claregalway, Corrofin, Oughterard and Tuam are AFAs at risk from fluvial flooding only, Galway City is from both fluvial and tidal flooding.

Although included as an AFA in UoM 30, Claregalway is being studied separately, parallel to the CFRAM process, in order to obtain some quicker outcomes and no design flows have been developed for this AFA.

2 Hydrology of the study area

2.1 Catchments

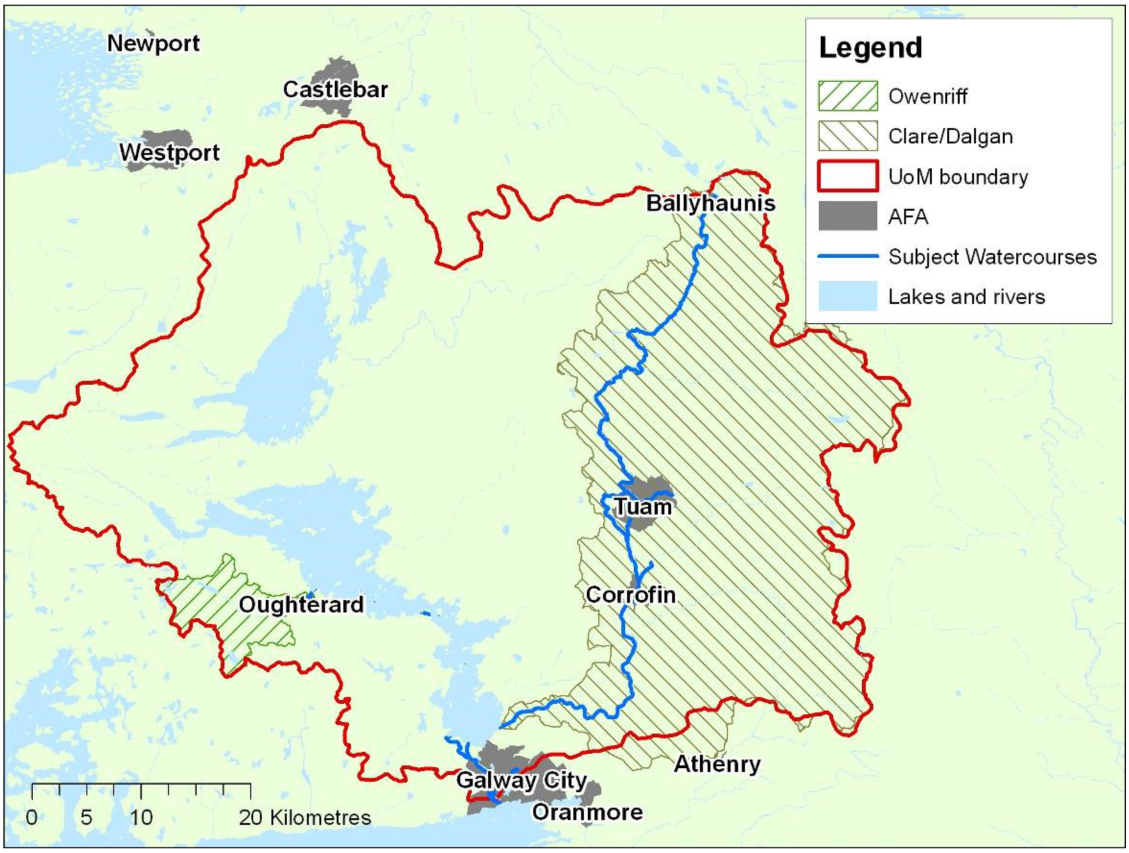
The whole unit of management forms a single catchment, the Corrib. Galway city lies at the mouth of the River Corrib, shortly downstream of Lough Corrib.

Design flows are needed for:

- The Clare River and some of its tributaries.
- The River Corrib at Galway.
- The Owenriff River at Oughterard.

The map below shows the two individual catchments of the Clare/Dalgan and the Owenriff. The catchment of the Corrib covers almost the entire UoM. There is a discrepancy between the UoM boundary and the ArcHydro catchment boundaries provided for use in the project to the east of Galway. The ArcHydro catchment at this location extends eastward towards Athenry (in UoM 29); this has been checked against LIDAR elevation data and found to be realistic. A similar conclusion has been found for the UoM 29 catchments.

Figure 2-1: Subject catchments in UoM 30

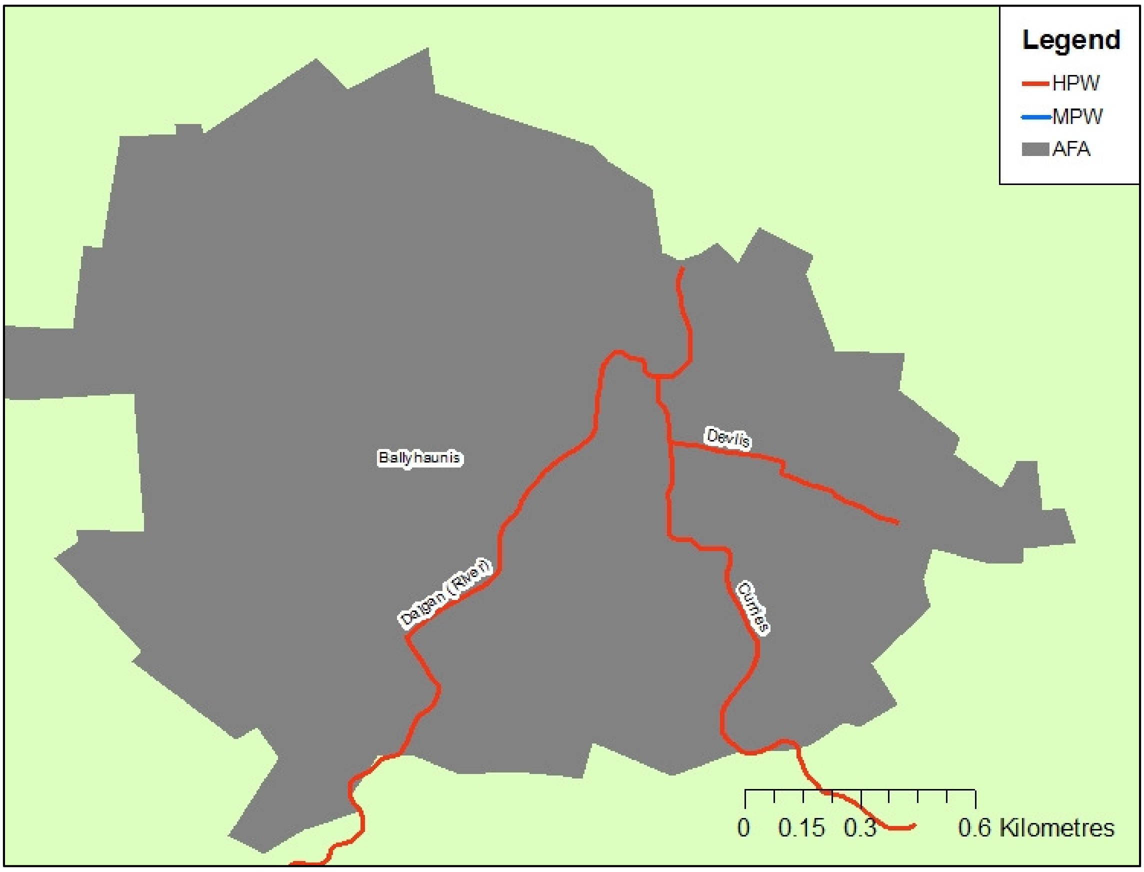


The descriptions below mention catchment descriptors defined in the Flood Studies Update (FSU) Research. Details of these descriptors can be found in the relevant FSU report[1]. Maps of selected catchment descriptors can be found below. Further details of the geology, soils and land use within the catchments can be found in the WCFRAM Strategic Environmental Assessment Scoping Report[2] and further details of each specific watercourse can be found in the WCFRAM Hydraulic Modelling Report for UoM 30.

Clare/Dalgan River

The Clare/Dalgan catchment contains the AFAs of Ballyhaunis, Tuam and Corrofin.

Figure 2-2-1: Ballyhaunis AFA



[1] Compass Informatics (2009). Flood Studies Update Programme. Preparation of Physical Catchment Descriptors (PCD). Pre-final draft report to Office of Public Works.

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

Figure 2-3: Tuam AFA watercourses

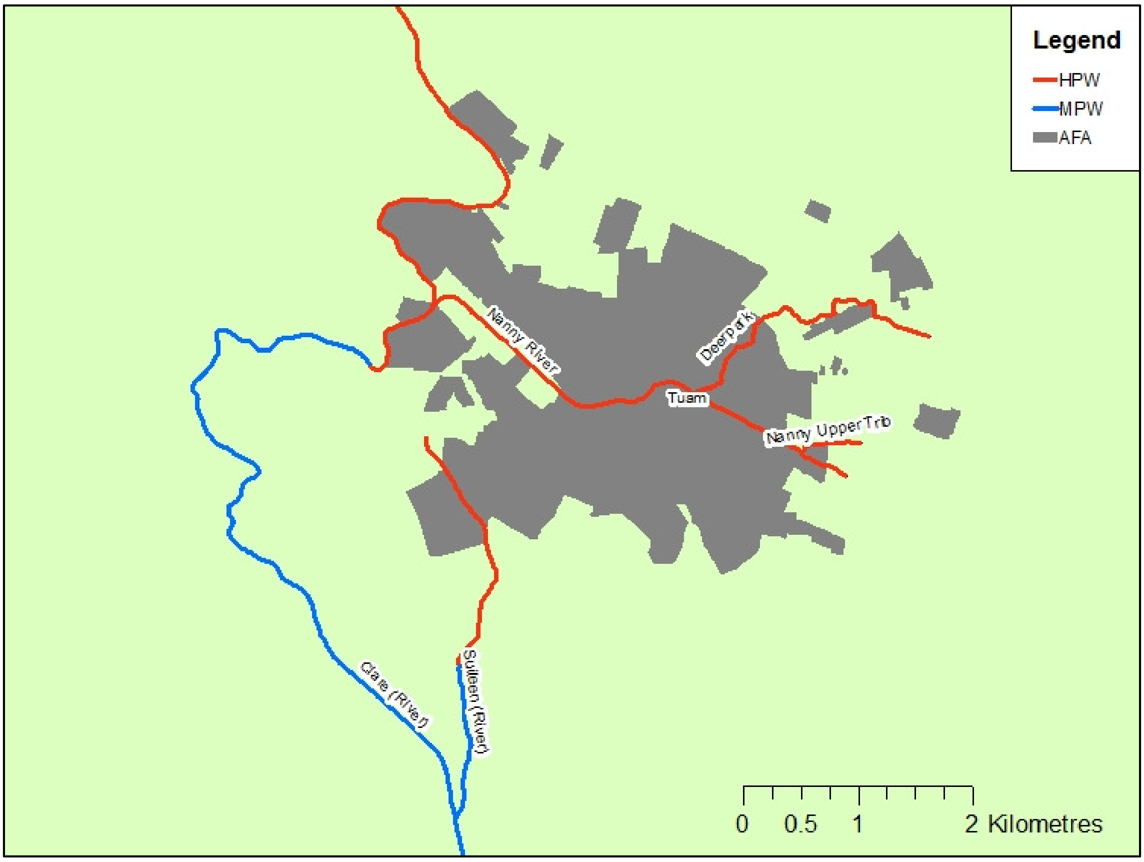
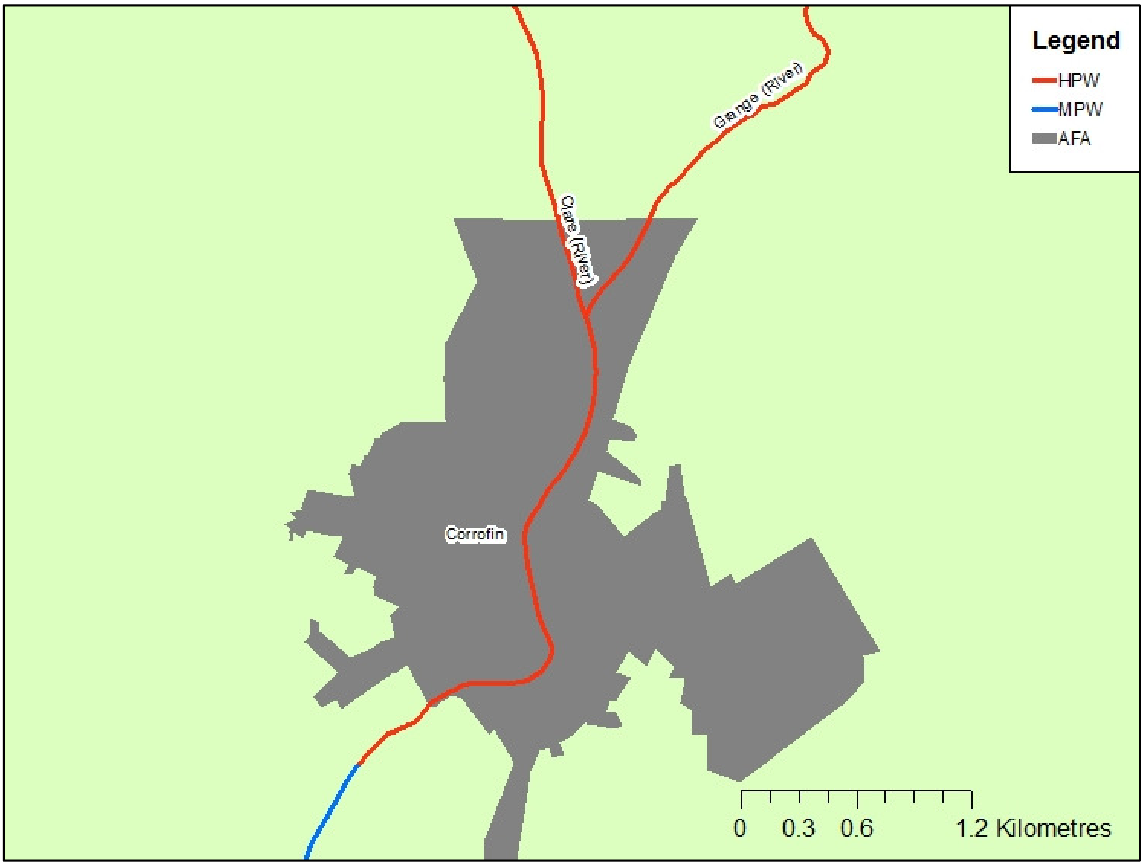


Figure 2-4: Corrofin AFA watercourses



The description below is based on that in the Ryan Hanley (2010)[3] study. It is augmented by reference to FSU catchment descriptors and other sources of information.

The Clare River forms part of the Corrib-Mask catchment. The Clare River catchment is approximately 1,108km^2 which is around 30% of the Corrib catchment. The catchment areas to the downstream limit of the Ballyhaunis, Tuam and Corrofin AFAs are 24km^2, 540km^2 and 704km^2 respectively. The catchment is low-lying, with a mean altitude of 63m. The gradient of the watercourse as a whole (S1085) is 0.73m/km, which is low. The Mask catchment is located to the north west of the river basin and drains into Lough Corrib; there are no sites being investigated in the Mask catchment.

The Clare River, with a reach of approximately 93 kilometres from its confluence with Lough Corrib, rises approximately 8km above the town of Ballyhaunis, County Mayo. Its principal tributaries, working in a southerly direction from its source in County Mayo, are the Sinking River, the Nanny River, the Grange River and the Abbert River.

The mean annual rainfall is 1100mm, and this varies little across the catchment. The mean annual rainfall to the downstream limit of the Ballyhaunis, Tuam and Corrofin AFAs is 1190mm, 1110mm and 1100mm respectively.

The Geological Survey of Ireland (GSI) quaternary map indicates bedrock and shallow till as the overburden cover for the study catchment. This quaternary is generally referred to as free draining. The Flood Studies Report winter rainfall acceptance potential map shows very high infiltration capacity for the catchment (SOIL type 1). However, the baseflow index (BFI) as predicted from soil characteristics is 0.54, indicating only a moderate degree of soil permeability. There are small pockets of lower or higher BFIsoil values in the catchment, but most tributaries show values in the range of 0.50 to 0.65.

The catchment area is underlain by a pale to medium grey, bedded, fossiliferous, medium grained limestone called the Burren limestone. This limestone is generally pure with low clay content. It is often present at or close to the ground surface with only a thin cover of free draining sandy till (boulder clay). The catchment forms part of a Regionally Important Karstified (conduit) Aquifer. Karst features such as turloughs, springs and swallow holes are located throughout the catchment. The type of flooding experienced in karstified catchments can include the backing up of sinking streams with inadequate underground channel capacity or the flooding of closed depressions by rising groundwater.

The storage capacity of the karst is limited. Cawley and Cunnane (2010)[4] point out that the persistent rainfall of November 2009 "resulted in completely saturated catchment systems with the flood storage available in the lakes, floodplains, turloughs and groundwater systems including the karst storage completely exhausted resulting in elevated runoff rates and record flood levels and flow rates".

The present day drainage network has been significantly influenced by arterial drainage schemes, which have covered the entire catchment, carried out since the early nineteenth century to reduce winter flooding.

Prior to drainage, many streams within the present Clare catchment flowed underground or terminated in permanent lakes or turloughs due to the karst geology. These surface waters discharged in underground systems and emerged later as groundwater further down the catchment. A large permanent lake existed north of Corrofin at the confluence of the Grange River and the River Clare. The Abbert River ended in a turlough at Ballyglunin and was not linked by a surface channel to the River Clare. Water in these turloughs flowed into swallow holes and from there via underground conduits until it re-emerged as large springs. An extensive turlough was also located between Corrofin and Turloughmore. Water flowed underground through swallow holes in this turlough and re-emerged in springs such as at Loughgeorge. There was no surface water channel from this turlough to Lough Corrib.

[3] Ryan Hanley (2010) Study To Identify Practical Measures To Address Flooding On The Clare River. Report for OPW.

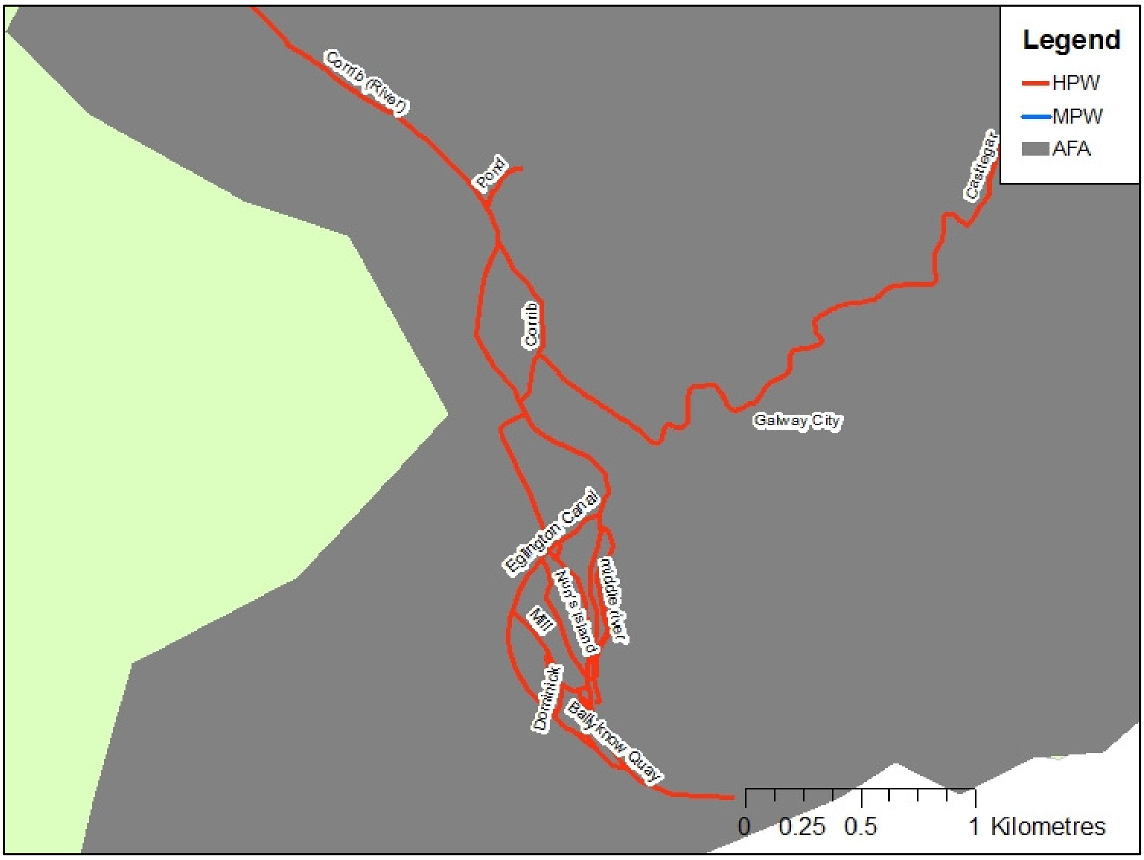
[4] Cawley, A. and Cunnane, C. (2010) 02 - Comment on the November 2009 Flooding in the Shannon and Corrib Systems. Irish National Hydrology Conference 2010.

Arterial drainage works in the Clare catchment initially involved removing the water from the upper and middle parts of the catchment, and the reduction of flooding at Ballyglunin and Corrofin.

A further drainage scheme was carried out by the OPW in the 1950s and 1960s on the Clare River. The scheme involved continuous channel excavation along the whole length of the Clare River, including deep rock cuts at Lackagh, Corrofin and Conagher. Similar works were also undertaken on the tributaries and smaller watercourses. The objective was the relief of 135,000 km^2 of agricultural land from flooding and water logging.

River Corrib

Figure 2-5: Galway City AFA watercourses



The Corrib flows along a short channel through Galway City and joins the outlet of Lough Corrib to the sea. Its catchment is large (3140km^2) and heterogeneous. 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.

For the Corrib catchment as a whole the mean altitude is 65m and the gradient of the longest (S1085) is 0.58m which is very low. This is because the Clare River is the longest watercourse flowing into Lough Corrib. The mean annual rainfall across the catchment is 1422mm.

The major influence of Loughs Corrib and Mask is measured by the catchment descriptor FARL (Flood Attenuation due to Reservoirs and Lakes) which is 0.66. Lough Corrib is the second largest lake in Ireland, with an area of 178km^2. It has a major influence on the nature of flood flows along the River Corrib through Galway. Just over a quarter of the Corrib catchment area drains via Lough Mask, to the north of Lough Corrib. Lough Mask has an area of 89km^2 and drains into Lough Corrib via underground karst conduits.

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)[5]. 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[6].

In 1959, the 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 and 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[7].

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 the shoreline and lower reaches of tributary rivers. The original design envisaged that this upper limit level would be reached at a flow of 11,000 cusecs (311m^3/s). This upper limit has been exceeded almost every year, apart from 1995 and 2005[8]. The effectiveness of the barrage in controlling lough levels was investigated in a report by OPW in 1987. It found that there is a lack of control due to friction losses along the channel between the lough and the barrage - i.e. the main hydraulic control is the channel rather than the barrage. Even with all gates open all year, the study found little reduction in the highest peak lake levels. The report concluded that the channel needed enlarging to discharge enough flow. Peak lake levels were found to be independent of levels at the start of flood events, the implication being that drawing the lake down in advance of floods would not be worthwhile. OPW have a record showing which sluices are open at a twice daily time step.

Despite the findings of the hydraulic modelling study in 1987, it has been reported that residents along the Clare River have a strong and near-unanimous perception that the water level in the vicinity of Claregalway and further upstream is affected by the operation of the sluice barrage (Ryan Hanley, 2010)[7]. This conflict between local perception and hydraulic calculations is not unique to the Corrib catchment!

Due to the large size of Lough Corrib, wind setup can result in significant differences between water levels at opposite ends of the lake (up to 0.4m). This can reduce water level at the outlet, thus reducing discharge so that high lake levels persist longer.

[5] T. W. Freeman (1957): Galway—the key to west Connacht, Irish Geography, 3:4, 194-205

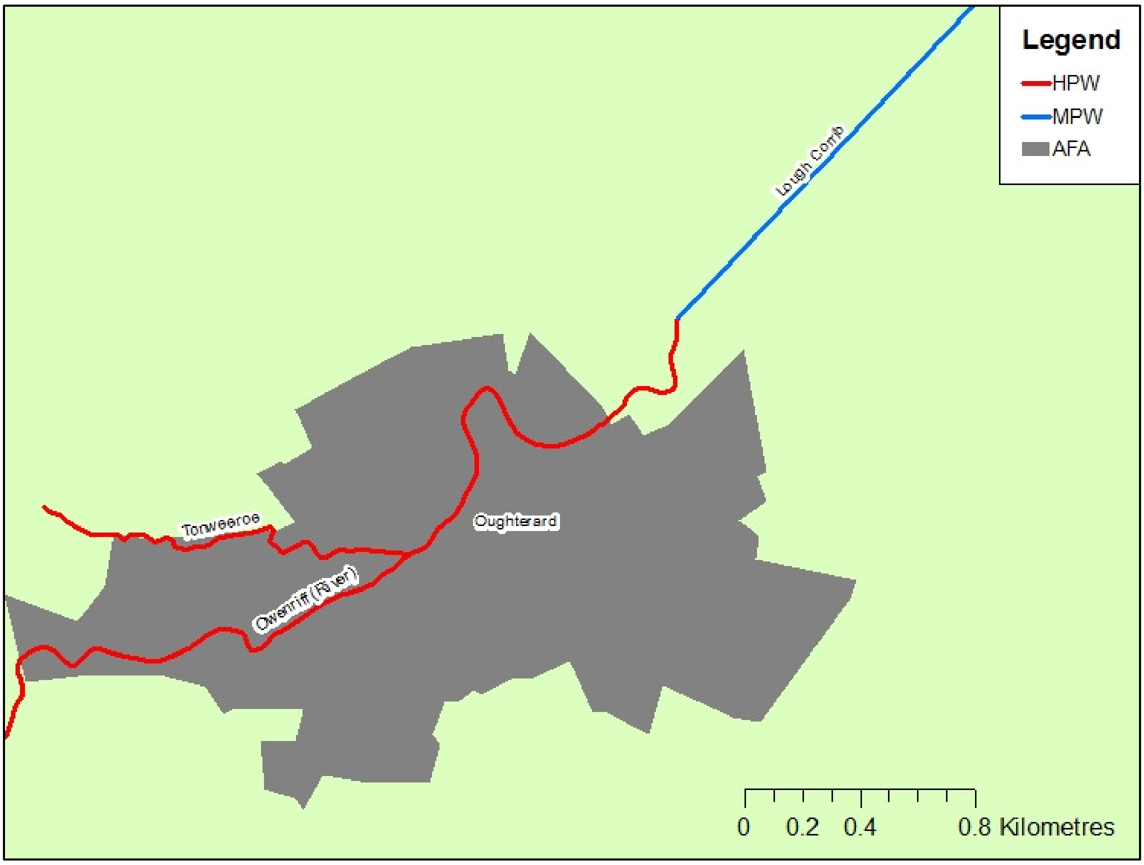
[6] Eamon de Buitlear (1985) Irish Rivers.

[7] Hydro-Environmental (2008) Impact of proposed remediation measures on flooding at Southpark and Grattan Road Galway. Report to Galway City Council.

[8] Ryan Hanley (2010) Study To Identify Practical Measures To Address Flooding On The Clare River. Report for OPW.

Owenriff River

Figure 2-6: Oughterard AFA watercourses



The Owenriff has a medium-small catchment (67km^2) at the point where it enters Lough Corrib. The catchment is hilly, with altitudes up to 250-300m. The mean altitude is 112m. The gradient of the watercourse as a whole (S1085) is 5.6m/km, fairly steep. There are several lakes in the upper part of the catchment. Their influence is indicated by the value of FARL, 0.75, which signifies a large amount of attenuation.

The annual average rainfall is 1907mm as might be expected given the topography and location of the catchment.

The bedrock geology includes Precambrian metamorphic rocks with granite intrusions. Two thirds of the catchment is covered with peat, largely upland blanket bog. Deeper well-drained mineral soils are found on the lower lying land. The BFI as predicted from soil characteristics is 0.48, indicating an average degree of soil permeability.

The catchment is almost entirely rural and 25% is forested.

2.2 Maps of selected catchment descriptors

The maps below show how catchment properties vary across the unit of management. Each point indicates the properties of the catchment draining to that location. The FSU research derived values of catchment descriptors at 500m intervals along flow paths for all catchments draining an area of at least 1km^2.

Figure 2-7: Standard-period annual average rainfall, SAAR

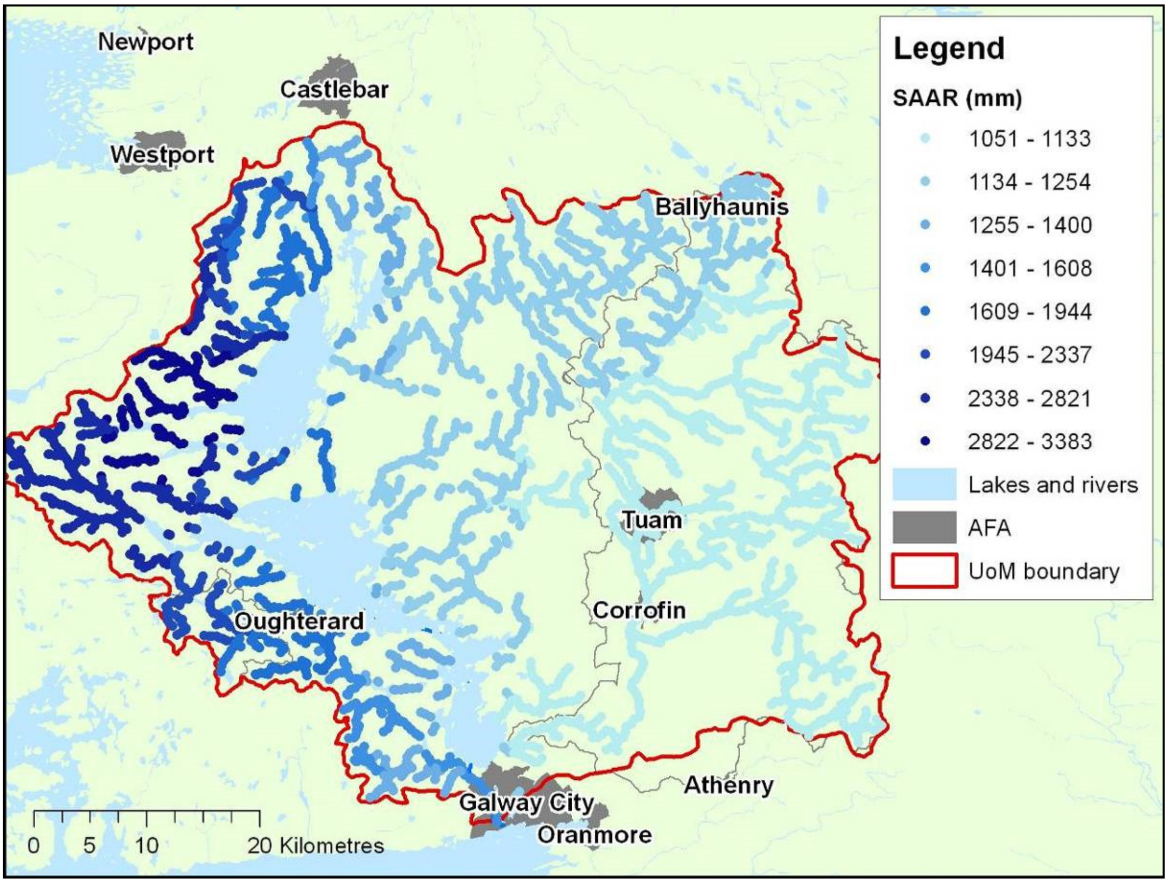


Figure 2-8: Baseflow index estimated from soil properties, BFI_{SOIL}

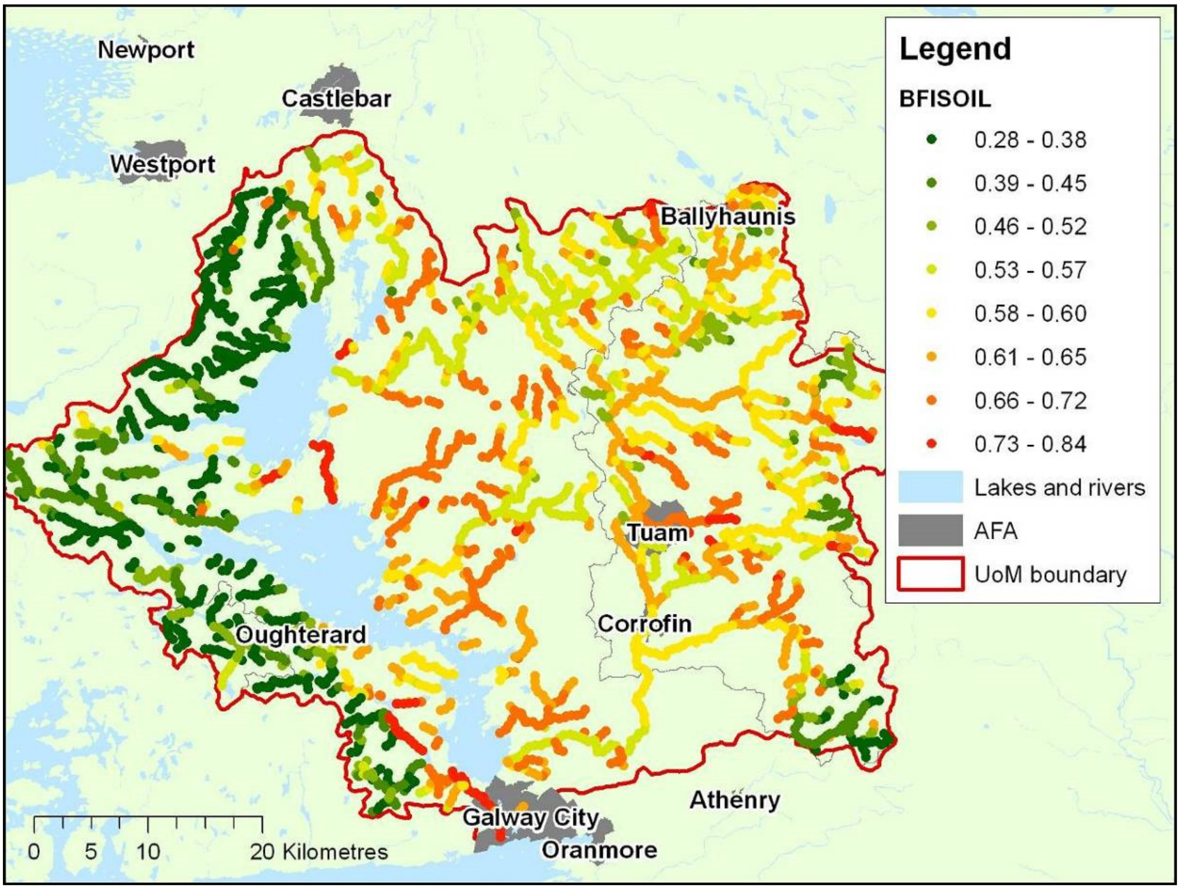


Figure 2-9: Slope of the main watercourse in the catchment, S1085

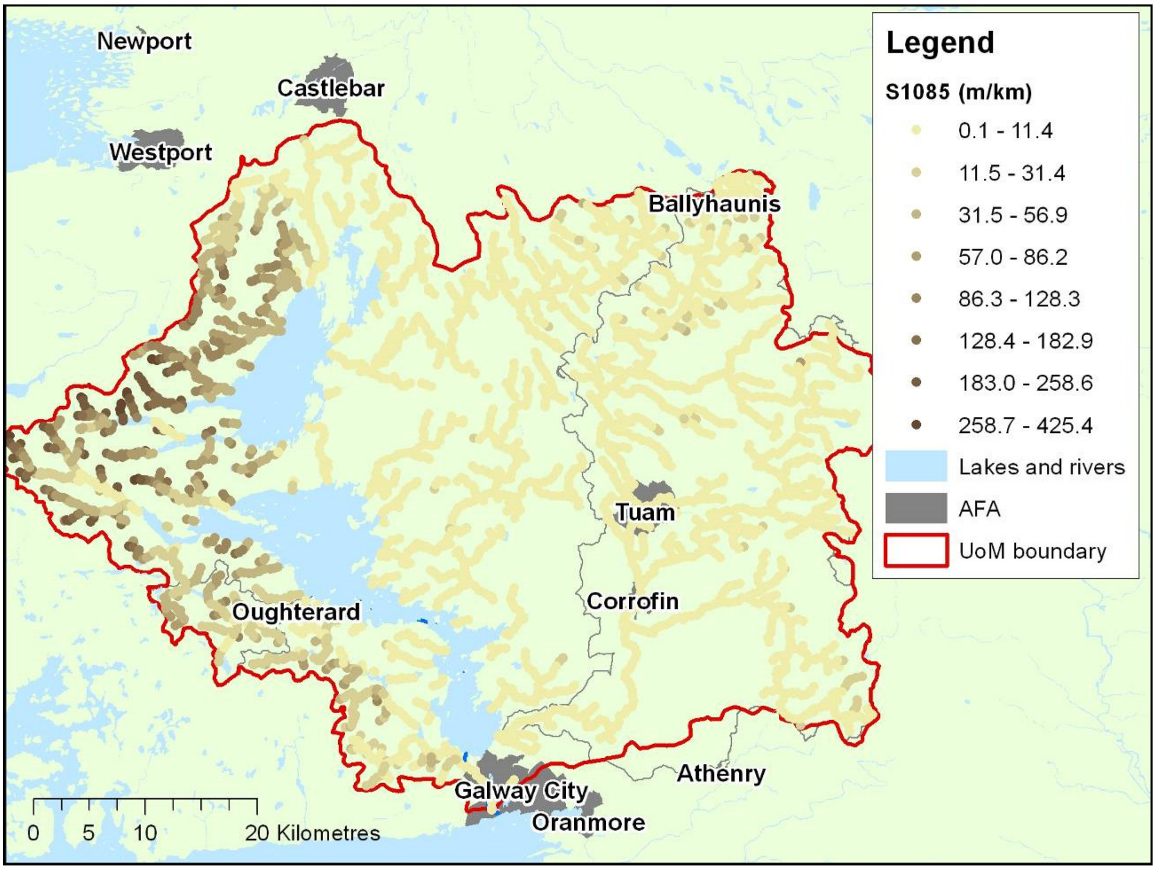
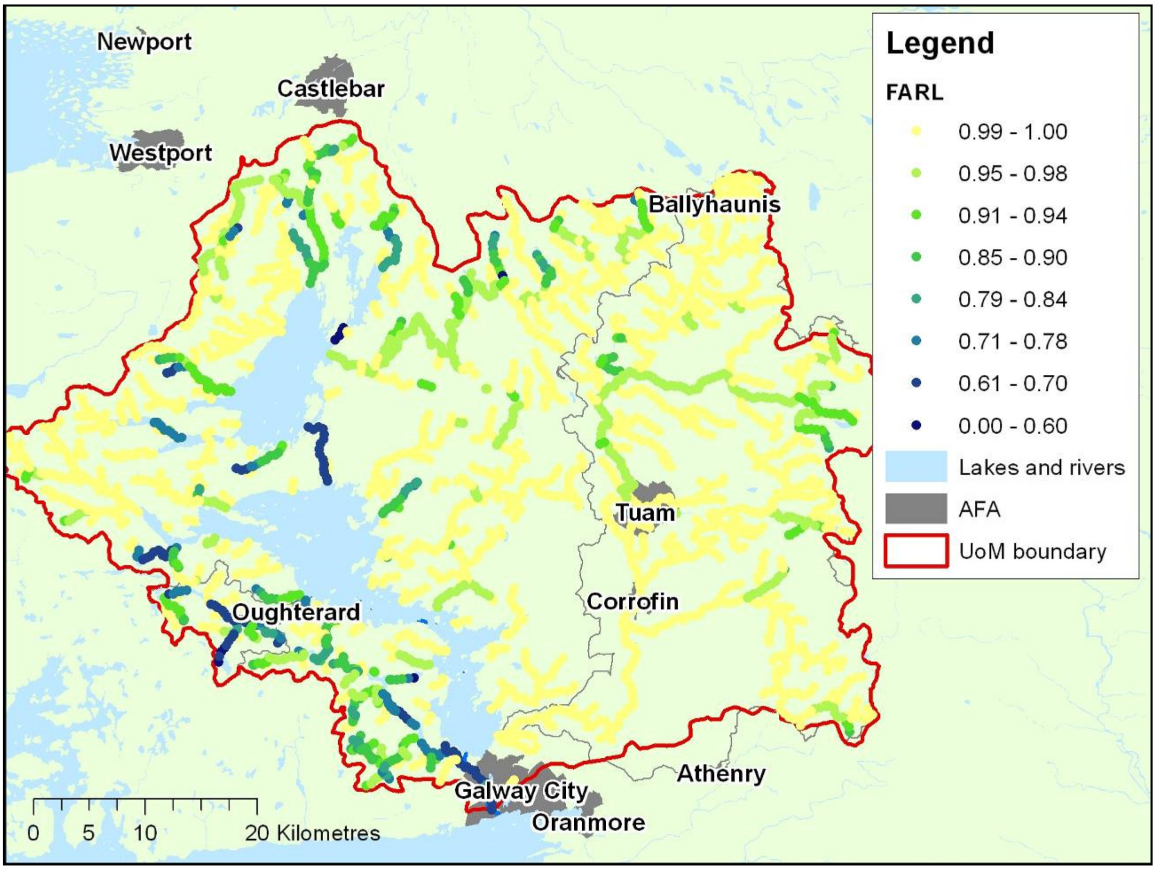


Figure 2-10: Flood attenuation by reservoirs and lakes, FARL



3 Hydrological data

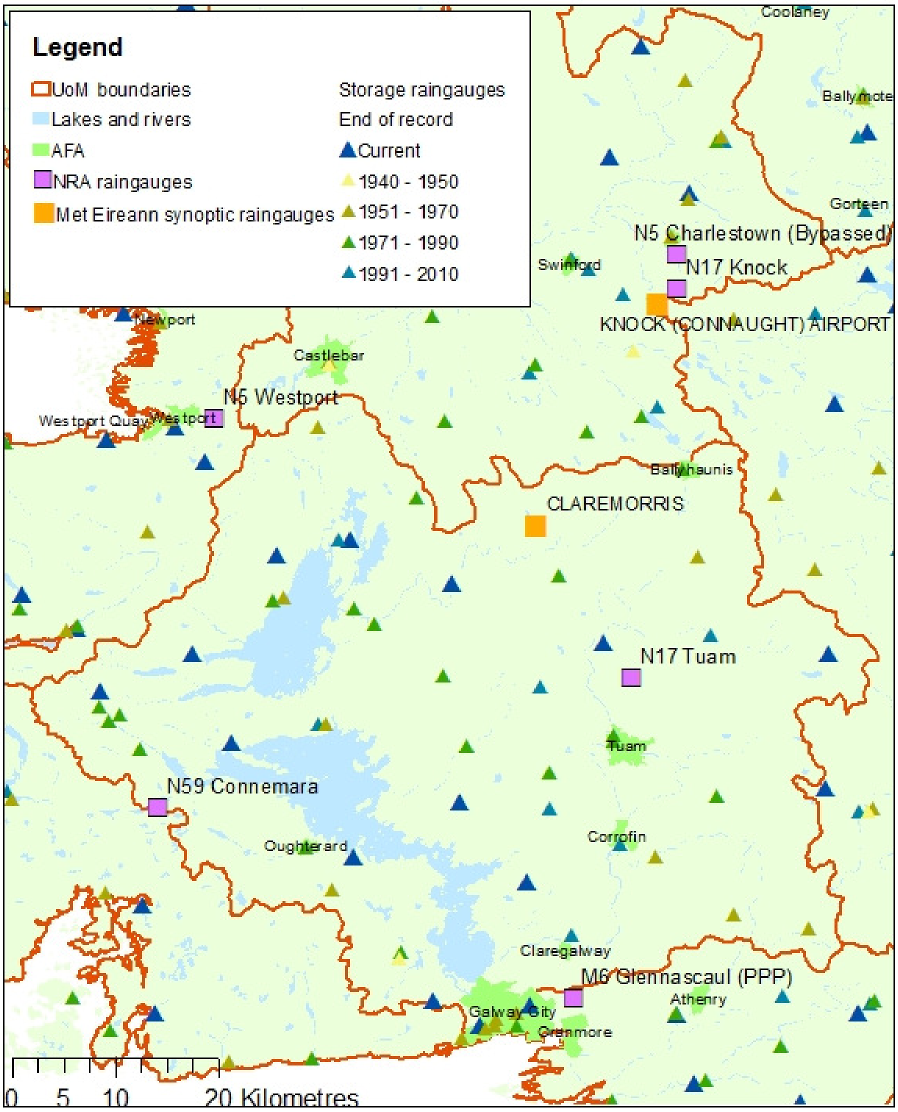
3.1 Meteorological data

Figure 3-1 shows raingauges (past and present) for which digital data is held by Met Éireann within this unit of management. There is one synoptic raingauge (i.e. a recording gauge that measures rainfall at a sub-daily time step), at Claremorris, along with another at Knock Airport which is just outside the area and to the north.

Data from all the gauges shown has been provided by Met Éireann. Some of the gauges have digital data available from the 1940s. Claremorris synoptic gauge has data from 1950. All Met Éireann rainfall datasets are subject to quality control procedures and thus have been treated as high-quality data. However, consistency checks have revealed a small number of suspect daily totals, which are described in the rainfall event analysis summary sheets (Appendix G). Apart from these exceptions, the rainfall data is regarded as fit for purpose.

Analysis of the rainfall data, from synoptic source, tipping bucket gauges and storage gauges, was reported in the Inception report and in Section 6.3 which describes lag analysis. Additional rainfall data is collected by the National Roads Authority using rainfall sensors. This dataset was provided after completion of the inception phase and so it has not been incorporated in the analysis of rainfall events. Data was provided for some but not all NRA gauges shown on the map. The NRA data was considered for additional lag analysis as described in Section 6.3.

Figure 3-1: Raingauge locations



3.2 Fluvial data

Figure 3-2 shows the river gauging stations in the catchments where AFAs have been identified within this unit of management. It shows only those stations at which a continuous record of river level is available, excluding staff gauges where occasional readings are taken. It includes closed gauges as well as current ones. In total there are nine river level gauges that have been judged as potentially useful for this study. At six of these nine gauges it is possible to calculate flow from the observed water levels using a rating equation:- Corrofin (30004), Ballygaddy (30007), Claregalway (30012), Claremount (30019), Ballyhaunis (30020) and Oughterard D/S (30101). Review and extension of rating equations for these stations is described in Section 3.3. Of these six stations, Oughterard (30101) and Claremount (30019) do not have existing ratings. The 'Other gauges' shown on the map will be used in the development of pooling groups.

Figure 3-2: River gauge locations

Summary information on the gauges and their relevance to this study is given in Table 3-1. River level and flow data, where available, has been provided for all these gauges by the OPW and EPA.

Table 3-1 Summary of river level and flow gauges

Ref. No.	Name	Catch -ment Area (km^2)	Start of record	End of record	Flow available ?	FSU quality class	Comments
30004	CORROFIN	699	1951	-	Yes	A1	Rating review gauge
30007	BALLYGADDY	470	1974	-	Yes	A2	Rating review gauge
30012	CLAREGALWAY	1073	1996	-	Yes	B	Rating review gauge
30019	CLAREMOUNT	63	1976	2003	Yes	U	Rating review gauge (although closed)
30020	BALLYHAUNIS	21	1975	-	Yes	B	Rating review gauge
30045	HAZELHILL	22*	2001	2007	No	n/a	
30061	WOLFE TONE BRIDGE	3136	1950	-	Yes but only from 1972 to 2004	A1	See note 3 below.
30098	DANGAN	3121	2003	-	No	n/a	Close to 30061. Water level data may be useful for model calibration.
30101	OUGHTERARD D/S	64	2001	-	Yes	U	Rating review gauge, replacement for 30019

*From supplied hydrometric data register only

Notes:

1. The start of record is given as the earlier of the year from which continuous digital data is available or the year from which flood peak data are available. Some gauges have earlier records available on paper charts.
2. FSU quality classes indicate the extent to which high flow data can be relied on as judged by the Flood Studies Update research programme. Class A gauges are thought to provide reasonable measurement of extreme floods, and thus are suitable for flood frequency analysis (the best gauges being classed as A1); class B are suitable for calculation of moderate floods around QMED and class C have potential for extrapolation up to QMED. Class U indicates gauges thought to be unsuitable at the time of the FSU research. These quality classes were developed around 2005-2006 and some may no longer be applicable following recent high flow gaugings.
3. At Wolfe Tone Bridge there are no gaugings available before 1972 and OPW have advised that it is not advisable to calculate flows before 1972 using a rating derived from later gaugings, thus the only data available before 1972 is water level. The gauge was moved in 2004 and a rating has not yet been established for the new location. There is understood to be reasonable confidence in the high flow data between 1972 and 2004, which are derived by interpolation between successive low tides as the water level is affected by tidal backwater effects during the higher parts of the tidal cycle.
4. All gauges with flow available have rating equations and check gaugings. All gauges listed have annual maximum series.
5. All gauges are operated by OPW apart from Claregalway and Ballyhaunis, which are operated by Galway and Mayo County Councils respectively

There are also several lake level gauges which have been used in the analysis, for example in estimating design lake levels for use as initial conditions in model runs. On Lough Corrib there are four gauges: from the north, Cong Pier, Barrusheen, Annaghdown Pier and Angliham. Although some of these gauges have apparently been in place since the 1950s, the longest digital record is from 1972 at Cong Pier (station 30084).

All AFAs benefit from nearby gauging stations:

- At Ballyhaunis there is a gauge on the River Dalgan with peak flows available from 1988 onwards. The rating equation has been reviewed as part of this CFRAM study. The existing rating can be treated with moderate to low confidence for high flows.
- At the upstream end of Tuam there is a gauge on the Clare River, Ballygaddy, with flood peak data from 1974. The rating equation has been reviewed as part of this CFRAM study. The existing rating has a good degree of confidence for in-bank flows (up to QMED) but much less so for larger floods.
- At Corrofin there is another gauge on the Clare River with flood peak data from 1964. The rating equation has been reviewed as part of this CFRAM study. The existing rating has

a good degree of confidence up to QMED but shows considerable scatter for larger floods. There is also a gauge downstream on the Clare at Claregalway, although the record of flow data is fairly short (1996 to date).

- At Galway there is a gauge on the Corrib at Wolfe Tone Bridge which has flood peak data available from 1972 to 2004. There is understood to be reasonable confidence in the high flow data, which are derived by interpolation between successive low tides as the water level is affected by tidal backwater effects during the higher parts of the tidal cycle.
- At Oughterard there is a gauge installed in 2001 for which there is currently no rating equation. The gauge was a replacement for an earlier site shortly upstream at Claremount which also had no rating equation for flood flows. Ratings for both stations have been created and/or reviewed during this study.

3.3 Review of rating equations

Six gauges in UoM 30 have been identified by OPW for rating reviews: Corrofin, Ballygaddy, Claregalway, Claremount, Ballyhaunis and Oughterard.

All gauging stations have been visited in order to assess the physical characteristics of the river channel and floodplain such as hydraulic controls on water level (at low and high flows), hydraulic roughness and potential bypass routes in flood conditions. Existing rating equations, available at four of the six stations, have been assessed by comparison with check flow gaugings and confidence limits have been calculated to indicate the uncertainty associated with the rating across the range of flows. They were found to fit check gaugings well in most cases, and some are gauged up to very high levels which gives some confidence in calculation of flood flows. Appendix A contains recommendations on the type and extent of hydraulic modelling needed for extending the existing ratings or developing new ones.

Most gauges fall within reaches which will also be modelled for the purpose of flood mapping as they are close to AFAs. The development of hydraulic models provides an opportunity to extend the rating equations above the range of flows for which check gaugings are available.

Extended or new ratings have been produced at all six rating review gauges in UoM30, with moderate to high confidence at five gauges. The exception is Claremount where confidence in the rating is low. This is because of a number of factors that combine to give large uncertainty in the rating at high flows. For one thing, the gauge has been removed and there are no check gaugings since 2001. Another complicating factor is the nature of the hydraulic control, which may be influenced by a series of waterfalls and cascades which were not possible to survey owing to the presence of freshwater pearl mussels. The check gaugings do not extend to flood flows, and the initial extended rating, derived from a model that was calibrated to fit the gaugings, predicted flows that were inconsistent with those recorded by the new downstream gauge at Oughterard during the period of overlapping records. The final modelled rating gives flows that are more consistent with those at Oughterard but at the cost of making some assumptions about the hydraulic processes. Given that flow data is available shortly downstream from a gauge that has high confidence in its rating, the hydrological analysis has given little weight to flows at Claremount derived from the new rating.

At the remaining four rating review gauges, which have existing ratings, the revisions to ratings have led to relatively small alterations in the estimated magnitude of the median annual maximum flow, QMED. At Claregalway and Ballyhaunis there is no change in QMED. At Ballygaddy QMED has decreased by 6% and at Corrofin it has decreased by 9%.

The results of the rating reviews can be found in Appendix A. Rating equations at other gauging stations, such as those in the east of the unit of management, are available from the operator of the station, i.e. OPW or local authorities.

3.4 Tidal data

Figure 3-3 and Table 3-2 detail the location and available data associated with tidal gauges around the west coast of Ireland. Many of these gauges have been recently installed and are part of an ongoing project to develop a centrally controlled Irish national tidal network.

Due to the large distances between the gauges within the Western CFRAM study area and the short timeframe that data is available for, the use of this data for the purposes of calibration will be limited. Where the gauge is located at the AFA (Galway and Sligo) and there is a tidally influenced gauge located on the watercourse there will be good confidence in the suitability of the gauge data for the site. Where the AFAs are situated between gauges, (Ballina, Newport, Westport, Louisburgh, Clifden and Roundstone), there will be much lower confidence in data extrapolated to the AFA. The effects of the local inlets and bays on tidal levels will not be known and calibrations using this data should be treated with caution.

Figure 3-3: Tidal gauge locations

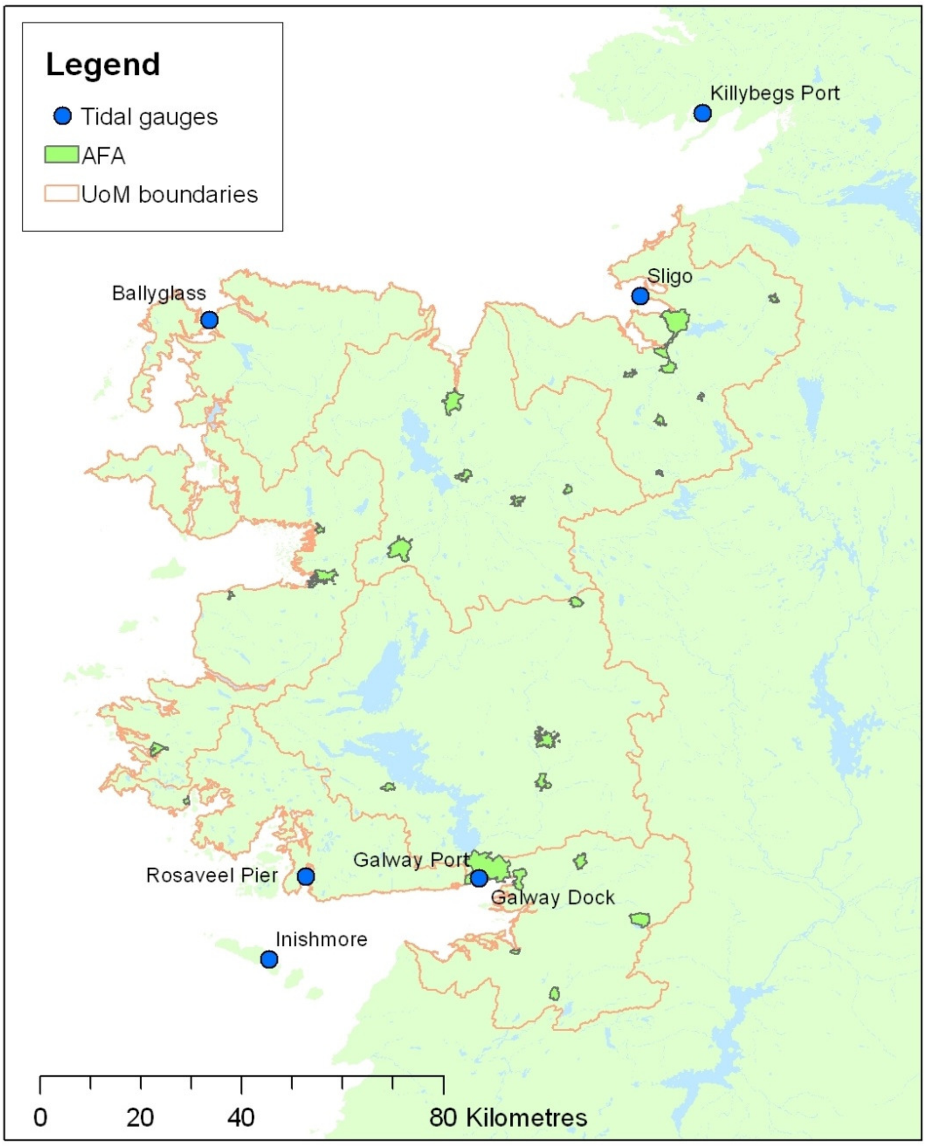


Table 3-2 Summary of tidal gauges

Name	Operating Authority	Start of record	End of record	Comments
Killybegs	Marine Institute	Mar 2007	-	
Sligo, Rosses Point	Marine Institute	Jul 2008	Aug 2013	
Ballyglass	Marine Institute	Apr 2008	-	
Inishmore	Galway Co. Co.	Apr 2007	-	Currently inactive due to harbour works
Rosaveel Pier	OPW	Jul 1986	-	
Galway Port	Marine Institute/Galway Port Company	Mar 2007	-	
Galway Dock	OPW	Sep 1985	Nov 1989	

3.5 Historical flood data

Information on historical flooding is helpful in developing an understanding of flood risk in the area and can help guide the estimation of design flows.

A considerable amount of information on historic flooding and natural disasters was available for UoM30, dating back as far as the mid-19th century. The majority of the information from the early years was limited to landslides and gales, but the associated flooding incidents were reported sporadically. However, information on flooding found in newspapers (and other sources) from the early 20th century also included some indication of the magnitude and/or extent of the flood (rather than just a comment such as "the most severe flooding ever", etc.). The following sources of information were used for the investigation of historic flooding.

- Irish Newspaper Archives (www.irishnewsarchive.com). The search included newspapers such as Irish Independent 1905 - 2011, Irish Press 1931 - 1995, Freemans Journal 1763 - 1924, Tuam Herald 1837 - 2000, Sunday Independent 1905 - 2011, Connacht Tribune 1909 - 2011.
- Hickey, K. (2010) Deluge. Ireland's weather disasters 2009-2010. MPG Books, Bodmin.
- A flood chronology for the Western River Basin District compiled by Kieran Hickey of Dept of Geography, NUI Galway, for the purposes of this study.
- Archer, D. (2011) Northern Ireland flood chronology. Personal communication.
- Database of historical weather events (http://booty.org.uk/booty.weather/climate/wxevents.htm)
- Local history websites and books.
- Previous flood studies for the area, as described in Section 3.2.
- Papers published in journals or presented at conferences.
- Reports and flood outlines available on www.floodmaps.ie.
- Information provided by local authorities during the flood risk review.
- Hydrometric data, in particular long-term flow and rainfall records.

Most of these sources can be regarded as good-quality datasets, although any anecdotal information, particularly if it has been gathered some time after the flood event, has been treated with appropriate caution.

Analysis of the historic information is described in Section 5.2.

4 Method statement

The general approach followed for estimating design flows in this unit of management was developed during the inception stage. This chapter of the report sets out the thinking behind the methods that have been chosen, focusing on the nature of the catchments (described in Chapter 2), the data available (described in Chapter 3) and the needs of the study (described below).

4.1 Needs of the study

The specification calls for estimation of design flood parameters for eight AEPs, ranging from 50% to 0.1%.

Estimation of design groundwater conditions is not required, as groundwater flood mapping is being covered in a separate nationwide study. However, in karst areas such as much of this unit of management, there is not always a distinct boundary between fluvial and groundwater flooding. Some rivers are affected by outflows from or inflows to underground karst systems. The ongoing groundwater flood mapping study does not include any modelling of karst flows in these catchments (concentrating instead on analysis of water levels at turloughs). Therefore design flows will be based on analysis of river data, which will implicitly account for the influence of upstream karst features up to the magnitude of the events recorded.

There are five AFAs in this unit of management for which design flows are required. Design flows are needed for:

- The Clare River and some of its tributaries. There are AFAs at Ballyhaunis (on the River Dalgan at the head of the Clare catchment), Tuam (mid-way down the catchment at the confluence of the Nanny River with the Clare) and Corrofin (shortly downstream of Tuam, between the confluences of the Grange and Abbert Rivers). Flow estimates will also be needed for points on the Clare River and its tributaries (at their confluence) in between the AFAs because the entire length of the watercourse downstream of Ballyhaunis will be modelled. Refer to Figures in Section 2.
- The Owenriff River at Oughterard. Refer to Figures in Section 2.
- The River Corrib at Galway. Refer to Figures in Section 2.

The specification calls for HEPs to be located upstream, downstream and centrally at each AFA and at all gauging stations. Points must also be located upstream and downstream of tributaries contributing more than 10% of flow in the main channel with no greater spacing than every 5 km. These guidelines have been followed wherever possible when locating these points, in addition to adding a point wherever the catchment area increases by 10%.

However, in certain locations the guidelines have been adapted. For example, until the hydrological analysis is undertaken it is not possible to ascertain which tributaries contribute 10% of main channel flow; therefore AFAs are defined for those tributaries that contribute greater than 10% of catchment area. Elsewhere it may be the case that the location of a point at the upstream extent of the AFA is not necessary, when another point is located nearby (i.e. at a tributary confluence). It is also not practical to add a flow estimation point everywhere the catchment increases by 10% on very small tributaries - this would result in an unmanageable number of points. Where this is the case a minimum point spacing of 400m has been employed (this has superseded the 200m spacing proposed in the Inception Report as initial results highlighted no significant change in design flows on these small watercourses at this spatial scale).

The locations and catchment boundaries of HEPs are included as ArcGIS shapefiles within the digital deliverables from the Western CFRAM project, Section 12.

Catchment boundaries for each HEP have been obtained from the information supplied by the OPW (which were derived for implementation of the Water Framework Directive). These have been checked using Arc Hydro, as described in the Inception Report. Catchment descriptors for each HEP were obtained from the FSU datasets, with adjustments made where catchment boundaries were in error, again as described in the Inception Report.

4.2 Choice of method

There are two quite distinct types of catchment for which design flows are needed. On the lower Clare and Corrib, floods are prolonged and some are difficult to regard as single events because they occur as a result of sequences of rain storms. Although the primary impact of a flood may be due to the peak water level that is reached, secondary damage is largely the result of the duration of flooding and relates to the time that economic activity is suspended and to the cumulative social, structural and agricultural impacts of long term inundation. As river basin size increases, secondary damage becomes an increasing proportion of total damage (Anderson et al., 1993[9]). A consequence is that accurate estimates of flood durations and volumes will be important on these catchments.

In contrast, the Owenriff catchment is short and steep with little storage available and thus floods are much briefer and can be characterised more fully by their peak flow and level.

Because there are gauging stations in or near to all AFAs, the natural choice of method is to estimate both design peak flows and design hydrographs from locally recorded data where its quality and length of record are adequate. Peak flows have been estimated from QMED derived from at-site gauged data or from catchment descriptors which have then been adjusted by data transfer using upstream or downstream gauges as pivotal sites where possible.

Flood growth curves were initially derived from a combination of single-site and pooled analysis, with comparisons made between the two at all gauges with enough good-quality annual maximum flow data. This analysis incorporated the revised or newly-created flood series at all gauging stations in the Western CFRAM for which satisfactory rating equations have been developed during the rating reviews.

After reviewing the flood outlines produced by model runs which used the first iteration of design flows, some revisions to design flows were made in order to ensure flood levels and extents were not underestimated for the most extreme events. These revisions comprised applying the FSR rainfall-runoff method to estimate the gradient of the upper portion of the growth curve, for return periods in excess of 100 years at all HEPs.

Characteristic flood hydrographs for flow estimation points at and near gauging stations have been based on analysis of observed hydrographs (Appendix D) and assessed, at key gauges, against the results of flood volume frequency analysis (see analysis for Ballygaddy in Appendix E). At locations between gauges, or for setting inflows to the model from tributaries, a variety of methods for defining characteristic flood hydrographs have been tested, full details of which are included in Section 6. These included:

- Deriving a characteristic hydrograph using the parametric method from FSU Work Package (WP) 3.1 in which a hydrograph (standardised to have unit peak) is represented by a combined gamma and exponential distribution whose parameters are estimated from catchment descriptors. A potential drawback of this approach is that it can result in hydrograph durations that are not realistic given the size of the catchment.
- The above approach with parameters adjusted by reference to any nearby similar catchments for which observed flood hydrographs are available.
- The Flood Studies Report rainfall-runoff method, in which hydrograph shapes are determined largely by the characteristics of the catchment, i.e. time to peak and annual average rainfall.

Continuous simulation was also considered for this UoM but ultimately discounted. Details of the rationale are found in the Inception Report.

[9] Anderson, R.J., dos Santos, N. and Diaz, H.F. (1993) An analysis of flooding in the Parana/ Paraguay River Basin. Laten Dissemination Note 5. Latin America and Caribbean Technical Dept. Environment Division. World Bank. Washington DC.

5 Estimation of peak flows

5.1 Descriptive analysis of flood peak and flood volume data

Analysis of flood peak data at six gauging stations, 30004, 30007, 30012, 30020, 30061 and 30101, is recorded in Appendix B and summarised here. These are the gauges that have been used to estimate design flows for the study watercourses because they are appropriately located and have suitable peak flow data.

The magnitude of estimated design flows will be based closely on analysis of local flood peak data where it is suitable, so it is important to develop an understanding of the statistical characteristics of the datasets. This includes testing for non-stationarity (i.e. trends or step changes) and detection and discussion of any outliers. Each gauge in the appendix is represented by a summary sheet showing a plot of the annual maximum flow series, analysis of trends and seasonality, flood frequency analysis (where the record is long enough) and summary statistics for the largest floods. Appendix B also includes an analysis of flood volume data at one gauge, Ballygaddy.

The longest record on the River Clare, at Corrofin, showed no evidence of long-term trend in flood peaks. This is consistent with the findings of Cawley and Cunnane (2010)[10].

On the River Clare/Dalgan at Ballyhaunis, Ballygaddy and Claregalway, the highest flood on record was November 2009. It was not outstandingly higher than other events. The AEP for November 2009, estimated solely from analysis of at-site flood peak data, is 2% at Ballyhaunis and 1% at Ballygaddy. The AEP for Ballygaddy is similar to that presented in Ryan Hanley (2010)[11] and Cawley and Cunnane (2010).

However, on the Clare at Corrofin, where the flood peak record is longest (extending back to 1964), the 2009 flood is slightly smaller than the event of November 1968. There is some uncertainty as to the accuracy of this level as it was queried in a note written on the water level recorder chart (Cawley and Cunnane, 2010). The growth factor for the 1968 flood (flow divided by the median annual maximum flood QMED) is 2.4, compared to a growth factor of 2.3 for the 2009 event.

On the River Corrib at Galway (where a flow value for the 2009 flood cannot be calculated – see Table 3.1), the highest flood on record was in January 1975.

Appendix B includes analysis of peak flows on the Owenriff at Oughterard and Claremount. During the inception phase it was noted that the Owenriff at Claremount should be treated with great caution. A rating has been derived in order to refine the recorded AMAX series at the location, however uncertainty in the rating remains high.

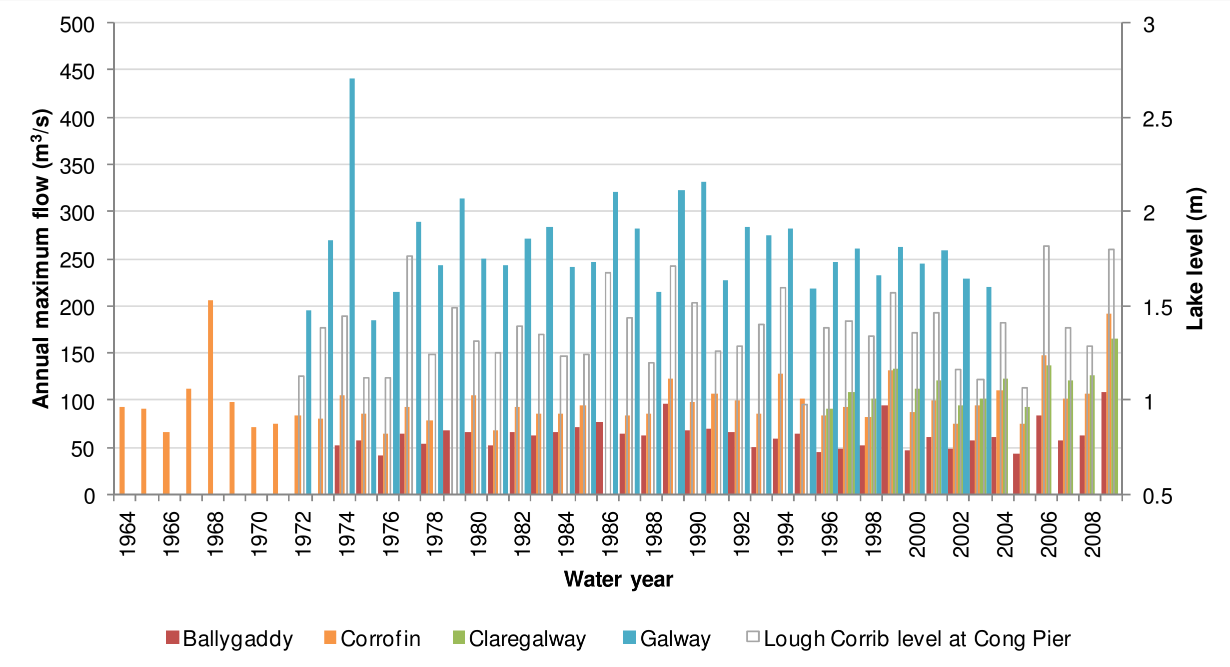
Most stations show a distinct flood seasonality, with floods generally occurring in October to April. At Galway, the onset of the flood season is rather later, in late November, presumably due to the lag time and storage available in Lough Corrib.

A comparison of flood peak series at several gauges, including a level gauge on Lough Corrib, is shown in Figure 5-1. The stations are, from the upstream end of the River Clare, Ballygaddy (30007), Corrofin (30004), Claregalway (30012) and then, downstream of Lough Corrib, Wolfe Tone Bridge (30061). It can be seen that peak flows on the Corrib are very much higher than on the Clare. It appears that the large increase in catchment area from Claregalway to Galway, along with the inclusion of steep, high-rainfall subcatchments, more than compensates for the attenuation effect of Lough Corrib.

[10] Cawley, A. and Cunnane, C. (2010) 02 - Comment on the November 2009 Flooding in the Shannon and Corrib Systems. Irish National Hydrology Conference 2010.

[11] Ryan Hanley (2010) Study To Identify Practical Measures To Address Flooding On The Clare River. Report for OPW.

Figure 5-1: Flood peak series at gauges in UoM 30



5.2 Analysis of longer-term flood history

Information on the impacts of both recent floods and events that pre-date the gauged records was collected from the sources listed in Section 3.5. The information was reviewed in order to provide relevant qualitative and, where possible, also quantitative information on the longer-term flood history in the area. For earlier flood events, the information was often limited to only a brief notion about flooding occurring at various locations, however, in some cases it was possible to detect the extent or even magnitude. These include comments such as "River Clare has expanded ten times its normal width", "the worst storm in 40 years" or "flooding worst in 102 years" (references can be found in Appendix C).

A chronology of flood events is given in Appendix C, along with a visual time-line which summarises the findings in terms of relative magnitudes of different events, as assessed from both gauged data and the historical review.

The longest flood peak dataset is for the Clare at Corrofin. The gauged record dates back to 1964, but reliable flood peak data are available only for 1972 onwards (see discussion in Appendix B). The historic review identified a large flood in November 1968. The gauged data indicated that the 1968 flood was the highest on record, but it occurred before the start of reliable flood peak data and so its magnitude relative to more recent events such as November 2009 is not known. The Connacht Tribune described the 1968 flood as the worst in North Galway for over 45 years. It is difficult to incorporate this information in the flood frequency analysis given the lack of a reliable way of estimating a flow for the 1968 flood.

5.3 Overview of method for flood peak estimation

At all HEPs in UoM 30, design peak flows for return periods up to 100 years have been estimated using the Flood Studies Update (FSU) method as described in research reports produced from FSU WPs 2.2 and 2.3.

The locations and catchment boundaries of HEPs are included as ArcGIS shapefiles within the digital deliverables from the Western CFRAM project, Section 12.

Because FSU methods are not fully released for general use at the time of writing, it was necessary to make some decisions about how to apply the methods presented in the reports, and to develop software to enable application of the methods. The sections below set out how the FSU methods have been applied. They have been implemented using JBA's web-based flood estimation

software, JFes, in combination with the package WINFAP-FEH which has been applied to produce single-site flood growth curves.

The FSU method for estimation of peak flows is an index flood method, involving two stages. The index flood can be thought of as a typically-sized flood for a particular catchment, and in the FSU it is defined as the flood with a 50% probability of being exceeded in a particular year. This is equivalent to the median of the annual maximum flood series, denoted QMED. The first stage of the method involves estimating QMED, and in the second stage a flood growth curve is estimated. The growth curve is a dimensionless version of the flood frequency curve which defines how the flood magnitude grows as the probability reduces, i.e. for more extreme design floods. The design flood for a particular exceedance probability is then simply calculated as the product of QMED and the value of the growth curve for that probability (known as the growth factor).

The sections below provide more detail on how each step was approached.

5.4 Estimation of QMED

The most reliable estimates of QMED are obtained directly from suitable quality flood peak data, as the median of the annual maximum series. At locations without high flow data, QMED can be estimated, with lower confidence, using a regression equation based on seven different physical catchment descriptors, in conjunction with an urban adjustment, developed in FSU WP 2.3. It is often possible to improve on this initial estimate of QMED by refining it using the process of data transfer, in which a representative gauged catchment with suitable quality data is identified and an adjustment factor for QMED calculated as the ratio of the gauged to the ungauged estimate of QMED at the gauging station. This factor is then used to adjust the initial estimate of QMED at the ungauged site, under the assumption that the factorial error in the QMED regression model is similar for two catchments. In the terminology of the FSU research reports, the gauging station where the adjustment factor is calculated is referred to as a donor site. The term pivotal site can also be used.

Some guidance on identifying suitable donor sites is given in FSU WPs 2.2 and 2.3. The WP 2.2 research compared various ways of adjusting QMED and found that the best was to select the next gauging station downstream as a donor (if available). Selecting the closest upstream gauge was also found to perform well. Selecting a more distant gauge that is similar in terms of catchment properties was found to perform less well. The report on WP 2.3 emphasises the value of locally-informed hydrological experience in selecting donors, and recommends taking into account several factors including the degree of similarity of the subject and donor catchments, the quality of the gauged estimate of QMED and the possibility of choosing multiple donors in some cases.

For the Western CFRAM, donors have been chosen according to the following general approach:

- Where there is a gauging station on the same river as the subject site, with a comparable catchment area (up to several times larger or smaller) and no major change in physical characteristics, it has been selected as a donor.
- Where there are gauging stations upstream and downstream of the subject site, in general the adjustment factor has been calculated as a weighted average of the factor at each gauge. Weights are based on area, with more weight given to the gauge whose area is more similar to that at the subject site. Exceptions to this include situations where the downstream gauge lies below a major lough, in which case it has not been used to calculate adjustment factors for locations upstream of the lough. An example of this calculation is given below:

$$Weighted\ adjustment\ factor = \left(\frac{DS\ area - HEP\ area}{DS\ area - US\ area} \times US\ Adj\right) + \left(\frac{HEP\ area - US\ area}{DS\ area - US\ area} \times DS\ Adj\right)$$

Where
DS area = Catchment area of downstream gauge (km^2)
US area = Catchment area of upstream gauge (km^2)
HEP area = Catchment area at HEP (km^2)
DS Adj = QMED adjustment factor at downstream gauge
US Adj = QMED adjustment factor at upstream gauge

- If neither of the above apply, for example if there is no gauging station on the river or the closest gauge is a long way downstream with a catchment many times larger, then a gauging station on a nearby catchment whose characteristics (area, slope, rainfall, lough influence) are similar to those of the subject site has been chosen as a donor.
- If none of the above apply, which is often the case for subject sites on very small catchments, no donor site has been chosen and QMED has been estimated solely from catchment descriptors.

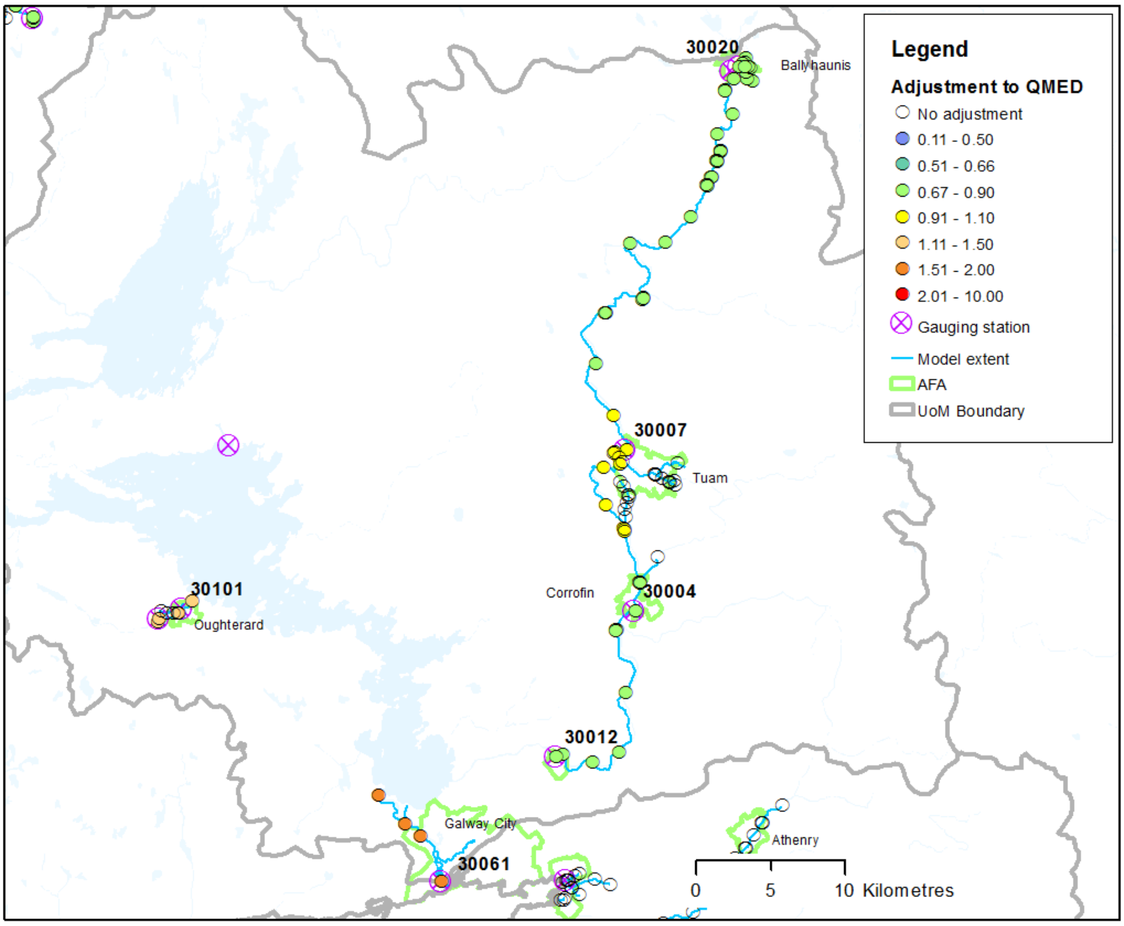
For any subject sites that are located at gauging stations, QMED has been estimated directly from the flood peak data. At gauges that were included in the Western CFRAM rating review process, QMED was estimated from the newly created annual maximum series, calculated from the revised rating. Elsewhere, the original annual maximum series supplied by OPW or EPA were used. The original and revised flood peak series can be seen in Appendix B.

Figure 5-2 shows the adjustment factors for QMED both at the gauging stations (i.e. QMED from flood peak data divided by QMED from catchment descriptors) and at all the ungauged HEPs. Most gauges in UoM 30 show only moderate QMED adjustments, generally in the range 0.67 to 1.10.

Adjustment factor ratio for gauge 30007 (Ballygaddy) is particularly high (0.908) as the gauged data and catchment descriptor estimates of QMED are similar in magnitude. Gauge 30004 downstream also has a relatively high adjustment factor, 0.883. The HEPs along this reach use a weighted adjustment of these factors, giving relatively high values along the Clare/Dalgan Rivers compared to up and downstream where the adjustment factors are in the range of .07-0.8.

On the River Corrib through Galway City, QMED from flood peak data is substantially higher than that predicted from catchment descriptors, resulting in an adjustment factor of 1.56. It is likely that the estimates from catchment descriptors exaggerate the attenuation of flows through Lough Corrib; the gauged data at Wolfe Tone Bridge (30061) provide a more accurate estimate of QMED.

Figure 5-2: Adjustment factors for QMED for gauges and hydrological estimation points



At ungauged locations, adjustment factors are calculated either from nearby donor gauging stations (chosen using the approach outlined above) or set to 1, i.e. no adjustment, where no suitable donors could be found.

A record of the adjustment factor applied at each HEP is provided in Appendix F.

5.5 Estimation of growth curves

Using the FSU approach, flood growth curves can be derived from analysis of annual maximum flows either at the site of interest (single-site analysis) or at a group of gauging stations chosen from a wide area (pooled analysis).

5.5.1 Sites suitable for single-site analysis

Single-site analysis uses annual maximum flows solely at the gauge of interest to estimate flood growth curves. It was carried out at all gauging stations included in the flood peak analysis (Appendix B).

Single-site estimates are typically avoided as they are vulnerable to the length and quality of peak flow data. Where the AMAX record length exceeds two times the return period, single-site estimates are deemed representative of the observed data. This record length is rarely achieved, particularly for higher return period estimates, therefore some weight can be given to single-site estimates if the record length is between one and two times the return period. Appendix B includes further consideration of the quality of the flood peak data, flood history and unusual catchment characteristics that may reduce confidence in pooled growth curves to ensure that the most representative growth factors were applied at each gauged location.

In UoM 30 single-site growth curves were deemed the most representative of the gauged catchment at Corrofin (30004) and Claregalway (30012). The single-site growth curves were applied at these gauges and nearby ungauged locations where appropriate. The application of growth curves to ungauged sites is discussed further in Section 5.5.6 below.

5.5.2 Selection of pooling groups

For pooled analysis, gauges are chosen on the basis of their similarity with the subject catchment according to three catchment descriptors, AREA, SAAR and BFIsoil. The report on FSU WP 2.2 presents two alternative equations for calculating the similarity of catchments according to these three descriptors. For the CFRAM, equal weight was given to each of these variables, applying the similarity distance formula given as Equation 10.2 in the report on FSU WP 2.2.

Not all gauges in Ireland were considered for use in pooling, because the analysis required to fit a flood growth curve makes use of the magnitude of each annual maximum flow, and thus it is necessary that even the highest flows are reliably measured. This excludes gauges where there is significant uncertainty in the high flow rating. The following gauges were considered as candidates for forming pooling groups:

- Gauges that were included in the Western CFRAM rating review process, where this led to a confident re-assessment of the rating, or to fitting of a new rating (13 gauges).
- Other gauges from the Western CFRAM area or elsewhere throughout the Republic of Ireland that are classed as A1 or A2 standard in the FSU dataset. This is the set of gauges that was used to develop the methods in FSU WP 2.2). OPW provided updated annual maximum series for their FSU gauges in March 2013 (91 of which are classed A1 or A2), containing data up to water year 2009-10. 28 additional gauges operated by EPA are classed as A1 or A2, and flood peak series for these have not been updated since the FSU research, so end in water year 2004-5.
- Gauges from Northern Ireland that are classed as suitable for pooling in the current version of the HiFlows-UK dataset (version 3.1.2, which contains data up to water year 2008-09) (37 more gauges).

The total number of gauges in the pooling dataset, allowing for some overlaps between the above categories, is 166.

The inclusion of gauges from Northern Ireland is beyond the work that was carried out for the FSU research. Adding these gauges increases the likelihood of finding similar catchments to form pooling groups, particularly for small catchments for which there is a shortage of gauged data in the Irish Republic. The fact that some parts of the Western CFRAM area are adjacent to catchments in Northern Ireland adds weight to the argument for including data from the North. In addition, research (Molloy, 2011)[12] has shown that there is no observable difference between the forms of flood frequency distribution followed by the annual maximum flood datasets of Northern Ireland and the Republic of Ireland and so it can be expected that data from Northern Ireland will be a useful addition to any pooled analysis.

One assumption has been made to enable the inclusion of Northern Irish data, that the catchment descriptor BFIHOST (used in the UK) can be considered equivalent to BFIsoil. Although the two descriptors are calculated from different datasets, they are both intended to measure the same quantity, i.e. the baseflow index, which is a measure of the proportion of the annual flow hydrograph that derives from storage in the catchment.

FSU WP 2.2 recommends creating pooling groups that contain 5T years of data in total, where T is the return period of interest. As advised in WP 2.2, and to avoid possible contradictions between growth curves for different AEPs, a single pooling group has been chosen for each location, based on an AEP of 1% which has been defined as the principal AEP of interest. This equates to a return period of 100 years, and thus each pooling group contains just over 500 years of data.

No alterations were made to the pooling groups derived using the process described above as the gauging stations had already been screened according to the quality of their flood peak data.

[12] Molloy, James (2011). A Comparison of the Stochastic Flood Hydrology of the North and Republic of Ireland. Unpublished MSc thesis, NUI Galway. Also presented as a poster at the National Hydrology Conference, 2012.

Although there is some evidence from research on UK data[13] that flood growth curves are affected by additional catchment descriptors such as FARL, the FSU research found that FARL was not a useful variable for selection of pooling groups (uncertainty was greater when FARL was included than when it was excluded) and therefore no attempt was made to allow for the presence of lakes in the composition of pooling groups. Similarly, no allowance was made for arterial drainage in selecting pooling groups.

The contents of each pooling group created at the site of gauging stations are listed in Appendix B. Where suitable flood peak data are available at the gauge, it is listed as the top-ranking gauge in the pooling group. Most groups can be seen to contain gauges from a wide range of locations across Ireland, although there are few from the east coast, where the annual rainfall is low enough to exclude most gauged catchments from pooling groups created using characteristics of catchments in the Western RBD. There are few gauges from Northern Ireland in most groups; the exceptions being groups created for the smallest catchments.

5.5.3 Selection of statistical distribution

FSU WP 2.2 recommends considering two parameter distributions for single-site growth curves, either the extreme value type 1 (EV1, known as the Gumbel) or the 2-parameter log-normal distribution (LN2). Restricting the number of parameters to two helps reduce the standard error of the fitted distribution, albeit at a cost of a potential greater bias compared with 3-parameter distributions. In this assessment both distributions have been fitted, and the goodness-of-fit assessed visually.

For pooled growth curves, WP 2.2 recommends considering 3-parameter distributions, because the extra data provided by the pooling group ensures that the standard error is lower than it would be for single-site analysis. The report states that either the generalised extreme value (GEV) or generalised logistic (GL) distributions are worth considering. In this assessment both have been fitted for each pooled analysis. In general the GL distribution results in a growth curve that is more skewed, i.e. it may give similar or lower growth rates to the GEV for moderate probabilities, but it has a stronger upwards curvature which results in a steeper growth curve for low-probability floods. Molloy (2011) found that the GL distribution gave a better fit than the GEV for the vast majority of pooling groups in both the Republic and Northern Ireland. For the present study, the choice of recommended distribution has been made on the basis of visual inspection of plots comparing pooled growth curves with plotted flood peak data at gauging stations. In most cases, the GL distribution has been preferred as it appears more consistent with at-site flood peak data and is less likely to underestimate design flows for low probabilities.

5.5.4 Fitting growth curves

Both single-site and pooled flood growth curves have been fitted using the method of L-moments, as recommended in the FSU research. To calculate the pooled curve, the L-moments for each gauge in the pooling group have been weighted according to the record length of the gauge. This ensures that more weight is given to longer records, which provide more reliable estimates of the underlying flood frequency distribution.

5.5.5 Choice between single-site and pooled growth curves

Initially, both single-site and pooled growth curves were fitted at all 26 gauging stations on watercourses to be modelled for the Western CFRAM where there are at least five years of reliable flood peak data. The resulting growth curves for gauges in UOM 30 can be seen in Appendix B. The graphs show the annual maximum flows for each gauge and both the single-site and pooled growth curves. The horizontal axis shows return period rather than AEP because the software (WINFAP-FEH) does not provide the option to plot AEP.

[13] Kjeldsen, T.R., Jones, D.A. and Bayliss, A.C. (2008) Improving the FEH statistical procedures for flood frequency estimation. Science Report SC050050, Environment Agency.

At each gauge a preferred growth curve has been selected. There is a large amount of guidance available on the choice between single-site and pooled growth curves, including FSU WP 2.2, Gaume (2006)[14] and Environment Agency (2012)[15]. Factors that have been considered include:

- The length of the flood peak dataset at the gauge;
- The quality of the rating curve for measurement of high flows;
- The degree to which the catchment is unusual and therefore likely to be less well represented by other catchments in the pooling group;
- Information available from longer-term flood history, including quantitative data such as longer flow datasets at nearby gauges and more qualitative data from reports of earlier floods;
- The degree to which the curves fit the plotted flood peak data, bearing in mind the uncertainty of the plotting positions used to control where the data displays on the return period axis.
- The implied exceedance probabilities for the highest floods on record according to each distribution, and whether these are likely given what is known of the impact of the floods.

As an example of this last point, if the pooled growth curve is much less steep than the single-site curve, it might imply that the highest couple of floods recorded at the site both have annual probabilities lower than 1%. While this is theoretically possible it is highly unlikely, and a more likely explanation would be that the pooled growth curve underestimates the true growth curve for the catchment in question.

At the other extreme, a pooled curve that is much steeper than the single-site curve would imply high probabilities for the top few floods on record. It is possible to calculate the statistical likelihood of these probabilities being correct. For example, how likely is it that a 30-year long record contains no flood exceeding a 10% annual probability (10-year return period)? This question can be answered by calculating the probability of no exceedances in any 1 year (0.9) and then raising 0.9 to the power of 30 to calculate the probability of no exceedances in 30 years, which works out as 0.04, i.e. it is very unlikely that there will be no exceedances. To answer the question for a number of exceedances greater than zero, the binomial theorem can be applied.

Such calculations are considered in the discussions in Appendix B to help decide whether pooled growth curves are realistic in some cases where they differ markedly from the plotted flood peak data.

In some cases, as noted in FSU WP 2.2, it may be appropriate to use a combination of a single-site and pooled growth curve. This approach is applied widely in the UK using the current FEH methods (Kjeldsen et al., 2008)16. For all but one of the gauges analysed in the Western CFRAM it was found possible to make a choice between the single-site and pooled growth curves without needing to create a compromise between the two.

In some cases, the choice was straightforward as there was little difference between the single-site and pooled curves. This was the case at Ballygaddy and Oughterard. At Ballyhaunis, the short record length meant a pooled approach was preferred. The single site curve is recommended at just one gauge, Corrofin. At Claregalway it is recommended that the single-site curve from the upstream gauge, Corrofin, is applied to promote consistency in the design flows.

Overall, across the Western CFRAM area, the pooled growth curve has been recommended at a little over two thirds of the gauging stations.

5.5.6 Growth curves for ungauged sites

The standard FSU approach is to develop growth curves for ungauged sites using pooled analysis. This has been applied at the majority of sites, with an individual pooling group created for each

[14] Gaume, E. (2006) On the asymptotic behaviour of flood peak distributions. Hydrol. Earth Syst. Sci, **10**, 233-243.
[15] Environment Agency (2012) Flood estimation guidelines. Operational instruction 197_08, issued June 2012.

[16] Kjeldsen, T.R., Jones, D.A. and Bayliss, A.C. (2008) Improving the FEH statistical procedures for flood frequency estimation. Science Report SC050050, Environment Agency

site. Both GL and GEV growth curves have been fitted, for comparison. There is moderate variation in the pooled growth curves across the Western CFRAM study area:

- The 1% AEP growth factor from the GEV ranges from 1.56 to 2.52 with a mean of 1.96.
- The 1% AEP growth factor from the GL ranges from 1.63 to 2.60 with a mean of 2.04.

As is often the case, the GL gives slightly higher growth factors for low AEPs as it tends to have greater skewness than the GEV.

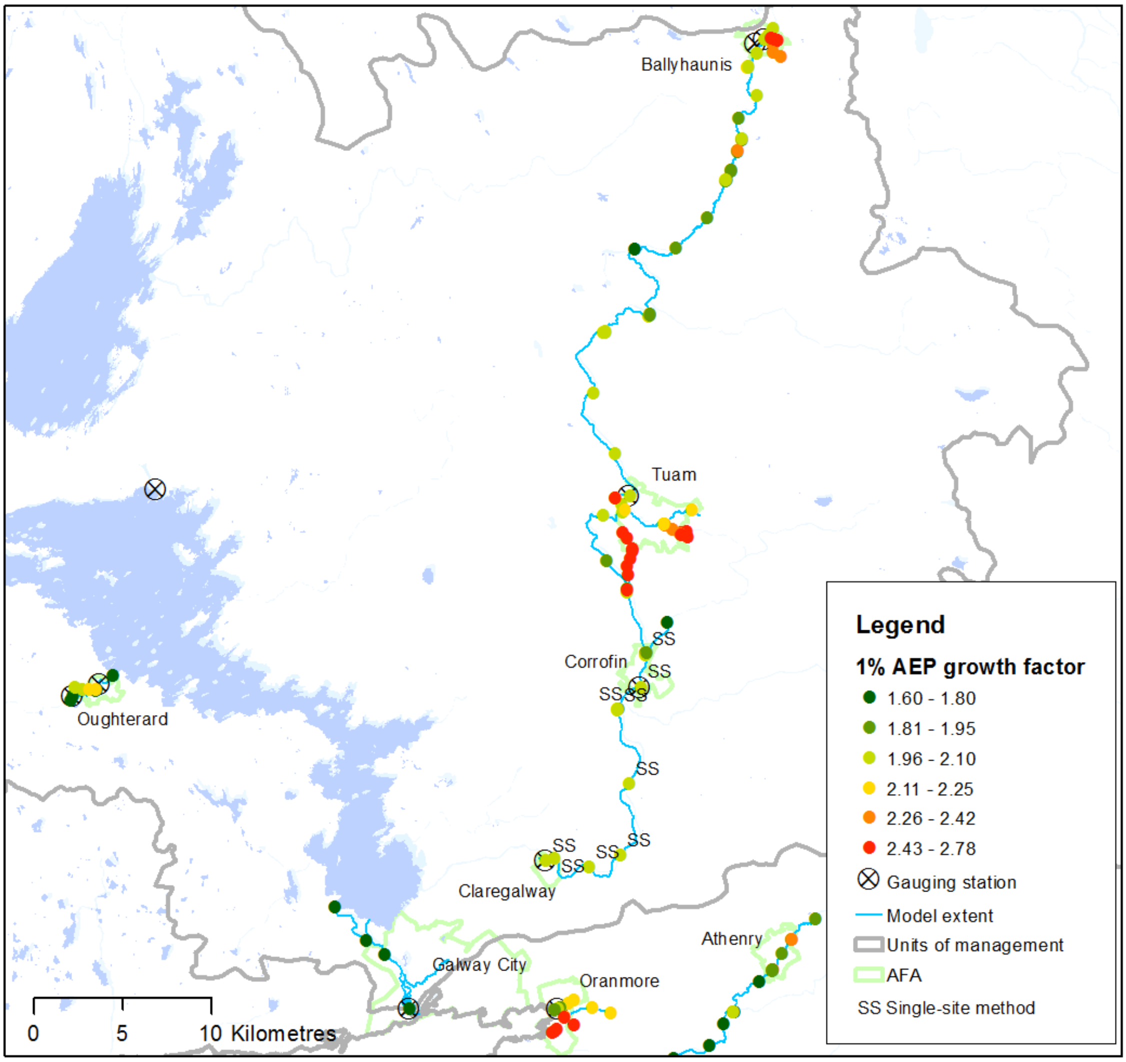
Given that the GL was judged to be the preferred growth curve at most gauging stations where pooled analysis was chosen, it was decided to adopt the GL for all ungauged locations too, apart from on watercourses with gauging stations where the GEV was chosen. As can be seen from the results in the above bullet points, the effect on the results if the GEV had been adopted would have been a reduction of the 1% AEP flow estimate by 4% on average.

For sites on watercourses where there is a gauging station nearby at which the single-site curve is preferred, it is not appropriate to use a pooled growth curve as this may result in a sudden jump in the growth factor, leading to spatial inconsistency in the design flows. For this reason, single-site growth curves have been selected in such situations. Judgment has been used in deciding how far away from each gauging station the single-site curve should be applied, before reverting to the pooled curve.

A record of the type of growth curve and the distribution applied at each HEP is provided in Appendix F.

Figure 5-3 shows the resulting growth factors for an AEP of 1%, i.e. the ratio of the 1% AEP flood to QMED. The major rivers (Clare, Clarinbridge and Corrib) show low growth factors in the range of 1.6 to 2.09, due to floodplain attenuation and storage in larger water bodies. The smaller tributary catchments have steeper growth curves, represented by 1% AEP growth factors up to 2.78 (for example, in Tuam). These growth factors can be compared with the Flood Studies Report (FSR) regional growth curve for Ireland, which has been superseded by the FSU methods. The FSR curve gives a ratio of 2.06 when dividing the 100-year return period factor by the 2-year return period. The newer FSU method allows the flood growth curve to reflect the characteristics of individual catchments, rather than imposing a uniform curve.

Figure 5-3: Growth factors for the 1% AEP flood



5.5.7 Extension of growth curves to the 1000-year return period (0.1% AEP)

After reviewing the flood outlines produced by model runs which used the first iteration of design flows, some revisions to design flows were made in order to ensure flood levels and extents were not underestimated for the most extreme events. The initial flood outlines showed little out-of-bank flow in some areas even for the 1000-year flood, which was considered unlikely to be realistic. The revisions included applying the FSR rainfall-runoff method to estimate the gradient of the upper portion of the growth curve, for return periods in excess of 100 years.

The reasons for favouring the rainfall-runoff method over the FSU curve are that rainfall growth curves can generally be treated with more confidence than flood growth curves (owing to longer records, greater spatial consistency and fewer problems with data quality) and that adopting this method avoids the extremely low gradient growth curves that were derived at some HEPs using the FSU methods. At some HEPs, the 1000-year flood was initially estimated to be as little as 13% greater than the 100-year flood. While there is no firm evidence on which to base estimates of floods as extreme as the 1000-year return period, this small growth rate was considered to be unrealistic. The corresponding percentages estimated from the FSR rainfall-runoff method did not fall below 44% (i.e. the 1000-year flood was at least 1.44 times greater than the 100-year flood).

In UK practice it is also common to see occasional very low rates of growth from 100-year to 1000-year floods, and a widespread approach is to derive the upper part of the flood growth curve from

an alternative method, usually the ReFH rainfall-runoff method. Environment Agency guidelines17 advocate this approach, and selection of the 100-year return period as a pivot point is near-ubiquitous in the UK.

The extension of the growth curves was carried out by using the FSR rainfall-runoff method to estimate the ratios of the 200-year to 100-year and 1000-year to 100-year floods. These were then multiplied by the estimate of the 100-year flood given by the FSU methods described above. The FSR estimates were derived using FSR rather than FSU design rainfall since the FSU rainfall statistics are not intended for extrapolation up to the 1000-year return period.

It was not necessary to apply all aspects of the rainfall-runoff method to calculate the required ratios. The gradient of the flood growth curve depends on two principal factors: the gradient of the FSR rainfall growth curve and the way in which the percentage runoff increases with rainfall magnitude as a result of the DPRrain term in the FSR calculation of percentage runoff. A simplified calculation was carried out, with a single value of the FSR rainfall parameters M5-2 day and Jenkinson's r applied to all catchments within a given UoM. The main variations in the gradient of the growth curve were due to the soil type, which was evaluated individually for each HEP from a digitised version of the FSR soils (WRAP) map.

A consequence of this adjustment is that the upper portion of the final CFRAM growth curves is steeper in areas with low SPR, i.e. more permeable soils. This is in accordance with expectations that permeable catchments, including karst areas, may occasionally experience particularly extreme floods during events which cause the catchment processes to switch to those associated with more impermeable catchments, perhaps due to filling of upstream storage in turloughs, caves and other karst features.

5.6 Final design flows

Design flows for each AEP and at each HEP have been calculated by multiplying the estimates of QMED by the appropriate growth factor, and by application of FSR rainfall-runoff ratios for the 0.2% and 0.1% AEP events.

The flows are supplied in Appendix F and also digitally in the form of a shapefile and a spreadsheet.

A summary of the methods used for estimating the design flows for each AFA in UoM 30 can be found in Chapter 7.

The final design flows have been used as inflows to the hydraulic models, in a process which is described in the relevant AFA Hydraulic Modelling Report.

5.7 Checks on the design flows

5.7.1 Calibration, validation and checking

The brief for CFRAM studies requires the consultant to "calibrate and validate the estimates of the design flood parameters … to recorded data as far as reasonably possible, based on historic or recorded flood event data."

The design flows have been derived by direct analysis of flood data, as far as its availability and quality permit, so they will naturally be consistent with that data. Flood data has been used to estimate QMED at gauges, to adjust QMED at ungauged sites, to fit growth curves, to decide between single-site and pooled growth curves, to estimate time to peak for the rainfall-runoff method and to derive average hydrograph shapes.

However, it cannot be claimed that the design flows have been calibrated or validated because, while measurements of river level and flow are feasible, there is no way of measuring the probability of floods. Thus there is no meaningful way of calibrating design flows against observations, unlike say calibration of a hydrological or hydraulic model in which model results can be compared against modelled flows or levels. Any so-called calibration of design flows would give a spurious impression of confidence in what are statistical estimates. Validation of resulting

[17] Environment Agency (2012) Flood Estimation Guidelines.

flood extents for various return periods has been undertaken as part of the hydraulic modelling work.

In addition, design flows have been checked using a number of tests intended to identify any results that fall outside expected ranges or are inconsistent with other results. The tests have included:

- Checks that growth factors are within expected ranges. The range of 1% AEP growth factors from the pooled analysis is 1.63 to 2.78. None of these values are unexpectedly high or low. This range can be compared with the equivalent factor taken from the FSR regional growth curve for Ireland: the factor for the 1% AEP divided by that for the 50% AEP gives a ratio of 2.06. Some of the single-site growth curves that are preferred over pooled curves have more extreme growth factors, as discussed in Appendix B.
- Checks on the AEPs for observed events that are implied by the derived flood frequency curves at gauging stations. The findings are described in Appendix B.
- Checks for spatial consistency between design flows at different locations. These are described below.

5.7.2 Checks for spatial consistency

Spatial consistency, or coherence, is an expected characteristic of design flow estimates throughout a catchment, reflecting the behaviour of the physical system. Estimates should vary gradually along the length of a watercourse unless there are features that reduce or increase the rate at which water is routed through the catchment, potentially causing a step change in flow.

Design flows can be deemed spatially consistent if they gradually increase downstream, with step changes only at confluences or decreases in the downstream direction where a physical cause can be attributed. It is therefore expected that peak flow estimates downstream of a confluence should be consistent with those of the tributary inflows, with:

Highest tributary flow estimate < Downstream flow estimate < Sum of peaks on tributaries

Given the variability in catchment characteristics and thus the timing and magnitude of peak flows, no fixed relationship can be given between the downstream flow estimate and those of the tributaries. It is therefore necessary to examine the modelled watercourses in turn to ensure that flows are consistent between confluences and that the above condition is met at confluences. If it is not, reasons should be determined for the inconsistency which can be taken into account during the modelling process.

Following the methodology outlined above for estimating the design flows, there is a fine balance between applying various methods between HEPs to account for local data and ensuring consistency between HEPs where different methods have been used. Various approaches have been incorporated into the study, such as applying weighted adjustment factors for QMED, using pooling groups and checking catchment descriptors to derive the most robust estimates throughout the catchment. Incoherence is possible where the chosen method changes between HEPs. Checks of both the physical causes for apparent incoherence and step changes as a result of the methodology are therefore particularly important to verify that realistic flow estimates are incorporated into the hydraulic models, and so detailed consideration of inconsistencies is discussed in each of the relevant hydraulic modelling reports.

The approaches in Section 5.4 describe the use of donor gauging stations, adjustment factors, weighted factors and catchment descriptors to estimate QMED. As these methods have been applied to various reaches, it is possible that changes in the adjustment factor for QMED, growth factors (in the case of HEPs where a pooled approach has been used) and direct estimates of QMED from catchment descriptors, may not be spatially coherent. Step changes in the flows were related back to each of these calculation stages where necessary.

Checks were made of the following at both the 50% and 1% AEP for AFAs and HEPs on all modelled watercourses:

- Consistency in flow estimates downstream
- Consistency at confluences
- Consistency with gauged data (where available)

- Consistency in flows between return periods.

Where spatial incoherence was apparent, catchment descriptors were reviewed for physical reasons for the flow estimate. Apparent spatial inconsistencies were found in some instances, typically for HEPs with small areas derived solely from catchment descriptors. These have been reviewed and can be explained by changes in the physical catchment downstream or large differences in catchment parameters between tributaries. The key observations and their potential causes have been summarised in Table 5-1 below.

Table 5-1 Reasons for apparent spatial inconsistencies

Observation	Potential Cause
Downstream flow estimate is less than the greater of the two tributaries	Occurs where the change in Area is outweighed by more extreme changes in other catchment descriptors. For example, where the influence of a lake, floodplain characteristics or extreme differences in rainfall characteristics on an incoming tributary affects downstream catchment descriptors such that there is a reduction in QMED, a change in pooling group members or both.
Downstream flow estimate is greater than the sum of the two tributaries	FSU QMED equation exacerbating extreme catchment descriptors downstream of confluence – typically where tributary catchments are considerably different in character (particularly BFIsoils/FARL)
Decrease in flow downstream – mid reach	Floodwaters spreading out into the floodplain or loughs between HEPs. Impermeable headwaters from soil characteristics or urban extent resulting in flow attenuation downstream. Increased runoff rates to the upstream HEPs due to impermeable soils may exacerbate flows. If the catchment becomes more permeable downstream, the increased area may not outweigh the increased infiltration and flows may decrease in a downstream direction.

Some of these apparent inconsistencies can be explained by a physical cause and therefore should be represented within the hydraulic model. It is also possible, particularly when QMED is estimated solely from catchment descriptors, that the influence of these physical changes is exacerbated by the FSU equation. In these cases, the HEPs should be used to derive the general flow patterns downstream which should be replicated by the model, but the peak flows derived for each HEP may not be matched exactly. In areas where the flood risk is high (for example, due to the presence of properties) it is recommended that flows are adopted that represent a conservative estimate of risk by applying the larger of the HEP design flows at the downstream location.

Inconsistencies in design flows may also arise from changes in method used within a catchment. Particular attention has been paid throughout the design estimate calculation to checking the consistency of the following:

- Adjustment factor for QMED downstream and at confluences
- Changes in pooling group and growth factors
- Consistency between HEPs where the method of using pooled or single site analysis changes.

The following examples describe the locations where these inconsistencies are most likely to occur:

Table 5-2: Locations where inconsistency locations

Cause
QMED adjustment factor differs significantly between upstream and downstream of a confluence – typically a result of changing catchment descriptors at the confluence
QMED adjustment factor is particularly large and no weighted adjustment is applied
Change in pooling group downstream reducing growth factors at a HEP
Inconsistency in QMED estimate as a result of change between pooled and single-site growth curves

Where the applied methodology appears to derive inconsistent flow estimates at HEPs, checks have been undertaken to ensure the calculations are correct. Consistent results are produced by each individual method however inconsistencies may arise where the method changes along a watercourse. The choice of methodology has followed a detailed examination of the flow characteristics for each reach and therefore in cases where such inconsistencies arise the flow estimates should be interpreted during the modelling stage as follows:

- If the HEP is located upstream, in the vicinity of an urban area, flows should be used which represent a conservative estimate of flood risk. For example, the greater of the tributary inflows should be applied downstream of the confluence in the case of a decrease in the flow downstream.
- If the HEPs upstream of a confluence represent two catchments of significantly different catchment characteristics, the tributary inflows should be treated with more confidence than the downstream flow estimate.
- Where step changes occur as a result of a change in methodology, the greater of the estimates should be applied. A weighted approach to the derivation of growth factors has been applied along certain reaches to minimise such step changes.

The final design flows derived for the HEPs reflect both the physical catchment and the methodology used to extrapolate QMED to estimate events of larger magnitude. There are a few instances where, due to the reasons listed above, design flows are not spatially consistent. Consideration will be given during the modelling process to these locations, matching the derived values where possible, but allowing for deviations where modelling judgment chooses to favour particular HEP estimates. This may include, but is not exclusive to the three examples listed above. Further details regarding these decisions will be included in the reporting of the modelling methodology.

6 Estimation of hydrograph shapes

6.1 Overview of approach to hydrograph generation

For the vast majority of rivers in the Western CFRAM, design flows have been derived using the FSU methods to estimate peak flows by statistical analysis. At locations where inflows to hydraulic models are needed, it is necessary to provide a hydrograph shape for combination with the estimated peak flows.

When setting inflows to hydraulic models it is important to create a set of inflows from the various tributaries that are consistent in terms of their magnitude, timing and duration, so that the hydrographs combine in a realistic way at confluences.

The FSU includes a set of methods (published in FSU WP 3.1) for creating normalised hydrograph shapes (referred to as *characteristic flood hydrographs*) on gauged and ungauged catchments. For gauged catchments, characteristic flood hydrographs can be created by averaging the widths of observed hydrographs, referred to as a Hydrograph Width Analysis (HWA). For ungauged catchments, the FSU method allows characteristic flood hydrographs to be produced using a mathematical function whose parameters can be estimated from catchment descriptors. These methods are intended for use at individual locations and do not provide any information on the relative timings of hydrographs at confluences. A technique for estimating the relative timings of inflows was developed in FSU WP 3.4, in which the time difference between the two peaks is estimated from a regression model using differences in the descriptors of the two confluent catchments.

An alternative approach to creating hydrograph shapes is the older Flood Studies Report (FSR) rainfall-runoff method, in which design flood hydrographs are created from a design rain storm in conjunction with a unit hydrograph whose time to peak can be estimated either from local hydrometric data or from catchment characteristics. The hydrograph can be scaled to match a preferred peak flow, for example estimated using FSU methods. An advantage of the FSR method is that all hydrographs for the various inflows to a model can be created from the same design rain storm, thus imposing a realistic structure in terms of duration and timing of the inflows.

Both the FSU and FSR methods have been tested, as discussed in the following sections. The results have been compared at a number of sites in order to select a preferred approach. For some of the largest rivers, a frequency analysis of flood volumes was carried out. The results have been used as a check on the volumes calculated from the hydrograph shapes when combined with the design peak flows.

The tests described in the sections below cover sites throughout the Western RBD as their aim was to provide information to assist the choice between alternative methods. The methods that were selected for individual AFAs in UoM 30 are summarised in Chapter 7.

6.2 Implementation of FSU hydrograph method

At gauging stations that are near either AFAs or upstream limits of hydraulic model reaches, characteristic flood hydrographs were created by taking the median widths of large numbers of normalised observed hydrographs. A characteristic hydrograph shape was created by fitting a combination of a gamma function and an exponential curve, the latter defining the recession portion of the hydrograph, to the median hydrograph widths. The analysis was carried out using the HWA software developed in FSU WP 3.1, and the results are given in Appendix D. The appendix includes results for all gauging stations that were analysed in the Western RBD, since the choice of method for application within each UoM has been based on examination of all the results.

At ungauged flow estimation points, characteristic flood hydrographs were derived using a combination of a gamma function and an exponential curve, as for the hydrograph width analysis. The report on FSU WP 3.1 presents a set of regression equations that allow the three parameters of these functions to be estimated from the following catchment descriptors:

- BFIsoil – the baseflow index estimated from soil characteristics

- FARL – a measure of flood attenuation due to reservoirs and lakes
- ALLUV – the proportion of the catchment covered in alluvial deposits
- ARTDRAIN – the proportion of the catchment that benefits from arterial drainage schemes
- S1085 – the slope of the main channel

An alternative method from WP 3.1, using a parabolic function whose parameters are the width of the hydrograph at 50% and 75% of the peak flow, was not applied as it defines only the top half of the flood hydrograph. The report on WP 3.1 emphasises that care should be taken in applying the methods for ungauged catchments, and that the resulting hydrographs should be verified against observations if at all possible.

The regression equations for predicting the parameters of the hydrograph functions have been criticised (for example in FSU WP 3.4) for not including any term that represents catchment size. One potential way round this limitation may be to adjust the parameters by transferring information from a representative gauged catchment, termed a *pivotal station* by OPW. This approach is not discussed in the report on FSU WP 3.1. One way to implement it would be to identify a nearby gauged catchment that is physically similar to the catchment of interest (in particular in terms of area or stream network length) and then calculate an adjustment factor for each hydrograph shape parameter similarly to the way in which pivotal stations are used for adjusting QMED, i.e. the initial estimate of the parameter, from the descriptors of the subject site, is adjusted using the ratio of gauged and catchment-descriptor estimates of the parameter calculated at the pivotal station.

OPW have developed a spreadsheet called Hydrograph Shape Generator (version 3) that is based on the FSU method but it implements the transfer from a pivotal station quite differently to the way discussed above. The spreadsheet is intended for internal OPW testing, interpretation and training and is subject to ongoing development and correction. It allows the user to select a pivotal station, stressing that selection of pivotal stations should be based on the user's knowledge of the area. Where local knowledge is not available, the spreadsheet selects a pivotal station on the basis of three descriptors: S1085, BFIsoil and FARL (the text in the spreadsheet says that AREA is used but the calculations in fact use S1085 instead). The spreadsheet then copies the gauged hydrograph shape parameters (which have been derived from hydrograph width analysis) directly from the pivotal station to the subject site, with an urban adjustment. It does not make any use of the regression equations produced in WP 3.1. It should be noted that the method of transferring parameters between catchments does not appear to be based on published research. Furthermore the spreadsheet, if applied without local knowledge, does not make any allowance for catchment size when determining hydrograph shape.

This spreadsheet has been used for comparison with the results of the WP 3.1 procedure (not including any adjustments to the procedure to allow for catchment size) for ungauged catchments at a number of example sites in the Western CFRAM. Pivotal sites have been selected manually, taking into account similarity and proximity of catchments. Catchment descriptors used in the derivation of the hydrograph shape parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. Local, hydrologically similar stations were preferred over those situated further away. In some cases, more than one pivotal site was selected to test the effect on the resulting hydrograph.

6.3 Implementation of FSR rainfall-runoff method

In the rainfall-runoff method, the shape and duration of design flood hydrographs depend on two factors: the time to peak of the unit hydrograph, Tp(0), and the duration of the design storm. The recommended storm duration D depends on Tp(0) and the annual average rainfall (SAAR), although in practice for catchment-wide modelling it is appropriate to use a common value of D for all subcatchments, in which case D may be derived from trial and error, aiming to find the critical duration for the main site(s) of interest within the model. The concept of critical duration is less relevant when the method is being applied only to determine the shape of flood hydrographs, which are to be scaled to match preferred peak flows, as is the case here.

The main influence on the duration of the design hydrograph is thus the value of Tp(0). This can be estimated directly from rainfall and river level data (most easily by calculating the catchment

lag time), or indirectly from catchment characteristics. A regression equation in Flood Studies Supplementary Report 16 (FSSR16) uses the following characteristics to estimate Tp(0):

- S1085 – the slope of the main channel
- URBAN – the fraction of the catchment classed as urban on OS mapping
- SAAR – the average annual rainfall
- MSL – the length of the main stream channel

All of these except URBAN are also FSU catchment descriptors. URBAN can be estimated from the FSU descriptor URBEXT using the approximation given in the report on FSU WP3.4:

URBAN = 1.567 URBEXT

The inclusion of MSL means that the duration of the resulting hydrograph will vary with the size of the catchment, unlike in the FSU method for ungauged catchments.

In the Western CFRAM, direct estimation of Tp from hydrometric data is not possible across all of the study area due to the sparse density of Met Éireann's recording raingauges. Tp has been estimated for 16 gauged catchments in UoMs 30 and 34, using rainfall data from the recording gauges at Claremorris and Knock. Data from additional raingauges operated by the NRA were provided part-way through the study. In UoM 30 there is a raingauge operated by the NRA by the N59 in Connemara (see Figure 3-1). This is usefully located for indicating the timing of rain over the Owenriff catchment. However, no data from this gauge was provided and so it was not included in the lag analysis and the response of the Owenriff catchment is too rapid for a lag analysis to be completed with daily rainfall data. Therefore it has not been possible to complete a lag analysis for Oughterard.

Some of the river gauges selected for lag analysis are not on watercourses to be modelled but were considered as potential donor sites for adjusting Tp(0) elsewhere.

Five events were selected for each gauge, and the geometric mean lag time calculated. This was converted to an estimate of the time to peak of the unit hydrograph using:

$Tp(0) = 0.604\ LAG^{1.144}$ (from FSSR 16).

The resulting Tp(0) values are shown in Table 6-1 below. The results are very variable. At only five stations is the catchment descriptor estimate of Tp(0) within 30% of the value calculated from hydrometric data. In some cases, catchment descriptors underestimate by a factor of around 5, although it is possible that the water level measurement is affected by backwater from the large loughs in the area. There are no clear spatial patterns, with large variations possible between nearby watercourses.

Given the wide variability it seems unlikely that these results will be useful for adjusting Tp(0) estimates on ungauged watercourses. They do, however, provide potentially useful information for creating hydrograph shapes for inflows to models that are close to gauging stations. The way in which these results have been used is described in Section 6.5.

Table 6-1: Results of lag analysis for estimation of time to peak of the unit hydrograph

Gauge	Tp(0) from lag (hours)	Tp(0) from catchment descriptors (hours)	Ratio of the two Tp(0) estimates	FARL	Comments
CARTRONBOWER (30001)	30.5	6.4	4.80	0.93	Possible backwater from Lough Mask
OWER BRIDGE (30002)	65.6	11.8	5.55	0.99	Possible backwater from Lough Corrib
CORROFIN (30004)	42.5	17.1	2.49	0.99	
FOXHILL (30005)	48.7	14.3	3.41	0.98	Possible backwater from Lough Mask
BALLYGADDY (30007_	21.0	15.0	1.40	0.99	
CLAREGALWAY (30012)	51.7	19.3	2.68	0.99	
BALLYHAUNIS (30020)	7.0	6.9	1.02	1.00	
CHRISTINA'S BRIDGE (30021)	11.9	13.2	0.90	0.99	
CLOONCORMICK (30037)	25.3	14.1	1.79	0.99	
BALLYLAHAN (34004)	13.4	14.0	0.96	0.96	
SCARROW-NAGEERAGH (34005)	7.5	12.0	0.63	0.93*	Despite fairly low FARL catchment descriptors overestimate Tp, possibly because they do not account for effects of the arterial drainage scheme.
CURRAGHBONAUN (34009)	15.1	8.1	1.87	1.00	
CLOONACANNANA (34010)	13.9	8.3	1.69	0.99	
GNEEVE BRIDGE (34011)	39.6	15.1	2.63	0.87	Low FARL so catchment descriptors are expected to under-estimate Tp
BANADA (34013)	12.6	4.9	2.58	0.99	
SWINFORD (34021)	5.6	5.2	1.08	0.95	
KILTIMAGH (34024)	25.7	11.6	2.21	0.92	
CHARLESTOWN (34031)	5.4	4.6	1.19	1.00	

* FARL has been revised from the value of 0.90 provided in the FSU catchment descriptors for station 34005 to account for the exclusion of an area including three loughs to the east of Kilkelly which was incorrectly included in the FSU catchment boundary. This is illustrated in the Inception Report. FARL was re-calculated manually, digitising the surface area of loughs from the OSI 1:50,000 map.
Catchment descriptors for these and all gauges discussed in the following sections are provided in the digital deliverables, Section 12.

6.4 Comparisons of alternative methods for hydrograph shape generation

6.4.1 General approach

Since the Western CFRAM covers a large number of watercourses, it is desirable to select a method for production of hydrograph shapes that is suitable for as many watercourses as possible, to minimise the need to apply multiple methods. The primary requirement is for a method that results in a realistic duration and volume of flood water for the design flood that will be used to run the hydraulic models. These aspects will affect the impact of the flood on land and properties, and the assessment of schemes for flood management. It is also important that the chosen method is capable of producing consistent hydrographs for input to models with multiple tributaries, as discussed above.

6.4.2 Summary of inception stage comparisons

The methods discussed above have been compared at two sets of example catchments. First, in the inception stage, hydrograph shapes were calculated directly from observed data using hydrograph width analysis at 21 gauging stations. The results were compared with hydrographs produced using the FSR rainfall-runoff method solely from catchment descriptors (Appendix D). This gives an indication of whether the rainfall-runoff method is capable of producing realistic hydrograph shapes at gauged sites, and therefore if results are likely to be applicable to ungauged sites.

For UoM 30, at Claregalway and Claremount, there is a close match between the two hydrograph shapes. At Wolfe Tone Bridge the FSR hydrograph is much narrower than that derived from observed events. This is to be expected because the FSR method does not account for the influence of lakes unless it is applied in conjunction with reservoir routing.

6.4.3 Additional tests for main stage

A second set of tests has been carried out for the main stage hydrology, at a set of five gauged and five ungauged catchments chosen to be representative of the typical range of catchment locations and sizes found across the Western RBD. The catchments are listed in Table 6-2, in Section 6.4.5. For these catchments, the following methods have been applied for calculation of hydrograph shapes:

- FSU with hydrograph shape parameters calculated from catchment descriptors using the regression formulae from WP 3.1 ("FSU ungauged" method).
- FSU transferring the hydrograph shape parameters from one or more pivotal sites, selected using judgement, with the transfer carried out using the spreadsheet from OPW ("FSU pivotal" method).
- FSR rainfall-runoff using catchment descriptors to estimate Tp(0) ("FSR" method).

For the five gauged catchments, hydrograph shapes from the above methods have been compared with those constructed directly from the observed data (taken from the inception phase analysis). For the five ungauged catchments, the shapes have been assessed in the light of the order of magnitude of hydrograph duration that would normally be expected for a catchment of that type.

In addition, a more objective assessment of the hydrographs has been carried out by multiplying the dimensionless hydrographs by the design peak flow and then assessing the resulting design flood hydrograph using the IBIDEM technique. IBIDEM stands for Interactive Bridge Invoking the Design Event Method and was developed within FSU WP 3.5. It involves assessing a design hydrograph produced using FSU (or other) methods in the light of the FSR rainfall-runoff model structure. IBIDEM is a web-based software package that calculates the time to peak and standard percentage runoff parameters that would be necessary for the FSR rainfall-runoff model to produce an output similar to the FSU design hydrograph. If the resulting parameters have unrealistic values it is an indication that the input hydrograph may not be appropriate given the nature of the catchment.

IBIDEM requires inputs including selected FSU catchment descriptors and a table of design rainfall depths for the catchment. The latter has been generated for each example catchment using the FSU design rainfall statistics (WP 1.2). For medium and large catchments, the design rainfalls have been calculated from spatially averaged parameters of the rainfall depth-duration-frequency

model. This is the approach recommended in Met Éireann Technical Note 61. For small catchments, parameters have been chosen at a single grid square within the catchment.

6.4.4 Results of visual comparison of shapes

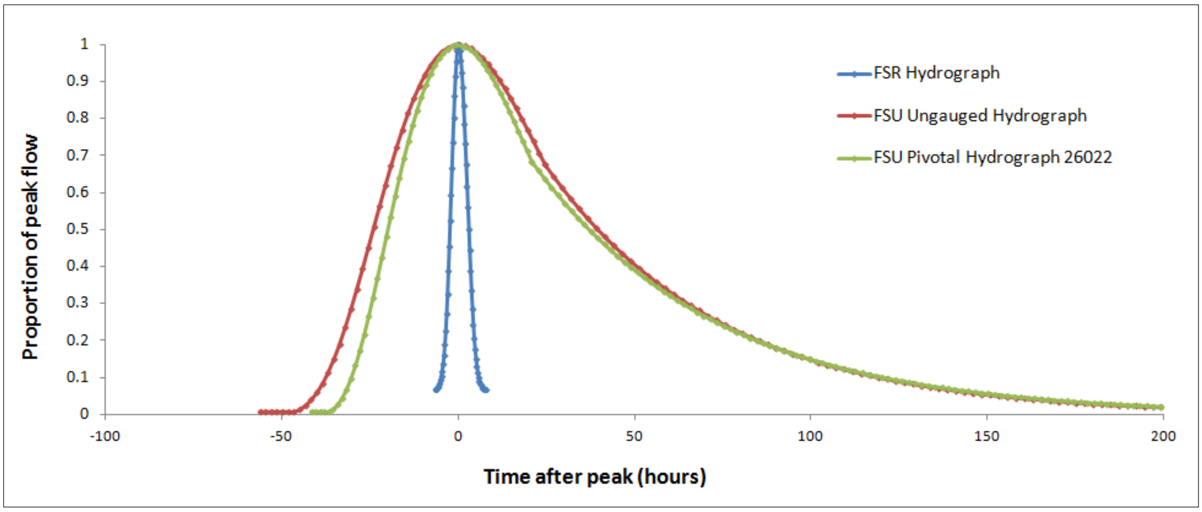
The visual comparison of shapes has been completed to confirm preferred choice of method at ungauged sites; the analysis has been completed at gauged sites so that each method can be compared against observed data. The results of the hydrograph shapes comparison are presented in Appendix E in the form of a summary sheet for each of the ten example catchments across the Western RBD showing the hydrographs and listing the parameters used to produce them and the pivotal sites that were chosen. Catchment descriptors for these and all gauges discussed in the following section are provided in the digital deliverables, Section 12.

Out of the five gauged catchments, the FSU ungauged method appears to give the best fit to observed hydrographs at two gauges, and the FSU pivotal method (implemented using the OPW spreadsheet) at another two gauges. At all four of these gauges, the FSR method gives a fit that is judged to be acceptable. At the fifth gauge, on the Castlebar River at Turlough, none of the methods tried gives a hydrograph that matches the observed events; the comparison of the methods for this gauge is shown in Appendix E.

For the five ungauged catchments, the results of the various methods were highly variable. The FSR hydrograph was similar to those from the FSU methods at one site (Grange at Corrofin) but produced a narrower (i.e. shorter-duration) hydrograph elsewhere. The FSU pivotal method produced a narrower hydrograph than the FSU ungauged method at four of the five sites, although the difference was minor in two of these cases.

The difference between FSU and FSR hydrographs was particularly marked for one of the example catchments, the Carrigans Upper watercourse at Ballymote in UoM 35 (Figure 6-1). At half the peak flow, the FSU hydrographs have a duration of 64 and 56 hours (ungauged and pivotal respectively) whereas the FSR hydrograph lasts for 5.25 hours. To put this into context, it is helpful to know that the catchment in question has an area of $2.5km^2$. In the absence of backwater effects (which are not represented by any of the methods applied), it would not generally be considered realistic for such a small catchment to give rise to floods that last for days.

Figure 6-1: Comparisons of hydrograph shapes for the Carrigans Upper watercourse at Ballymote



6.4.5 Results of comparisons using IBIDEM

The parameters fitted by IBIDEM for the ten test catchments are shown in Table 6-2.

Table 6-2: Results of IBIDEM tests to assess hydrographs at ten example catchments

(a) Example gauged sites

	30020 Dalgan at Ballyhaunis	32011 Bunowen at Louisburgh	34018 Castlebar at Turlough	35002 Owenboy at Billa Bridge	35073 Dalgan at Sligo
FSR hydrograph shape					
Time to Peak (hr)	7.0	4.7	5.8	5.2	9.7
Standard Percentage Runoff	8.6	91.4	**-1.7**	23.4	11.0
FSU hydrograph shape from catchment descriptors					
Time to Peak (hr)	15.2	Run failed	95.9	13.7	66.2
Standard Percentage Runoff	20.7	**>100**	32.7	53.5	51.0
FSU pivotal hydrograph shape (first donor)					
Time to Peak (hr)	45.5	3.1	66.4	97.0	9.2
Standard Percentage Runoff	59.7	70.3	25.4	**245**	10.5
FSU pivotal hydrograph shape (second donor)					
Time to Peak (hr)	6.8	n/a	62.2	9.6	n/a
Standard Percentage Runoff	8.5		24.2	37.9	
Median observed shape from HWA					
Time to Peak (hr)	10.7	3.7	99.2	7.2	68.5
Standard Percentage Runoff	13.2	79.0	33.5	30.2	52.1

(b) Example ungauged sites

	Athenry at Athenry	Carrigans Upper at Ballymote	Grange at Corrofin	Loughrea	Swinford at Swinford
Peak flow for 1% AEP from FSU (m^3/s)	8.1	4.6	40.2	6.1	8.2
FSR hydrograph shape					
Time to Peak (hr)	7.6	3.3	12.9	3.8	5.3
Standard Percentage Runoff	9.1	48.2	15.0	11.9	15.8
Storm Duration (hr)	17	7.25	27	8.25	13
FSU hydrograph shape from catchment descriptors					
Time to Peak (hr)	29.4	36.4	17.4	Run failed	14.3
Standard Percentage Runoff	30.0	**341**	21.2	**>100**	38.0
Storm Duration (hr)	63	79	37		33
FSU pivotal hydrograph shape					
Time to Peak (hr)	36.4	30.5	19.7	Run failed	11.9
Standard Percentage Runoff	35.3	**297**	24.0	**>100**	31.3
Storm Duration (hr)	77	67	41		27

Both the time to peak (Tp(0)) and standard percentage runoff (SPR) parameters fitted by IBIDEM provide useful information. However, they must be interpreted with care as IBIDEM is a rather complicated concept that, applied here, combines elements of several different methods.

For Ballymote, Loughrea, Louisburgh and Billa Bridge the IBIDEM runs using the FSU hydrograph shapes (from catchment descriptors, pivotal sites or both) resulted in inferred SPR values greater than 100%, i.e. physically impossible. There are three possible explanations for the very high SPR values:

1. The FSU hydrographs are too prolonged;
2. The supplied peak flow from FSU is too high for the catchment;
3. The FSR rainfall-runoff method, applied using FSU design rainfalls, is underestimating design floods for the catchment (hence it appears FSU design flows are over-estimated in comparison).

The first explanation seems very likely given the extremely long durations of some of the FSU hydrographs. This is a useful finding which helps to confirm that the FSU method of generating hydrograph shapes (whether applied using catchment descriptors or via OPW's pivotal spreadsheet) does not always yield hydrographs that are consistent with the properties of the catchment.

Elsewhere, in nearly all cases IBIDEM yields longer Tp(0) parameters when fitting to the FSU hydrograph shapes than to the FSR hydrographs. The consequence is higher fitted SPR parameters for the FSU hydrographs; this is because when the flood runoff is spread out over a longer time, it is necessary to produce a greater relative volume of runoff in order to match a given peak flow. Implied SPR parameters fall in the following ranges (ignoring results below 0% or above 100%):

- FSR hydrographs: 11% to 91%, mean 23%
- FSU hydrographs from catchment descriptors: 21% to 53%, mean 35%
- FSU hydrographs from pivotal site: 10% to 70%, mean 37%
- Median observed hydrographs from HWA: 13% to 79%, mean 42%

To put these values into context it may help to know that SPR when estimated from the FSR soil maps (WRAP maps) ranges from approximately 10% at Athenry and Loughrea up to 28% for the Grange at Corrofin and at Ballyhaunis, 37% for Ballymote, Swinford and Turlough and 50% for Louisburgh, Billa Bridge and Sligo. At Athenry, Loughrea and Ballymote the implied SPR parameter from the FSR hydrograph gives a reasonably close match to that estimated from soil characteristics. At Corrofin, Swinford, Turlough and Sligo the FSU hydrographs give a closer implied SPR to that estimated from soils. Elsewhere the picture is more varied. These results should not be taken to mean that hydrograph shapes are necessarily any better if they give a closer match to SPR values from the WRAP maps; there are various possible reasons for the discrepancies, as discussed below.

When IBIDEM is applied to a design flood hydrograph whose shape has been generated from the FSR rainfall-runoff method, the fitting process in IBIDEM will inevitably yield a hydrograph with a very close fit, whose Tp(0) parameter is more or less identical to the time to peak of the unit hydrograph that was used to generate the initial hydrograph shape. On gauged catchments the fitted Tp(0) from the FSR method can be compared with that fitted to the median observed hydrograph shapes. This replicates the visual comparison of hydrograph shapes carried out in the inception stage. For three of the five catchments there is a reasonably close match. The exceptions are Sligo, where Lough Gill results in major attenuation that is not accounted for in the FSR method, and Turlough. For the Castlebar at Turlough the FSR hydrograph has a Tp(0) very much shorter than that fitted to the observed hydrographs. This large discrepancy is also manifested in the implied SPR which is negative for the FSR hydrograph. There are three possible explanations for this and for some of the other fitted SPR values from FSR hydrographs that appear to be on the low side (such as Ballymote and Ballyhaunis):

1. The FSR hydrograph shape is too narrow for the supplied peak flow, hence the volume of runoff is too low;
2. The supplied peak flow from FSU is too low for the catchment;
3. The FSR rainfall-runoff method, applied using FSU design rainfalls, is underestimating design peak flows for the catchment (hence it appears FSU design flows are overestimated in comparison).

Explanation number 3 is a likely candidate in some cases, given the widespread tendency for the FSR rainfall-runoff method to result in design flows that exceed those obtained from direct analysis of flood peak data. However, for the Castlebar at Turlough the more likely explanation is that the FSR hydrograph is unrealistically narrow. As discussed in the Inception Report, it is difficult to understand why observed hydrographs at Turlough are so prolonged, and further investigation of possible backwater effects or groundwater interactions may be possible once the MPW hydraulic model is complete.

6.5 Overview of selected approach for hydrograph shapes

For most hydraulic models it is recommended that hydrograph shapes are produced using the FSR rainfall runoff method. The principal reasons for this decision are:

- The FSU hydrograph shape method for ungauged catchments, whether applied using catchment descriptors or the pivotal catchment approach implemented in OPW's spreadsheet, does not take into account the size of the catchment and so can produce hydrographs that appear unrealistic.
- At four of the ten test catchments for which IBIDEM was applied, the FSU method resulted in inferred SPR values greater than 100%.
- At many of the 23 gauging stations for which median hydrograph shapes have been created, the FSR method gives an acceptable match to the observed hydrograph, even without any adjustment of the time to peak using local data.
- It is possible to adjust the time to peak using the results of lag analysis on some catchments, thus ensuring that the FSR method incorporates local hydrometric data. On some other catchments (lacking recorded raingauge data) it is possible to adjust time to peak by trial and error to better match observed hydrographs.
- The FSR method, applied using 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.

For UoM30, where results from lag analysis are available the time to peak has been adjusted for model inflows close to the gauging stations at Ballyhaunis (adjustment factor 1.02), Ballygaddy (1.40 from lag analysis but 2.00 gives a better match with flood volume frequency analysis as discussed below) and Corrofin (2.49).

The duration of the FSR hydrograph is affected by the duration of the design storm as well as the time to peak of the unit hydrograph. As mentioned above, a uniform design storm duration will be applied to each sub-catchment within a model. Because the FSR method is being used only to control the shape of the hydrographs rather than to provide an accurate representation of the catchment response and therefore magnitude of the peak flows, it is not appropriate to use this method there is no need to identify 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. This approach has the potential to overestimate flood risk on smaller tributaries where the storm duration has been developed with the larger watercourse in mind. The resulting flood risk on these tributaries will be reviewed within the hydraulic modelling phase and, if necessary, additional runs with the storm duration more suitable to the size of the tributary will be completed. The sensitivity of the flood risk extents to the assumption that the critical storm duration can be derived from catchment descriptors, where no other information is available, will be investigated as part of the hydraulic modelling work.

There are some individual AFAs where it is worthwhile using an alternative approach to generating hydrograph shapes, i.e. applying a characteristic hydrograph calculated as the median of the widths of observed hydrographs. This approach is recommended for:

- AFAs close to gauging stations where the FSR hydrograph does not fit observed hydrographs well (for example due to the influence of storage in the catchment),

- where results from lag analysis are not available
- and where flood risk is predominantly from one river with insignificant inflows through the AFA (which reduces the importance of considering the relative timing of model inflows).

The results of the hydrograph width analysis are shown in Appendix D and this approach has been applied at Galway in UoM 30. The Corrib catchment at Galway is unusual owing to the major attenuation provided by Lough Corrib. The FSR hydrograph shape is much narrower than the median of observed hydrographs, and so the latter has been adopted as the preferred design hydrograph shape.

6.6 Checks against volume frequency analysis

A statistical analysis of annual maximum flood volumes has been carried out using flow data from Ballygaddy gauging station (30007). The results, which were obtained before the rating review was completed, are shown in Appendix B. They indicate the expected volume of flood water over a given duration for a given AEP. This approach provides the opportunity for an independent check on the volumes of the design flood hydrographs developed using the approach outlined above, in combination with the design peak flows estimated as described in Chapter 5.

At Ballygaddy, the preferred hydrograph shape is derived from the FSR method with time to peak set to 21 hours from lag analysis. When scaled to match the estimated 1% AEP peak flow of 120m^3/s, the design hydrograph gives a volume of 17.7Mm^3 (million cubic metres) accumulated over 4 days. The corresponding 4-day volume from the volume frequency analysis is 28Mm^3. This would reduce to approximately 26 Mm^3 as a result of the revisions to the rating described in Appendix A.

For the 10% AEP flow of 82 m^3/s, the design hydrograph has a 4-day volume of 12Mm^3. The corresponding 4-day volume from the volume frequency analysis is 22Mm^3.

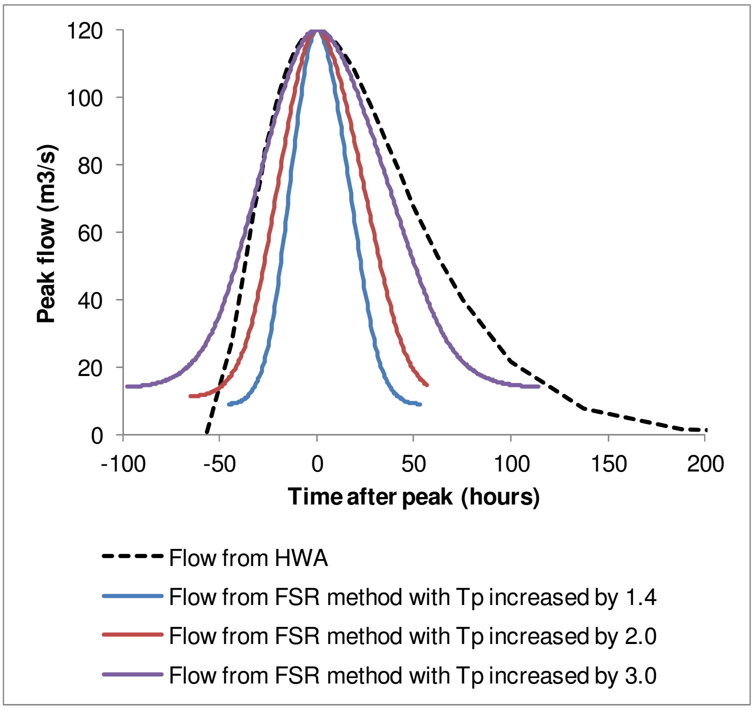
For both AEPs the initial design hydrograph has a volume considerably lower than that estimated from volume-frequency analysis. There are several possible reasons for this discrepancy, including:

- Uncertainty in the statistical estimates of volume frequency, due to the limited record length at Ballygaddy – this may be considerable for an AEP of 1% but lower for the 10% AEP.
- Uncertainty in the statistical estimates of flood peak frequency.
- Systematic variation in flood volume with flood magnitude.
- Uncertainties in the fitting of the gamma/exponential distributions to the hydrograph widths calculated by the HWA software, which are not defined below a certain percentile.
- Unrealistically short duration for the design flood hydrograph shape.

Despite the fact that the time to peak which controls the duration of the hydrograph has been estimated from lag analysis, it appears that the last bullet point listed above is the primary reason for the discrepancy. Figure 6-2 shows alternative hydrographs, all scaled to fit the estimated 1% AEP peak flow of 120m^3/s. It can be seen that the initial FSR hydrograph, with Tp increased by 1.40 following lag analysis, is much narrower than the median of observed hydrographs calculated from HWA. It is necessary to increase Tp by a factor of 3 to obtain a hydrograph of comparable width to the HWA result. However, this leads to a flood volume higher than that obtained from volume frequency analysis.

As a compromise it is recommended that the time to peak for Ballygaddy is increased by a factor of 2 from the original estimate made from catchment descriptors. This yields a 1% AEP design hydrograph with a volume of 24m^3 over the highest 4-day period, close to the approximately 26 Mm^3 obtained from volume frequency analysis. A similar adjustment was considered at Corrofin, since this is near to Ballygaddy on the Clare River, but this is not necessary because the time to peak for Corrofin is already increased by a factor of 2.49 from the original estimate made from catchment descriptors, in accordance with the results of the lag analysis (see Section 6.5).

Figure 6-2: Comparisons of hydrograph shapes for the Clare at Ballygaddy



7 Summary of flood estimation process

7.1 Summary of steps leading to design flood hydrographs

The chapters above have described a detailed investigation of alternative methods and provided a justification for the chosen approach. A summary of the process that has been followed to implement this approach is given in Table 7-1. It shows how there are some differences in the ways that gauged and ungauged locations have been treated. The table is a deliberately simplified summary and there will be some locations where the methods applied are slightly different from those outlined in the table. The following section outlines the approach used for each individual AFA.

Table 7-1: Summary of flood estimation process

Step	HEP with flow data	Ungauged HEP with suitable donor site	Ungauged HEP with no donor site
1	Obtain catchment descriptors from FSU dataset, amend or create from other datasets if necessary e.g. if the catchment is smaller than covered by the FSU digital data.		
2	Estimate QMED from annual maximum flows	Estimate QMED from catchment descriptors and adjust using ratio from one or more donor sites	Estimate QMED from catchment descriptors
3	Estimate flood growth curve from both single-site and pooled analysis and decide which is more appropriate	Estimate flood growth curve from pooled analysis unless single-site growth curve is preferred at nearby donor site.	Estimate flood growth curve from pooled analysis.
4	Extend flood growth curve for AEPs lower than 1% using ratios from FSR rainfall-runoff method growth curves.		
5	Multiply QMED by flood growth factors from growth curve to obtain design peak flow for each AEP		
6	Derive hydrograph shapes from observed hydrographs and FSR methods and decide which is more appropriate.	Derive hydrograph shapes from FSR rainfall-runoff method with Tp adjusted using lag analysis if results available at donor. Or – use hydrograph shape derived at donor if observed shape preferred there.	Derive hydrograph shapes from FSR rainfall-runoff method, with time to peak estimated from catchment descriptors.
7	Scale hydrograph shape so that the peak flow matches that calculated at step 4, for each AEP.		

7.2 Summary of approach followed at each AFA

Table 7-2 lists the methods that have been applied at each AFA to estimate QMED, the flood growth curve and the design hydrograph shape. It includes the reference numbers of donor or pivotal gauging stations that have been used to adjust QMED or provide hydrograph shapes. In some cases, different methods have been used for different watercourses or different hydrological estimation points (HEPs). The table provides a summary of the various methods used in such cases. A more detailed audit trail of the calculations is available in the digital deliverables, which provide information on the method used at each individual HEP, including those on MPWs which are not listed in the table below.

Table 7-2: Methods used to estimate design flood hydrographs at each AFA

AFA	Name	QMED method	Growth curve method	Distribution	Hydrograph shape
Corrofin	Clare – upstream of gauge 30004	DT – Pivotal 30007 and 30004	P	GL	RR-LAG
	Clare – downstream of gauge 30004	DT – Pivotal 30004	SS-30004	G	n/a – will be routed by model
	Grange	CD	P	GL	RR
Galway City	Corrib	DT – Pivotal 30061	P	GL	HWA - 36001
Oughterard	Owenriff	DT – Pivotal 30101	P	GL	RR
	Tonweeroe	CD	P	GL	RR
Tuam	Clare	DT – Pivotal 30007 and 30004	P	GL	RR-LAG or RR-ADJ
	Lough Park, Nanny, Suileen	CD	P	GL	RR
Ballyhaunis	Dalgan, Devlis, Curries	DT – Pivotal 30020	P	GL	RR-LAG

Meaning of codes:

QMED methods - Data Transfer (DT)[18] */ Catchment Descriptors (CD)*

Growth curve method - Pooled (P) / Single Site (SS)[19]

Distribution - General Logistic (GL) / Gumbel (G) / Generalised Extreme Value (GEV)

Hydrograph shape – FSR rainfall-runoff (RR) / FSR rainfall-runoff with Tp(0) adjusted from lag analysis (RR-LAG) / FSR rainfall-runoff with Tp(0) adjusted to match HWA results (RR-ADJ) / hydrograph width analysis from observed events (HWA)[20]

[18] DT – If data transfer method adopted, pivotal station chosen is detailed

[19] SS – If single site method adopted, station number for which the growth factors have been derived is detailed

[20] HWA – If hydrograph width analysis adopted, station number for which the hydrographs have been analysed is detailed

8 Applying design flows to the river models

8.1 Introduction

Inflows for the river models will be specified in accordance with the guidance developed for FSU WP 3.4. As hydrodynamic models are being used to represent the rivers, there is the potential for conflicts between the flow simulated by the river model (routed from hydrological inputs applied at the upstream model limits) and the design flows estimated by hydrological methods. In modelling a flood event of a given probability throughout a river system, there is no guarantee that hydrographs scaled to match design flows at model inflows will result in the preferred design flows being reproduced further downstream within the model.

The report on WP 3.4 suggests that the following four factors should be considered when assessing how to apply design inputs to a river model:

1. The extent of the model (for example, whether it includes just one watercourse or extends up its tributaries as well).
2. The presence of gauging stations close to points of interest within the model.
3. The degree of dependence between the upstream and downstream ends of the model, and between any tributaries (or non-modelled inflows) and the main river.
4. The importance of backwater effects.

8.2 Approach adopted for the CFRAM

This section sets out the approach that is expected to be applied when carrying out design runs of hydraulic models. This work is still under way and so the final approach may change, and readers should refer to the hydraulic modelling reports for a record of the method that is finally adopted.

When the extent of a model is short, i.e. there is little change in catchment area along the model reach and little opportunity for attenuation, then setting inflows to the model is expected to be straightforward (apart from perhaps on some small urban watercourses where flows may be affected by hydraulic constrictions such as culverts). The inflow to the model will be set to the design flood hydrograph for the corresponding HEP, and the peak flow at key points within the model will be checked against design flows for the corresponding HEPs. Significant discrepancies, while considered unlikely, will be investigated and applied as appropriate through the hydraulic modelling process by applying additional lateral flows where appropriate.

Longer model reaches, particularly on MPWs or on watercourses that include major loughs, provide more opportunities for changes in flow due to interactions between tributaries or attenuation. As suggested in the guidance, the first step will be to model a design run of the entire River Clare model, with inflows set as described below. If this does not give an adequate representation of design peak flows and flood durations throughout the model reach, we will divide the model into several reaches, each of which will be run separately.

One of the main considerations in the FSU guidance is the location of gauging stations within the model reach, because it is at these sites that the greatest confidence can be placed in the design flows.

As a starting point, the magnitude of inflows from tributaries that contribute a significant proportion of the downstream flow will be set using the exceedence probabilities given in the FSU guidance, which depend on the degree of similarity between the catchments of the main river and the tributary. Where necessary, additional lateral inflows will be applied to keep the modelled flow in the river at a realistic value on long model reaches where there are no major confluences. Lateral flows have been developed where required using the FSU methodology to achieve flows at HEP points. Where possible, the use of intervening areas (which are not true catchments) will be avoided as advised in the FSU guidance on river modelling. Since the design flows are not being calculated from a rainfall-runoff model, there is no need to create intervening areas to ensure that runoff from all parts of the catchment is allowed for.

The relative timings of inflows will be specified using the FSR rainfall-runoff method since it has been found that it gives a more realistic representation of hydrograph shapes for ungauged inflows (Chapter 6).

The approach of adjusting model inflows in order to match a preferred hydrological estimate of the peak flow is not recommended as suitable in all cases by the FSU guidance. One example of an exception is on river reaches where flows are influenced by hydraulic backwater effects. This will apply on the lower part of the Clare River, downstream of Claregalway, as the river approaches Lough Corrib. On this reach the preferred approach will be to use the hydraulic model to work out the flow in the river given a suitable input hydrograph at Claregalway and a downstream boundary at Lough Corrib.

On the River Corrib in Galway and the Owenriff River at Oughterard the model reaches are relatively short with little change in catchment area and so application of inflows to the models is expected to be more straightforward.

9 Assumptions and uncertainty

9.1 Assumptions

The hydrological analysis relies on a number of general assumptions, which have been necessary given the requirement to estimate design floods for large numbers of locations and for probabilities that include very rare events. Through the study it has been possible to test and refine many of these assumptions. The principal assumptions that remain are:

9.1.1 Assumptions regarding data

- The design flows rely heavily on the availability and quality of flood flow datasets. At rating review gauges, it has been possible to check the quality of the flow measurement and extend the rating up to high flows. For UoM30 rating reviews were carried out at all relevant flow gauges except Wolfe Tone Bridge, where it has been necessary to assume that the existing rating is reliable for flood flows.

9.1.2 Assumptions regarding hydrological processes

- It is assumed that hydrological processes that operate during extreme floods (down to an AEP of 0.1%) are similar to those that govern more moderate floods that have occurred during the period of gauged records.

9.1.3 Assumptions regarding methods of hydrological analysis

- For small ungauged catchments, it is assumed that the error introduced by adjusting QMED using a much larger donor catchment will be greater than the benefit (in terms of standard error) of applying the adjustment, and so QMED has been estimated solely from catchment descriptors on such catchments.
- It is assumed that, for the majority of HEPs, the FSR rainfall-runoff method gives a more realistic hydrograph shape than the FSU ungauged catchment method, with or without adjustment using a pivotal site. This assumption has been tested at a set of example catchments as discussed in Chapter 6.

9.2 Uncertainty

The brief for the CFRAM requires degrees of confidence to be presented in the mapped flood outlines. Flood frequency estimates are inherently uncertain because they cannot be measured or formally validated against observed data.

For the Western CFRAM, design flood hydrographs have been developed for a wide range of flood AEPs (down to 0.1%, corresponding to a return period of 1000 years) and for a large number of locations. There is inevitably a large degree of uncertainty in the results, particularly at ungauged locations and for low AEPs. It is important that the results produced in this study are not taken as the final word on flood frequency for the Western RBD. The uncertainty in the design flows is likely to be the largest source of the uncertainty in the modelled water levels and mapped flood outlines produced in the CFRAM study.

This uncertainty can be broken down into different components:

- **Natural uncertainty**, from the inherent variability of the climate.

  This is a substantial source of uncertainty. The longest record of flood peak data that has been analysed in UoM 30 is 39 years, at Corrofin. Some of the pooling groups include longer records, up to around 60 years in some instances. In a few cases it has been possible to augment the recorded flood peak data with information from longer-term flood history, although little quantitative information has been found. There is a great deal of uncertainty in extrapolating from these relatively short records to estimate design flows that are expected to occur once in 100 or 1000 years on average.

  Natural uncertainty can be classed as *aleatory*. Aleatory uncertainty describes the random occurrence of values about a mean that can be appropriately described by a probability

distribution; as a result confidence intervals can be assigned to this distribution and associated with mapped outputs.

- **Data uncertainty**, from the measurement of flood flows. As discussed above under assumptions, the degree of uncertainty in some of the rating equations within UoM 34 is unknown.
- **Model uncertainty**, which includes aspects such as the choice and fitting of flood frequency distributions and the application of ungauged catchment methods such as the regression equation for estimating QMED and the procedures for defining hydrograph shapes.

  The uncertainties associated with data measurement and models or analysis techniques can be classed as *epistemic*, i.e. associated with knowledge. Some sources of epistemic uncertainty describe variation that do not occur randomly and so cannot be described probabilistically. It is therefore difficult to assign limits to this uncertainty as the true range of values can vary widely.

There is an increasing desire to see uncertainty discussed and presented in flood mapping and assessment investigations. However many of the uncertainties in this work are epistemic and confidence intervals based on probability distributions cannot be derived. A recent publication[21] suggests it might be better to represent such uncertainties "possibilistically". This can be done through scenarios or sensitivity testing.

In considering how to assess uncertainty for use on the CFRAM it is important to understand where probability distributions can be applied to uncertainty and where sensitivity tests need to be used to investigate uncertainty.

Quantifying uncertainty

It is possible to quantify some elements of uncertainty. Where an index flood approach is applied to derive design flows, uncertainty can in theory be assessed on the two components used in the development of the hydrology, the index flood (QMED for the FSU method) and the growth curve.

The standard error (SE) is a measure used to describe uncertainty about an estimate of something, when the estimate is based on the data in a sample. It represents only the aleatory uncertainty and does not account for any possible bias in the procedure for estimating design flows.

Factorial standard error (FSE) is a term used occasionally in flood hydrology to describe errors from an estimate made from a multiplicative process, such as the regression equation that estimates QMED from a multiple of catchment descriptors. These two measures of uncertainty in a design flow Q are related thus:

$FSE = 1+ (SE/Q)$

The uncertainty in QMED can be assessed using the equations for SE and FSE provided in the FSU WP2.2 report. These are provided for estimates derived from catchment descriptors or at gauge sites:

- For QMED estimated from catchment descriptors: FSE=1.37
- For QMED estimated from N annual maximum flows: $SE = 0.36/\sqrt{N}$

So for many small ungauged HEPs, where no suitable donor catchment could be found, the FSE in QMED is 1.37. For HEPs at gauging stations, the typical FSE for UoM 30 is 1.06, increasing to 1.09 at Claregalway and 1.12 at Oughterard where the record is shorter.

At most locations, growth curves are derived from pooled analysis. In discussing the standard error of pooled growth curves, the FSU WP2.2 report states that the uncertainty in the design flow for any return period is dominated by the uncertainty in QMED. This result differs from the findings of research elsewhere (such as Kjeldsen and Jones, 2006[22]). While the difference may be due to the unusually low skewness of Irish flood datasets, there is a risk that the overall uncertainty in design flows could be underestimated if it is assumed that even for very long return periods the

21 Framework for Assessing Uncertainty in Fluvial Flood Risk Mapping, Flood Risk Management Research Consortium Research Report SWP1.7, 2011.

22 Kjeldsen, T.R. and Jones, D.A. (2006). Prediction uncertainty in a median-based index flood method using L moments. Water Resources Research 42, W07414.

factorial error is similar to that calculated for QMED. However, for the purpose of this study the findings of the WP 2.2 report will be taken at face value, and hence calculation of uncertainty in design flows from pooled growth curves will be limited to the consideration of factorial errors in QMED.

The standard error for single-site flood frequency curves (which have been applied at Corrofin) has been estimated using theoretical expressions given in the FSU WP2.2 report (Section 13.2). When a Gumbel distribution is fitted, the SE depends on the scale parameter, the number of annual maximum flows and the return period. The scale parameter is that for the flood frequency curve, not the flood growth curve which is what is shown in Appendix B. The resulting standard error is:

- At Corrofin (scale parameter 19.0): the SE for the 1% AEP flood is 14.3m^3/s. This is 8% of the 1% AEP design flood.

Confidence intervals

If it can be assumed that factorial errors in QMED are normally distributed, the factorial error can be used to construct approximate confidence intervals for QMED. The 95% confidence interval, i.e. the range in which we are 95% confident that the true value of QMED lies, is equal to ($QMED/FSE^2$, $QMED.FSE^2$).

Therefore approximate 95% confidence intervals for the estimated design peak flow Q are as follows:

- 0.89Q to 1.12Q for HEPs at (or very close to) Corrofin, Ballygaddy and Wolfe Tone Bridge gauging stations
- 0.86Q to 1.16Q for HEPs at (or very close to) Ballyhaunis gauge
- 0.80Q to 1.25Q for HEPs at (or very close to) Oughterard gauge
- 0.54Q to 1.85Q for ungauged HEPs with no donor adjustment applied

It is important to realise, as discussed above and below, that these represent only part of the uncertainty in the design flows.

For ungauged HEPs where a donor adjustment has been applied, the confidence interval can be expected to lie somewhere between the values for gauged and ungauged sites. This is obviously a very large range. The nearer the HEP to the gauge along the river network, and the more similar the catchments, the closer will be the confidence interval to that which applies at the gauge. The FSU research did not produce any statistical model that could be used to quantify how the uncertainty in QMED estimation reduces as a result of applying a donor adjustment, and so any attempt to quantify the uncertainty for ungauged HEPs where a donor adjustment would be subjective and open to challenge.

By the same method, 95% confidence intervals for the 1% AEP design flow Q estimated from single-site growth curves are:

- 0.75Q to 1.34Q for HEPs at (or very close to) Corrofin and Claregalway gauges

These confidence intervals do not make the assumption that the FSE is invariant with return period, and thus may be a fuller description of the uncertainty than those given above for pooled growth curves. However, they do not include any allowance for bias in the estimation procedure or for errors in the rating curves, to which single-site flood estimates are particularly sensitive.

Sensitivity testing

Other sources of uncertainty cannot be easily quantified. There is scope to examine some of them through sensitivity testing. This has been carried out in aspects of the analysis, for example by comparing growth curves fitted using different distributions (Appendix B), QMED adjusted using different donor gauges or design flood hydrographs derived using different methods (Chapter 6).

Further sensitivity testing will be carried out as part of the hydraulic modelling work to quantify the effect that these quoted bounds of uncertainty have on the predicted extent of flood risk.

10 Design sea levels

10.1 Synopsis

This chapter details the methodology of work undertaken to produce design tidal curves on the coast of the Western RBD. Tidal graphs are required to be used at the downstream boundary of the Galway hydraulic model and in the calculation of the wave overtopping inflows for the same AFA.

The work described in this chapter covers the whole of the Western CFRAM study area.

10.2 Design tidal graphs

A design tidal graph is a time-series that quantifies how sea-levels are expected to change through time during an extreme event. It is these design tidal graphs that are used to drive the still water component of the flood inundation model at its offshore boundaries. Creation of design tidal graphs requires three principal sources of information: an extreme sea level (ESL) estimate for the return period of interest; a design surge shape, and; a design astronomical tide.

Initial assessments were made into the data available for the three required sources and the most relevant source locations were selected respective to each study site shown in Table 10-1.

Table 10-1: Locations of data sources required for the design tidal graphs

Model location	HAT tide gauge	ESL data point location code	Surge profile
Westport	Inishgort	W41	Inishgort
Galway	Galway	W6	Galway
Kinvarra	Galway	W3	Galway
Sligo	Sligo Harbour	NW6	Sligo
Ballysadare	Sligo Harbour	NW6	Sligo
Ballina	Killala Bay	NW1	Sligo
Newport	Inishgort	W42	Inishgort
Louisburgh	Roonah Bay	W39	Inishgort
Clifden	Bofin Harbour	W29	Inishgort
Roundstone	Roundstone Bay	W23	Galway

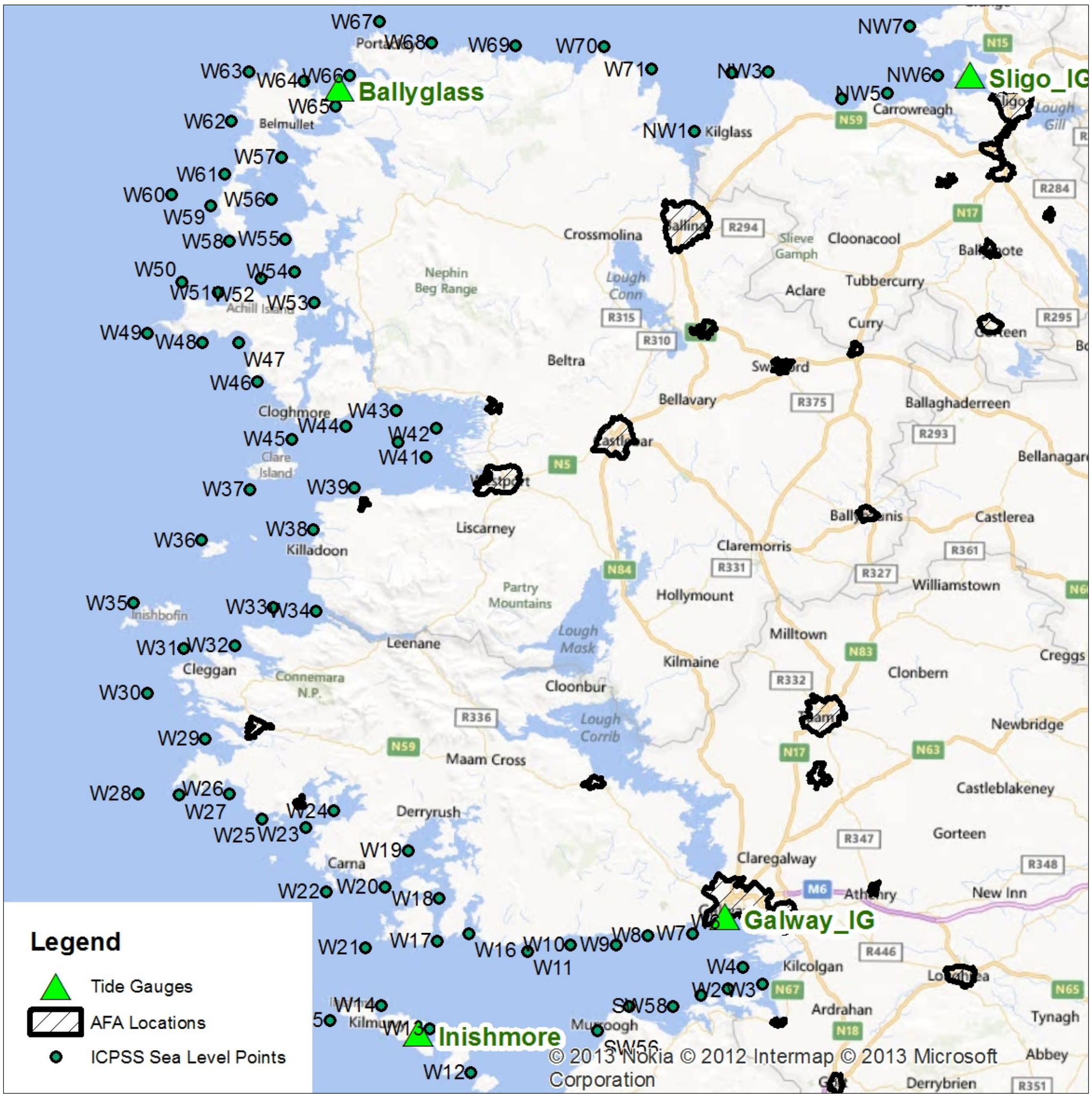
The ESLs used in the derivation of the design tidal-graphs were taken from the Irish Coastal Protection Strategy Study Phase 3 - West Coast[23] report; shown in Table 10-2 and Figure 10-1. These were based on a global tidal model developed by Kort and Matrikelstryreslen in Denmark.

Table 10-2: ESLs (mOD) for each respective study site

	Return Period (years)							
Location	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**1000**
Westport	2.79	2.96	3.09	3.21	3.37	3.49	3.61	3.88
Galway	3.06	3.21	3.32	3.42	3.56	3.67	3.77	4.02
Kinvarra	3.17	3.31	3.40	3.50	3.62	3.71	3.80	4.02
Sligo	2.50	2.64	2.73	2.82	2.94	3.03	3.12	3.33
Ballysadare	2.50	2.64	2.73	2.82	2.94	3.03	3.12	3.33
Ballina	2.44	2.56	2.64	2.72	2.8	2.91	2.99	3.18
Newport	2.85	3.03	3.16	3.29	3.46	3.58	3.70	3.99
Louisburgh	2.76	2.92	3.04	3.15	3.30	3.41	3.53	3.79
Clifden	2.69	2.83	2.94	3.04	3.17	3.27	3.37	3.60
Roundhouse	2.80	2.96	3.07	3.18	3.33	3.43	3.54	3.79

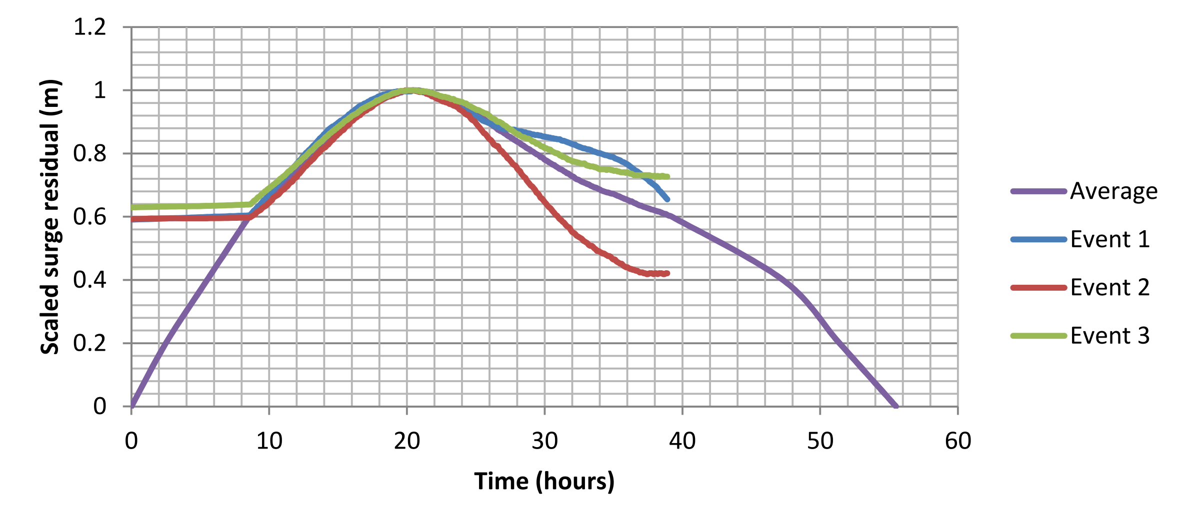
[23] OPW, 2011, Irish Coastal Protection Strategy Study Phase 3 – West Coast

Figure 10-1: West Coast ICPSS sea level points and tide gauges



Design surge profiles were derived from analysis of storm surge residuals supplied by the Marine Institute. The surge residuals from the largest three storm events (i.e. those resulting in the highest water level) were first identified. These three surge profiles were then normalised so all surge profiles peaked at one and then the average of these three profiles produced the design surge profile at each gauge. An example surge profile from Ballyglass is shown in Figure 10-2. Many of the large surge profiles were taken from periods of protracted storminess, leading to long periods of time with elevated surge residuals. In Figure 10-2 the surge residuals below 0.6m begin to plateau, therefore, to enable the extraction of a discrete surge profile for the design events, the levels below 0.6m were interpolated down to zero.

Figure 10-2: Surge profile analysis at Ballyglass



The underlying tide that will be used in the derivation of the design tidal graphs is the highest astronomical tide (HAT) profile, as predicted by the Admiralty Total Tide Software. Prediction sites recognised in Table 10-1 were extracted from the Total Tide software, with levels given to local chart datum.

With the above information collated, the design tidal-graphs were constructed by combining the design astronomical tide with the design storm surge. The peak of the storm surge was situated such that it occurred at low tide; this results in a more conservative tidal-graph, i.e. with a greater volume, than if the peak of the surge profile was situated at high tide. To demonstrate this it can be seen from Figure 10-3 that the overall volume of the design tidal curve is increased more if the peak of the surge is aligned with a trough of the underlying tidal series than if it was scaled to the peak of the tide. Effectively the peak of the event occurs on the falling limb of the surge resulting in a flatter, more prolonged tidal event as the peak of the surge passes through before the peak of the event.

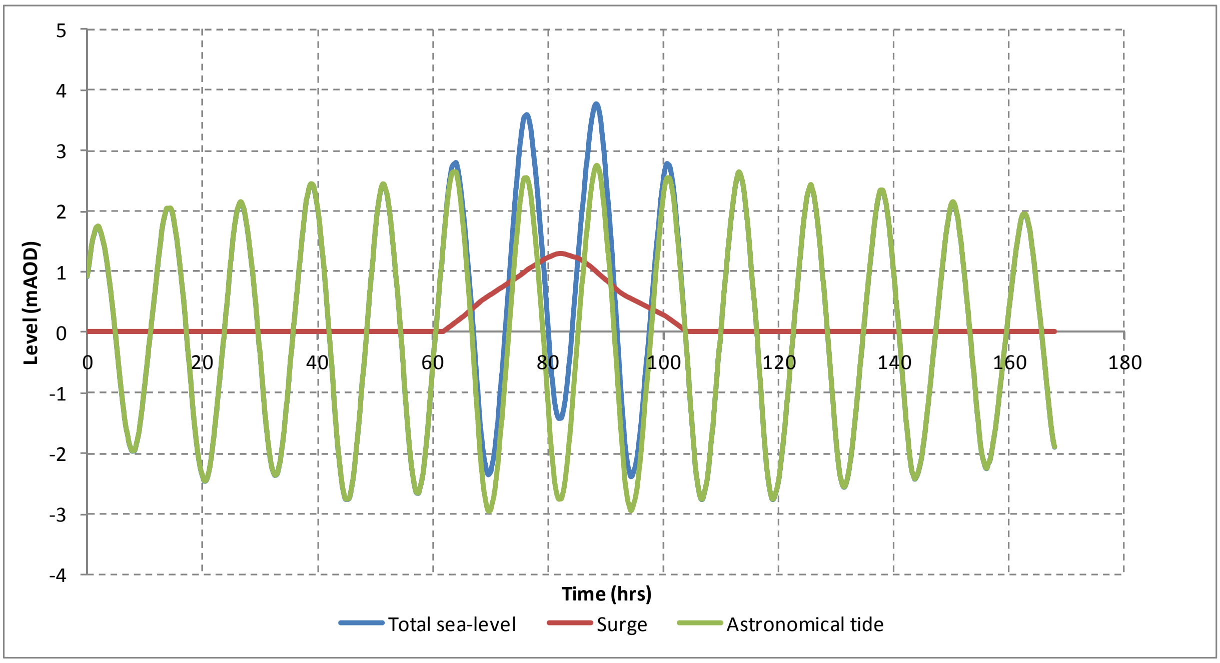
The design tidal curves were then corrected from Chart Datum, through Ordnance Datum Poolbeg to Ordnance Datum Malin Head. In recognition of the complexity of translating through three different datums a secondary correction factor of -0.15m or -0.1m was calculated in the Irish Coastal Protection Strategy Study and was applied to the design tidal curves. Table 10-3 shows the datum correction used at each study site. These corrections were applied so that the ESLs and tide data were in the same datum. The secondary correction is to allow for an error in the Malin datum correction that has been identified by the Irish Coastal Protection Strategy Study.

Table 10-3: Ordnance datum corrections used at study sites

Model location	From chart to Ordnance datum Poolbeg (m)	From Ordnance datum Poolbeg to Malin Head (m)	Secondary corrective (m)
Westport	0.11	-2.71	-0.10
Galway	-0.20	-2.71	-0.15
Kinvarra	-0.20	-2.71	-0.15
Sligo	0.69	-2.71	-0.15
Ballysadare	0.69	-2.71	-0.15
Ballina	0.72	-2.71	-0.15
Newport	0.11	-2.71	-0.10
Louisburgh	0.11	-2.71	-0.10
Clifden	0.00	-2.71	-0.10
Roundstone	0.00	-2.71	-0.15

As an example, the present day design tidal graph derived for a 0.5% AEP event for Galway is shown in Figure 10-3.

Figure 10-3: Design tidal graph at Galway for 0.5% AEP



10.3 Wave overtopping analysis

Wave overtopping has not been assessed at this stage of the project but will be covered under the hydraulics reporting.

10.4 Joint probability analysis

Joint probability analysis of the tidal and fluvial interactions has not been assessed at this stage of the project but will be covered under the hydraulics reporting.

11 Future environmental and catchment changes

11.1 Introduction

Specific advice on the expected impacts of climate change and the allowances to be provided for future flood risk management in Ireland is given in the OPW draft guidance[24], which calls for estimation of design flood parameters for two future scenarios, each intended to be a possible representation of flood conditions in 100 years time, i.e. around the year 2110:

- The Mid-Range Future Scenario (MRFS) is intended to represent a 'likely' future scenario, based on the wide range of predictions available and with the allowances for increased flow, sea level rise, etc. within the bounds of widely accepted projections.
- The High-End Future Scenario (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 increased flow, sea level rise, etc. at the upper bounds of widely accepted projections.

The scenarios encompass changes in extreme rainfall depths, flood flows, sea level, land movement, urbanisation and forestry. The allowances for each of these aspects, apart from urbanisation, are set out in the brief. The sections below set out how design flood parameters for the future scenarios have been defined.

11.2 Impact of climate change on river flows

The guidance states that flood flows shall be increased by 20% and 30% respectively for the MRFS and HEFS. This change has been implemented by scaling up the flood hydrograph for each HEP and for each probability by the specified percentage.

The brief also mentions impacts of climate change on extreme rainfall depths, but in the case of the UoM 30 there is no modelling of flooding directly from rainfall, and so there is no need to consider the change in extreme rainfall. It is already incorporated in the change in flood flows.

11.3 Impact of urbanisation

For urbanisation the approach adopted for the Western CFRAM is to calculate future urban growth patterns based on the core strategy for each county, which is in turn based on the settlement hierarchy detailed in the National Spatial Strategy (NSS)[25]. Although the plans and strategies do not extend to the 100 year horizon, they give an indication of where development is to be targeted for the plan period, which can be interpreted to be the likely focus of growth for the future.

The settlement hierarchy, as laid out in the NSS, has been reviewed, and the classification of each AFA in UoM 30 is shown in Table 11-1. Within the Western CFRAM area there are two gateways (Galway City and Sligo Town, including Oranmore and Willowbrook respectively), three hubs (Tuam, Ballina and Castlebar) and six smaller settlements which have been identified as having urban strengthening opportunities. It is in these 11 AFAs that urban growth will be focused over the plan period, and then over the next 100 years. An analysis of the Core Strategies for Galway City and County has shown a potential increase in housing land requirement of between 8 and 20%, based on the land shown as currently urban in the CORINE data set. In Sligo, development requirements are centred on Sligo town and environs, with a housing land requirement of 40 ha compared with 195ha across County Sligo; this target is centred largely on non-AFA settlements. A similar pattern of development requirement is seen in County Mayo, with a focus on the hubs of Ballina and Castlebar.

When reviewing the above analysis, the following should be borne in mind:

- No clear pattern was identified linking the percentage housing allocation to the rank of the settlement in the hierarchy.

24 OPW Assessment of Potential Future Scenarios, Flood Risk Management Draft Guidance, 2009

25 National Spatial Strategy for Ireland 2002-2020. The National Stationary Office

- The housing land targets span only the period to approximately 2020 (depending on the dates of the relevant Development Plan).
- The Development Plans themselves acknowledge that the land requirements are a conservative estimate (allowing for some 50% over zoning for market choice in development).
- Whilst it is possible to draw conclusions about the patterns of growth over the next 100 years, the scale of this growth is not known.
- All development plans include the requirement for SUDS to be included in new builds, so run off and flood generating potential should be reduced into the future.
- The aim of the guideline document, The Planning System and Flood Risk Management is to ensure flood risk does not become unmanageable within a catchment; over future development plan periods, SFRAs will be undertaken which will assess and reassess flood risks presented by planned development, and ensure those risks remain manageable.

Table 11-1 NSS Settlement Hierarchy

AFA	ID	County	NSS classification
Oughterard	300508	Galway	Centres in weak urban structure area
Galway City	300502	Galway	Gateway
Tuam	300510	Galway	Hub
Corrofin	300499	Galway	No classification

Future design flows have been tested using a future URBEXT value which is based on a percentage increase of the current URBEXT value, and then applying the urban adjustment formula developed in Flood Studies Update WP 2.3. The calculation involved first removing the effect of current urbanisation, converting the design flows to as-rural values, and then adding the effect of the possible future urbanisation.

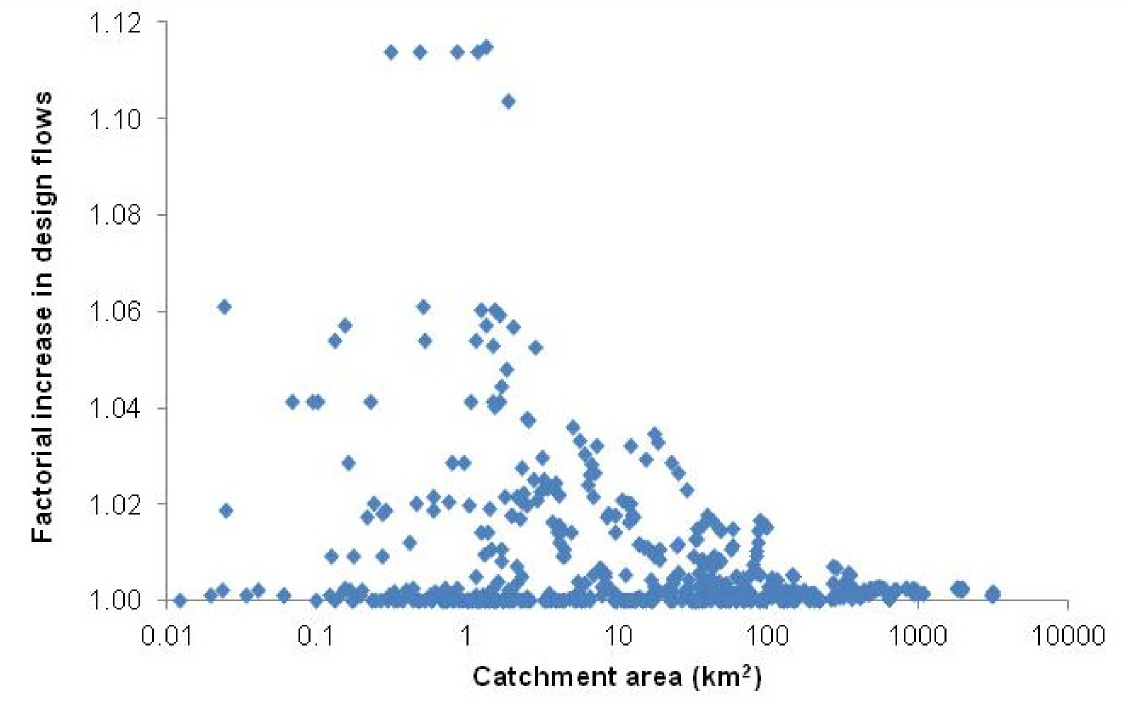
It should be noted that most methods that allow for the effect of urbanisation on design flows, including both the adjustment for QMED in the FSU and the allowances for time to peak and percentage runoff in the FSR rainfall-runoff method, are based on analysis of flood data from existing urbanised catchments. Most of these catchments include a wide range of development types, ranging from old town centres with no runoff mitigation measures to recent developments with SUDS or other measures aimed at restricting the runoff from the developed area. The downstream flooding impacts of future development should be minimised and so it is to be hoped that the allowances for the impact of urbanisation on future design flows represent a conservative worst case scenario.

For the majority of catchments the increase in flows is extremely minor, or non-existent as the existing urban proportion is extremely small, with little increase in QMED seen regardless of the scale of future urbanisation. Therefore for the MRFS a uniform 20% growth to URBEXT for all catchments has been applied, reflecting the maximum increase shown in the analysis of the core strategies, but recognising the capping factors on increases in flood risk discussed above. The maximum anticipated increase in QMED in this scenario is a factor of 1.11. The resulting increases in design flows are illustrated in Figure 11-1 which plots the factorial change in QMED (and hence in design flows for all AEPs) at every HEP in the Western CFRAM. The changes are plotted against catchment area, on a logarithmic scale. The plot shows how the application of a uniform increase in URBEXT results in a variable shift in flows; those catchments with a higher URBEXT value initially show the greatest increase in flows following the adjustment.

For the HEFS it is recommended that a uniform 30% growth to URBEXT is applied; this value has not been derived from the available data as described above but represents a conservative assumption in relation to the MRFS given the uncertainties associated with extrapolating this data over the 100 year time frame.

No change in the timing of the peak of the event as a result of the impact of urbanisation has been assessed. However the sensitivity of the models to changes in timings of the hydrographs is explicitly investigated within the hydraulic modelling reports.

Figure 11-1: Increases in design flows at each HEP as a result of future urbanisation



11.4 Impact of changes to forestry management

Changes to forestry management in a sub-catchment, either through deforestation or afforestation, can potentially influence flood risk by affecting surface water runoff. For the purposes of the Western CFRAM study the focus of interest is on the changes in practise that will in time result in an increase in flood risk downstream. This understanding will be used to inform the MRFS and HEFS.

Under the MRFS scenario outlined in the project brief, it is recommended that the impacts of afforestation are investigated through a decrease in time to peak of a sixth; this allows for potential accelerated runoff that may arise as a result of drainage of afforested land. This means the volume of water in the river is unchanged, but the rate at which it runs off the land into the watercourse is increased. The change in the time to peak can also have a positive or negative impact on flood risk depending on how it relates to the timing of peak runoff from contributing watercourses further downstream in the catchment.

Although the theory of forests acting as sponges soaking up water is popular, scientific studies have shown that the influence of forests on flooding and runoff is more complex[26]. Most of the well-known experimental hydrological studies of forestry have been undertaken in the UK, and have been on upland catchments, primarily investigating plantation forestry. In such cases, the effects of the forestry on runoff have been complicated by the influence of drainage ditches dug before the trees were planted.

Perhaps because of the complications of the crop cycle and management practices (such as drainage), there is little evidence from regional flood studies that the area covered by forest is a significant independent variable in the regression equations used for flood estimation[27]. However, this does not mean that forests have no effect on a local scale. Forests and forest soils (with their deep litter layer) are capable of storing and transpiring more water than grassland or arable crops. Therefore, where afforestation is occurring within a catchment, and in the absence of complicating factors such as drainage, one can expect a reduction in downstream flood volumes and an increase in time to peak.

26 UNFAO Center for International Forestry Research (2005). Forests and Floods. UNFAO.

27 Institute of Hydrology (1991). Plynlimon research: The first two decades. Report No. 109, Institute of Hydrology.

Applying the proposed MRFS changes to reflect the impact of afforestation globally to all HEPs across the study area will have a significant impact on peak flows, but this approach does not consider the spatial distribution of forests or the potential variability in runoff response over time across the Western CFRAM. Therefore to better understand the risks presented by changing land use patterns in the Western CFRAM area and to determine a more appropriate approach to the representation of changes in forest management in the MRFS and HEFS, a review of the distribution of the catchment characteristic 'FOREST' has been carried out. Although the area is largely rural, forestry practice is limited and is generally located in the upper parts of the river catchments, and tends not to form a large proportion of the land use on major rivers which flow through most of the AFAs.

Rather than apply a uniform adjustment factor to account for the impact of forestry, an analysis of each catchment has been carried out immediately upstream of the AFA. This reflects the fact that small scale changes in the upper catchments may not have an impact at the AFA downstream and often on a larger and less responsive river. Adopting a non-uniform approach also ensures that catchments which are largely urban are not also subject to forestry related changes in flow.

The HEPs upstream of an AFA were divided into three bands; those with a FOREST value of less than 25, 25-50 and over 50. Where FOREST is under 25 it was determined unlikely that any changes in forestry management would generate significant changes in flood risk, and certainly it would not be possible to say that any changes that were to occur would be linked to forestry; it is more likely that changes in arable farming practice or urbanisation would take place. A FOREST value of 25-50 shows a greater current forest cover, but one which is a combination of native woodland and managed conifer forests. Although changes to forest management practice in these catchments will occur, it is unlikely that sweeping changes would arise; instead the phased nature of forestry means that while some areas are cleared, others in the catchment are growing, thus balancing the impacts of drainage and felling. Whilst the changes in forestry management practices occurring in catchments with a FOREST value of greater than 50 are unlikely to have a combined significant impact, it was considered that there was enough of a potential impact to warrant further investigation. The only catchment where this was the case was in UoM35, and the impacts have been discussed in the relevant hydrology report.

It is therefore concluded that in UoM30 (and all others except 35) the likely impact of changes in forestry management practices are so uncertain, and relate to such a relatively small catchment area that the impacts should be excluded from the development of future the scenarios.

11.5 Sea level rise and land movement

Changes in sea and land levels in the Western CFRAM have been set out by the OPW at a national scale and no catchment specific changes are proposed as would be expected in these instances.

Sea level rise will be assessed by increasing levels by 0.5m and 1m in the MRFS and HEFS respectively. Land movement changes are only applicable for sites south of the Galway to Dublin line of which there are none within UoM 30.

11.6 Results: future flows

Design flows for the two future scenarios have been obtained by adjusting the present-day design flows, applying in combination the factors representing increases due to climate change and urbanisation but discounting forestry.

The overall factorial changes in design flow fall within the following ranges:

- For the Mid-Range Future Scenario (MRFS): from 1.20 to 1.34
- For the High-End Future Scenario (HEFS): from 1.30 to 1.53

Design peak flows at each HEP for both future scenarios are provided in Appendix F and with the digital deliverables associated with this report.

Associated with these flows, increases in sea levels of 0.5m and 1.0m will be applied for the MRFS and HEFS respectively.

12 Digital deliverables

12.1 Datasets provided with this report

Appendix F provides a table that lists the location of each HEP and the design peak flows for present-day conditions for the full range of HEPs. The table also provides a summary of how the flows were derived, i.e. the adjustment factor for QMED, the choice between a single-site or pooled growth curve and the distribution chosen for fitting the growth curve.

To avoid filling up the report with numerous long tables and to aid searching and copying of the results, more comprehensive results are provided digitally. The report is accompanied by the following digital deliverables:

- Shapefile of catchment descriptors for each HEP:

  This lists all the FSU catchment descriptors at each HEP. The source of the descriptors is recorded via the fields OPW_JBA (which distinguishes between descriptors taken straight from OPW's FSU dataset and those modified at JBA) and Node_ID (which records the name of the node in the FSU dataset on which descriptors have been based). This is relevant for very small catchments that do not appear in the FSU dataset. The AREA descriptor for each small catchment is calculated individually, but most other descriptors may be copied from a nearby FSU node.
- Shapefiles of catchment boundaries for each HEP:

  Catchment boundaries that have been created or modified by JBA are given in shapefiles with a name that corresponds to the label of the HEP. Catchment boundaries that have not been altered from the information supplied by OPW are in shapefiles that use OPW's naming convention (i.e. NODE_ID). A spreadsheet is included to enable cross-referencing between the label of each HEP and the corresponding shapefile NODE_ID.
- Shapefile of present-day design flows for each HEP:

  This gives the peak flows, as tabulated in Appendix F, but also contains more information on how the flows were derived, including the reference number of any gauging station located at the HEP, the reference number of any gauging station nearby whose single-site growth curve has been applied to calculate design flows at the HEP, and information on adjustment factors for QMED and growth curve derivation including FSR adjustment ratios as provided in Appendix F.
- Shapefile of future scenario design flows

Each of the above files covers all of the Western RBD.

In addition to the above the following files, which do not contain outputs from the hydrology study but have been included for information, have been supplied:

- A shapefile containing catchment descriptors for all gauges where catchment descriptors have been updated to reflect changes identified during the study
- A shapefile containing the surveyed watercourses.

Design hydrograph shapes are provided digitally in the form of inflows to the hydraulic models that are being developed.

13 Conclusions and recommendations

13.1 Conclusions

1. At all AFAs in UoM 30 it has been possible to derive design flows with reasonable confidence thanks to the presence of gauging stations for which records of peak flow and flood hydrographs are available. The exceptions to this are the small and medium-sized ungauged catchments, where no suitable donor catchments have been identified and QMED has been estimated from catchment descriptors only.
2. Design flows are expected to be more uncertain for low AEPs given the possibility that such extreme floods may arise from physical processes that do not make a significant contribution to events contained in the gauged records.
3. The methods of the Flood Studies Update have proved, in the main, straightforward to apply and suitable for the estimation of design flows on the wide variety of catchments in this unit of management. However, for design events greater than the 1% AEP, it has been judged appropriate to supplement the FSU methods with growth curves from the Flood Studies Report rainfall-runoff method.
4. Extreme tidal curves have been generated for Galway for use in inundation and wave overtopping modelling.

13.2 Recommendations

Several recommendations are offered at the conclusion of this report:

1. The design flows are suitable for the purposes of the Western CFRAM study.
2. At locations where the design flows are less certain (summarised in the Conclusions, above), future studies should consider the scope for improving the design flows. If significant flood risk is found to arise from any ungauged smaller watercourses, it is recommended that a flow gauging station is installed to allow future studies to estimate flows with more confidence.
3. The rating curves derived for UoM 30 would benefit from further check gaugings. Gaugings representing a variety of conditions at Corrofin (including the influence of the Abbert River, vegetation on the banks and blockage of the downstream structure), flows at Claregalway coinciding with high levels in Lough Corrib and high flows at Ballygaddy would improve confidence in the ratings.
4. It is recommended that any future flood studies include a review of ratings for Wolfe Tone Bridge, both for the period up to 2004 when the gauge was moved and the subsequent period for which a rating has not yet been established.

The two final recommendations are on the subject of the FSU methods:

5. It is recommended that further research is carried out aimed at improving the approach to derivation of characteristic flood hydrographs on ungauged catchments. It is difficult to have much confidence in the current method. The addition of a term representing catchment size would be of benefit, as would a study into the optimal way of identifying and using pivotal catchments to transfer information on hydrograph shapes.
6. It is recommended that OPW's recent research on small catchments is extended to examine the benefits (or otherwise) of adjusting QMED using donor/pivotal stations, given that there are rarely any nearby donor stations available on comparably sized catchments.

Appendices

A Rating reviews

1 Corrofin

1.1 Station description - from Inception Report

1.1.1 Gauge summary

Station name	Corrofin	**Site type**	Velocity-area
Station number	30004	**Watercourse**	River Clare
Grid reference	142640 243421	**Operator**	OPW

1.1.2 Location

The gauge is located on the left bank of the river immediately downstream of the road bridge.

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1.1.3 Gauge Datum

Gauge datum (mAOD)	21.98 (valid from 27/05/1975)
Means of confirmation (e.g. survey)	Supplied by OPW
Other comments (e.g. gauge boards)	Gauge board located on the gauging station housing structure

1.1.4 Description/ other comments

The gauge is located on the left bank of the river just downstream of the road bridge.

1.1.5 Control on stage discharge relationship

Type of section	Downstream face of bridge.
Low flow control(s)	At low flows the dominant hydraulic control will be the channel geometry 
High flow control(s)	At higher flows the control on water level will probably be the bridge. As water levels rise the bridge will exert a greater influence, particularly as the opening becomes surcharged. Another influence may be the confluence downstream. 
Bed slope	Local channel gradient has been estimated from 1:50,000 mapping to be approximately 0.0006.
Roughness	In general the in-channel hydraulic roughness will be low; however, in some locations densely vegetated banks will cause increased resistance once they have been overtopped.

1.1.6 Bypass routes

Out of bank flow will be very limited as the road leading to the bridge is raised above floodplain levels. The river channel is very deep and has the ability to take large volumes of water. There is also a substantial wall along the left bank which will limit out of bank flows on that side.

From the left bank, looking towards the upstream bridge face.	From the right bank, looking upstream along the right bank
	

1.1.7 Additional photographs

Looking downstream from road bridge	Looking upstream from road bridge
	
Looking downstream on left bank approx 300m upstream of gauge	Looking upstream (taken from Corrofin Bridge)
	

Looking upstream on right bank approx 200m upstream of gauge (the river channel can be seen on the right of picture)	Gauge and stilling well
	

Steep bedrock banks on the left bank approximately 10m upstream of the gauge.

1.2 Rating details

1.2.1 Check gaugings summary

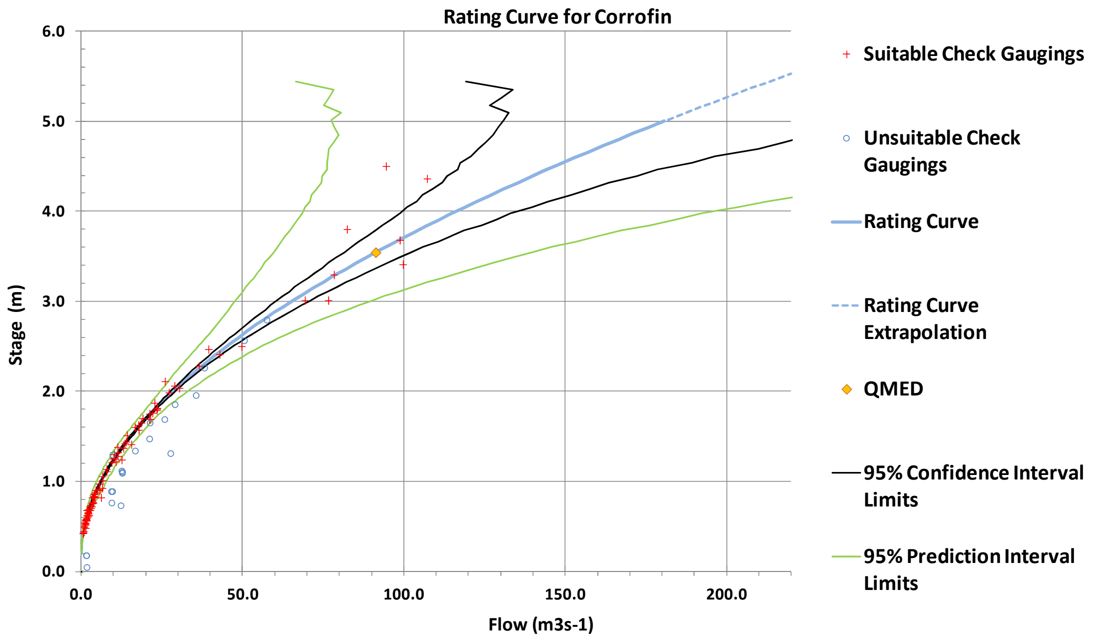
No. of gaugings	126 (104 since 1972 and 52 since 1988)	Date range	1972 - 2011
Maximum gauged stage (m)	4.50		
Approximate stage corresponding to QMED (m)	3.5	Extrapolation of rating to QMED (m)	n/a
Maximum observed stage (m)	5.32 (recorded in 1968; converted from old datum)	Extrapolation to highest flow (m)	0.82
Other comments	None		

1.2.2 Details of existing rating

Many ratings have been developed for this site and the current one is the second one provided with an ID of rating curve number 3. The current rating is considered applicable for data recorded after 1972. The rating is provided as having 5 limbs but the parameters for each of these limbs are the same. We have therefore applied a rating with a single limb to the data.

The parameters for the existing rating where $Q = C (h - e)^{\beta}$ are given below:

Limb No.	C	e	β	Min stage (m)	Max stage (m)
1	9.3000	0.1950	1.8900	0.0000	5.0000



Gaugings undertaken prior to 1972 (before which the current rating is not considered applicable) are included on the diagram above as "unsuitable check gaugings" but are not included in the statistical analysis.

1.2.3 Evaluation of existing rating

Overall agreement with check gaugings	The current rating effectively comprises a single limb which does appear to fit the check gaugings reasonably well.
Range of applicability	The supplied rating is considered reliable up to a stage of 5m. It is unclear how this limit has been derived but it is known that there is a bridge upstream (elevation unknown) which will affect the rating at high flows. It is possible that this limit is an estimation of when the influence of the upstream bridge may become significant. Extrapolation of the existing rating beyond this point should be undertaken with caution.
Stability of rating	There is a distinct difference between the upper and lower potions of the rating with regard to their stability (based on check gaugings). Below approximately 3m there is relatively little scatter within the data; consequently a high degree of confidence can be placed in the rating. Above this stage there is significantly more scatter evident within the check gaugings and the rating must be considered less reliable. The reasons for this increase in scatter are unknown but there are a number of possibilities. The most likely of these is that during large flood event the water level at the gauge site is also influenced by the downstream confluence with the Abbert River. Other possibilities include gauging errors (this is considered unlikely as the sole explanation given the constrained and accessible nature of the section), variation in hydraulic resistance of the banks (again unlikely due to the constrained nature of the section) and finally blockage of the downstream structure (this is possible, particularly during the larger events but should have been noted as the gauging was undertaken).
Uncertainty	The existing rating can be considered fairly reliable for lower flows (below approximately 3m); however, above this level the uncertainty increases rapidly. At QMED the 95% confidence interval has been estimated, by comparison with check gauging, to be approximately $10m^3/s$ or 22% of QMED.

1.3 Rating improvements

In order to improve our confidence in the high flows portion of the rating, in particular at flows higher than those that have been reliably gauged, we have developed a new hydraulic model. The following sections of this report describe how this model was developed and how the model results were used to derive an improved rating.

1.3.1 Summary of hydraulic modelling

Corrofin gauge is within the Clare River HPW for the Corrofin AFA. As such it will be included in the 1D-2D (ISIS-TUFLOW) hydraulic model of Corrofin. However for the purposes of the rating review a 1D only model has been used given the lack of floodplain flow in the area around the gauge.

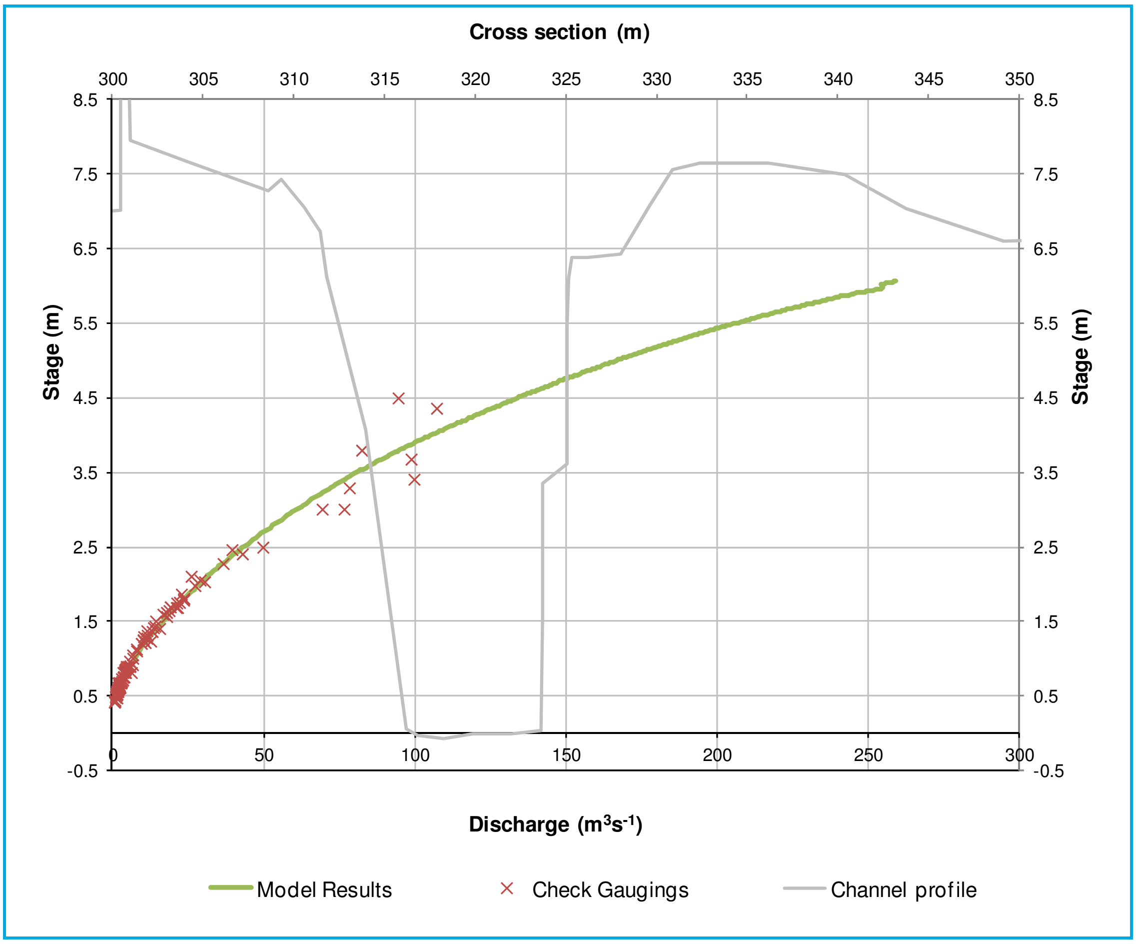
There is no formal channel bed control near the gauge. At low flows the river channel provides the hydraulic control on water levels. The channel is fairly uniform through the gauge location and maintained. The surveyed cross sections appear to provide an appropriate representation of the low flow channel control. There is a bridge immediately upstream of the gauge location which may provide some control at high flows. The confluence with the Abbert River downstream may also provide a degree of control on water levels. At higher flows the check gaugings are quite widely spread and the model bisects these quite well.

A channel Manning's n value of 0.040 has been adopted for the reach through Corrofin gauge in order to provide a rating curve with appropriate curvature to fit the range of check gaugings. Channel bank roughness is set to 0.070 given the dense vegetation.

The model run used to derive the rating curve was a single unsteady run using an estimated hydrograph shape starting at $2m^3/s$ and peaking at $260m^3/s$. The peak flow used gives a level over 0.8m above the highest recorded level at this site (November 2009).

The Abbert River joins the Clare River 2.5 kilometres downstream of Corrofin gauge. It has been suggested the flow in the Abbert River may influence the rating at Corrofin gauge. Some tests have been undertaken to determine the influence of this and it appears to be small. The Abbert River is ungauged so flows are not available to use in this investigation. Flow on the River Clare at the confluence is estimated to be around three times larger than the Abbert (based on QMED estimates from catchment descriptors). The rating curve model runs assume a peak flow of $50m^3/s$ on the Abbert River peaking at the same time as the River Clare. Doubling this to $100m^3/s$ increases the peak level at Corrofin Gauge by only 0.01m but a greater influence can be seen on the rising limb, up to approximately 0.1m. The impact is relatively minor and as the Abbert River is ungauged further work would seem unlikely to greatly improve the rating curve. It appears this will need to be accepted as additional uncertainty on the flow estimates at Corrofin gauge.

It is clear from these rating curve graphs that the model is able to replicate the hydraulic conditions at the gauge. It should be noted however the extrapolated rating curve is based on quite widely spread check gaugings, at fairly high flows. In addition the influence of the Abbert River is an additional uncertainty on the rating curve. This uncertainty in the rating was highlighted by the OPW during a review of the curve. The potential backwater influence from the confluence with the Abbert River has been investigated further and is detailed below.



Tests have been carried out varying the inflows and the downstream boundary with the 1D hydraulic model used for the rating review and with the 1D-2D model used for the AFA modelling. The tests looked at flows on the Abbert ranging between 5 and 100m^3/s and flows on the Clare ranging between 100 and 250m^3/s. The backwater effect is not apparent in the hydraulic modelling to anything close to the range of gaugings observed, particularly that from November 2009.

The gauging station is situated in a contained channel which drops around 4m relatively steeply between two areas of much flatter wide floodplain. This situation does not lend itself to a significant backwater effect as the level on the downstream floodplain is 2-3m lower than the gauge and the size of the floodplain means the volume required to raise water levels by the 2-3m to influence levels at the gauge is significant.

Without a backwater influence any change to the rating curve to try to meet the 2009 gauging will pull the curve away from the many more gaugings that fit closely to the modelled rating. These other gauging fall both sides of the modelled rating at mid-high flows and date over many years including recent ones in 2011, 2008 and 2006.

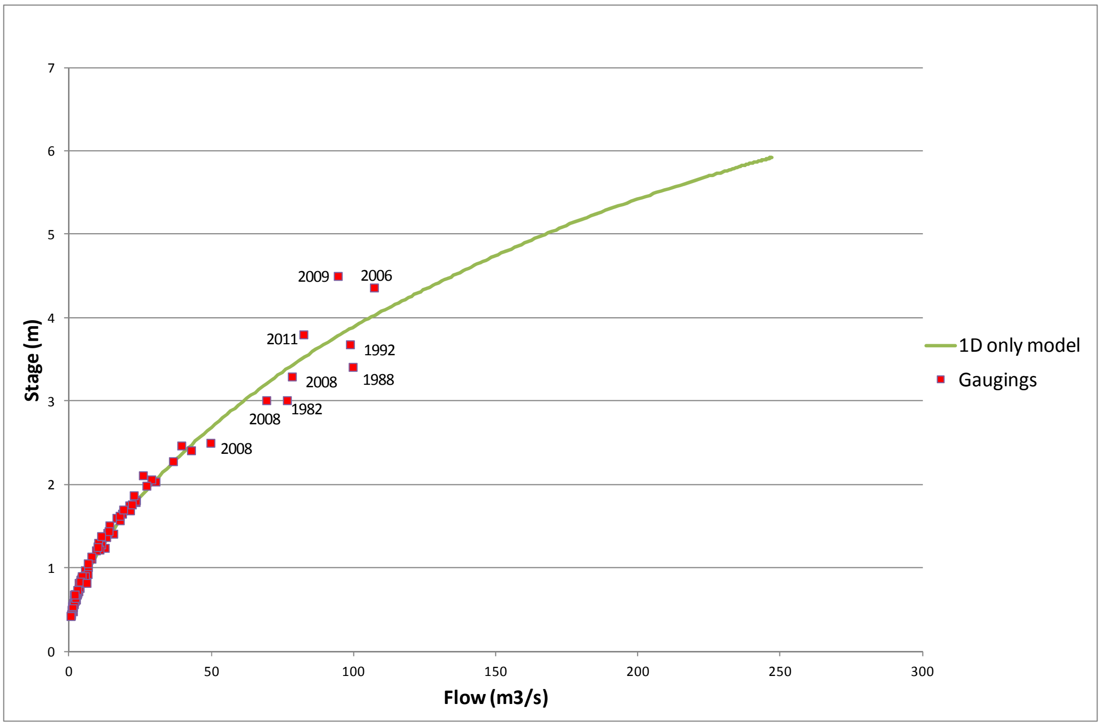
If a backwater influence were apparent a series of ratings could be developed to try to improve the flow estimates at Corrofin and relate those to flows on the Abbert but to do this would require a gauging station on the Abbert to be able to understand whether a backwater situation was likely to be developing. There is currently no recording gauge on the Abbert to give that information.

The flows at Corrofin derived from this rating, including for 2009 event, give modelled flood outlines that have been shown to Galway County Council and accepted as being appropriate, including property flooding in the Ballybanagher area to the upstream side of the gauge.

It is noted that use of the rating will give flows higher than those measured in 2009 and these do appear to give reasonable flood outlines. The accuracy of the check gauging is one possible explanantion of the discrepancy but not the only one. The other check gaugings in

recent years give a very different level-flow relationship. Achieving a single rating through all these is not possible. We have tested a possible influence of the Abbert and that does not appear significant. The other possibility is a groundwater influence upstream and/or downstream of the Corrofin gauge contributing to the flood extent and perhaps exacerbating a backwater effect through the gauge, particularly in 2009 (hence the low flow recorded for the level). A downstream gauge would be useful to confirm this but it may be that this is not a good site to have a flow gauge if a single flow-level relationship cannot be formed.

Given the information available and the model results JBA would recommend the modelled rating is retained for the CFRAM work.



1.3.2 Fitting a rating to the modelled results

A stage discharge-rating following a power law form has been parameterised based on the existing rating and the modelled stage-discharge relationship at the measurement section. The rating form applied is $Q=C(h-e)^{\beta}$ where:

h = river stage (m)

Q = river flow (m^3s^{-1})

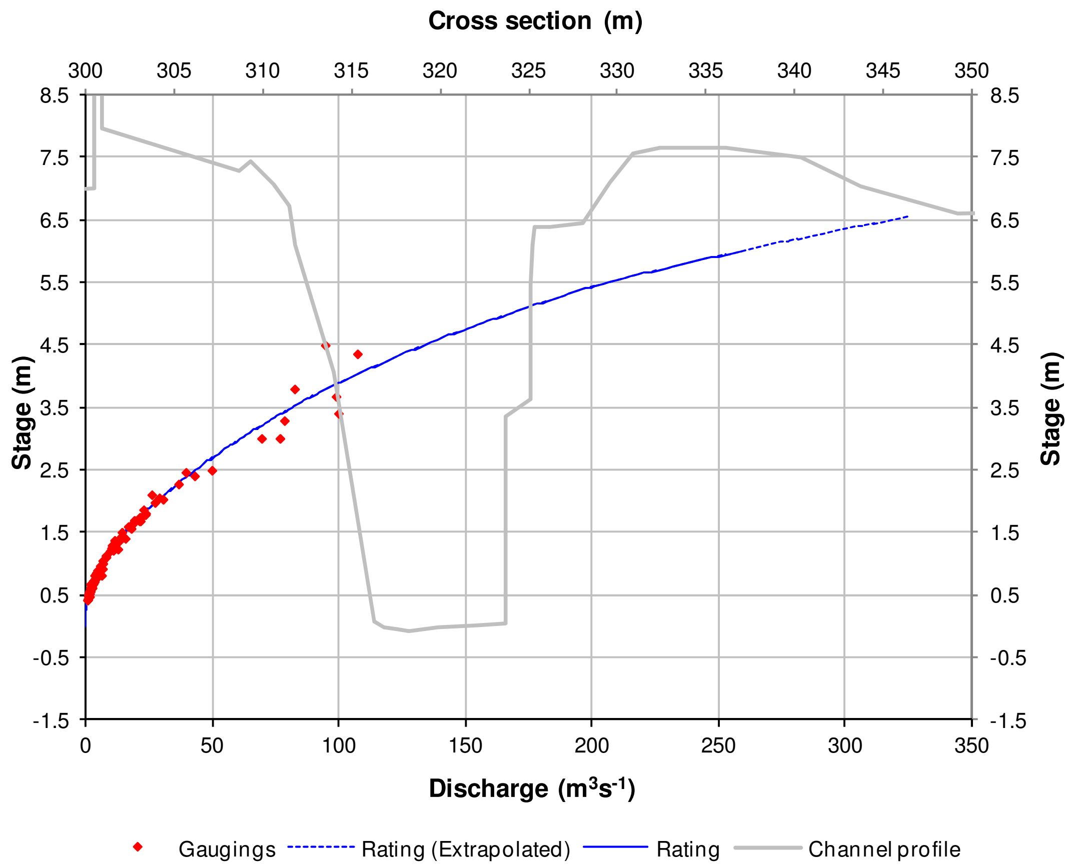
C, e, β are constants:

- the coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases.
- the coefficient β is related to the geometry of the channel and
- the coefficient e is related to the elevation of the bed relative to the gauge datum.

In fitting a power law to the modelled ratings, limb or segment breaks have been based on physical interpretation of hydraulic mechanisms and channel geometry, but only where supported statistically (evaluated based on the root mean square error). Fitting has been carried out using bespoke in-house rating curve fitting and evaluation software known as JRacuda. The proposed rating parameters can be seen in the table below with the rating's respective stage discharge pairs available in Section 1.4. The proposed rating form is visible in the graph below.

Limb	C	e	β	SG (min)	SG (max)
1	9.300	0.195	1.890	0.00	2.030
2	10.190	0.195	1.741	2.030	3.720
3	8.074	0.195	1.926	3.720	4.610
4	6.657	0.195	2.056	4.610	5.460
5	3.129	0.195	2.511	5.460	6.000

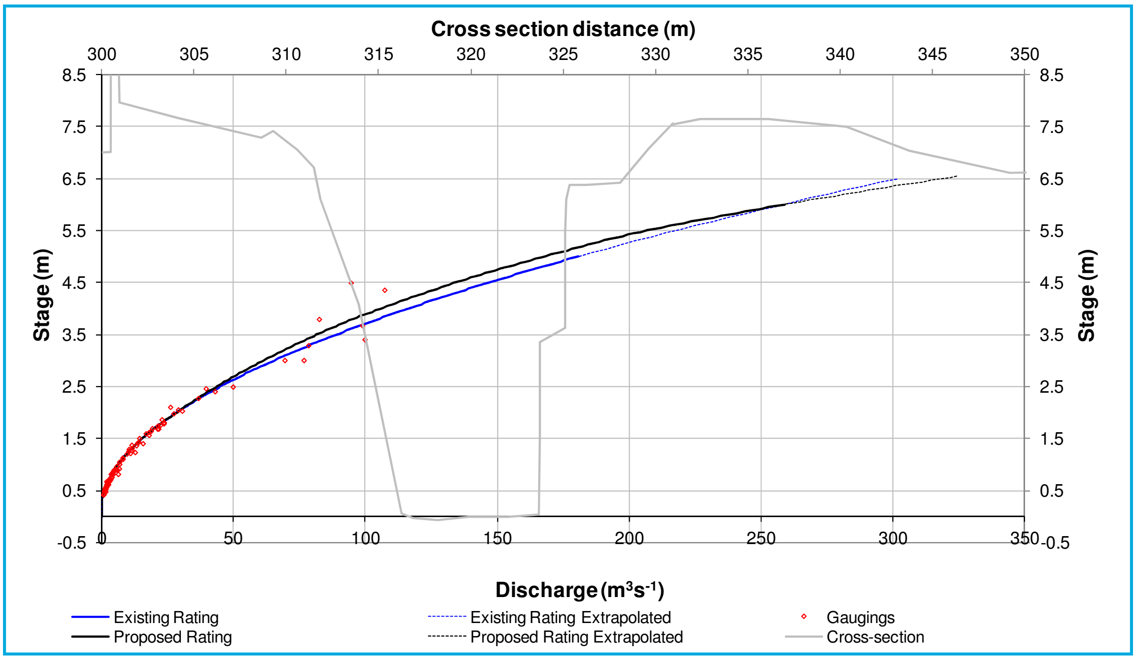
The proposed rating consists of five limbs to describe the hydraulic relationship at Corrofin up to a water depth of 6m above gauge datum (AGD). The proposed rating utilises the existing rating parameters to describe flows up to 29m^{3}s^{-1} before merging with the model derived rating. The hydraulic model allowed for additional detail to be included within the rating parameterisation and provide a more accurate representation of the hydraulic relationship at Corrofin than was previously possible (see graph below). As a result, the proposed rating contains four additional limbs to describe the full range of flows.



1.3.3 Comparison with existing rating

The graph below compares the existing with the newly proposed rating. The stage discharge relationship is identical below water depths of 2.03mAGD and comparable across the full range of the proposed rating until 6mAGD. Although the existing rating typically estimates a higher flow for a given stage, the deviation is relatively small being 7% and 9% at the highest and median annual maximum levels respectively. Beyond the upper limit of the proposed rating, the most extreme of events, extrapolation of the existing rating would estimate flows lower than that of the proposed rating. The AMAX series is not presented in this plot but can be found in Section 1.5.

Despite the relatively minor changes, the fact that the new rating is derived from a detailed, calibrated hydraulic model means we would recommend adopting this in preference to the existing one.



1.3.4 Overall agreement with check gaugings

The proposed rating fits very well to the available flow gaugings below 2.5mAGD. Above this level, there is greater scatter amongst check gaugings and as such increasing uncertainty in the suitability of the rating. The calibrated model provides a good estimate of the hydraulic relationship but there are no gaugings beyond 107m^{3}s^{-1} with which to calibrate the model or provide any validation during high flow conditions.

1.3.5 Range of applicability

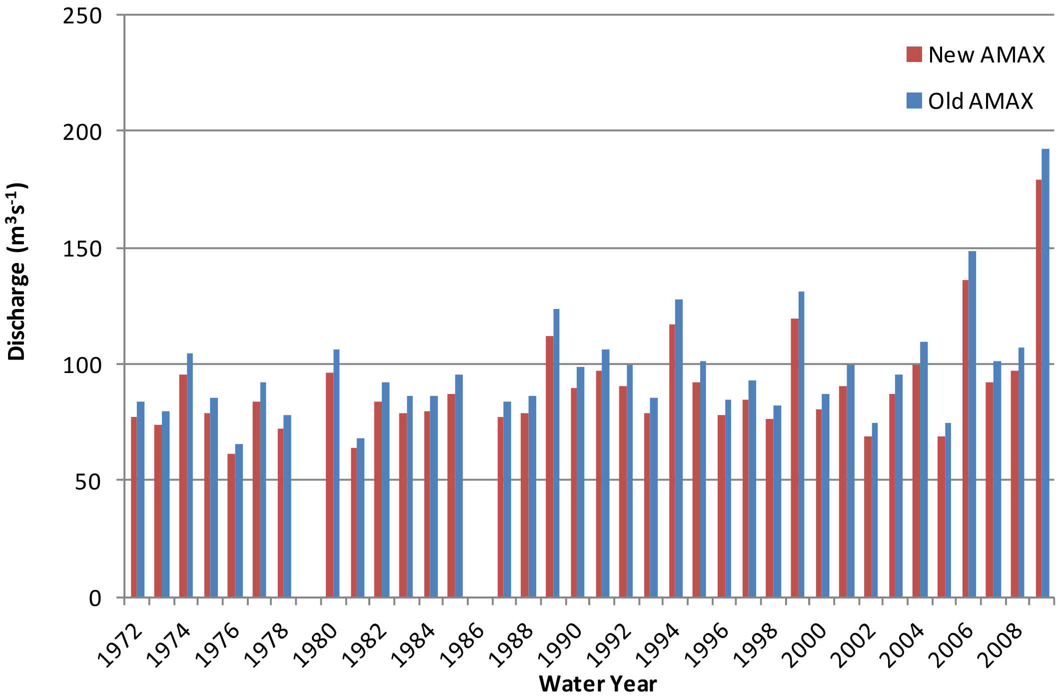
The upper limit of the rating is specified at 6mAGD, the peak stage value attained during the hydrodynamic simulation. For stage values beyond this there is less certainty on what form the rating will assume, although it is likely that the rating will plateau given the additional storage on the right hand bank. It should be noted, however, that this is unlikely as the channel is very deep and only the most severe of events will get out of bank at the station.

1.4 Proposed rating stage discharge pairs

Stage (m)	Flow (m^3s^{-1})	Stage (m)	Flow (m^3s^{-1})
6.0	258.962	2.9	57.618
5.9	247.906	2.8	53.960
5.8	237.139	2.7	50.405
5.7	226.659	2.6	46.954
5.6	216.462	2.5	43.608
5.5	206.547	2.4	40.367
5.4	197.713	2.3	37.234
5.3	189.983	2.2	34.209
5.2	182.412	2.1	0.000
5.1	174.999	2.0	28.394
5.0	167.744	1.9	25.494
4.9	160.646	1.8	22.742
4.8	153.706	1.7	20.138
4.7	146.923	1.6	17.684
4.6	140.336	1.5	15.381
4.5	134.265	1.4	13.230
4.4	128.324	1.3	11.231
4.3	122.511	1.2	9.388
4.2	116.829	1.1	7.701
4.1	111.276	1.0	6.172
4.0	105.854	0.9	4.804
3.9	100.562	0.8	3.597
3.8	95.400	0.7	2.557
3.7	90.459	0.6	1.685
3.6	86.014	0.5	0.986
3.5	81.664	0.4	0.465
3.4	77.410	0.3	0.131
3.3	73.254	0.2	0.000
3.2	69.196		
3.1	65.236		
3.0	61.377		

1.5 Impact on QMED and annual maximum series

The proposed rating typically results in a minor reduction in annual maximum flow estimates. Application of the proposed rating on available data from 1972 results in a reduction of the median annual maximum flood of approximately 9% from 93 m^3s^{-1} to 85 m^3s^{-1}. For the largest annual maximum flood in the 2009 water year, the flow estimate reduces by approximately 7% from 192 m^3s^{-1} to 179 m^3s^{-1}. The graph below illustrates what influence changing the rating has on the annual maximum series when applied across the period of record.



2 Ballygaddy

2.1 Station description - from Inception Report

2.1.1 Gauge summary

Station name	Ballygaddy	**Site type**	Velocity-area
Station number	30007	**Watercourse**	River Clare
Grid reference	142000 253772	**Operator**	OPW

2.1.2 Location

The station is located on the left bank of the River Clare, approximately 100m upstream of the R332 road bridge.

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2.1.3 Gauge Datum

Gauge datum (mAOD)	29.77 (Malin Head Datum)
Means of confirmation (e.g. survey)	Confirmed by 2012 survey
Other comments (e.g. gauge boards)	The gauge board is located adjacent to the gauge housing.

2.1.4 Description/ other comments

The gauge is located on the left bank just upstream of the control weir. The control weir was installed in 1974; prior to this the gauge was located in a natural channel. Both the left and right banks adjacent to the weir are vertical masonry walls and assuming the weir is in good condition (its crest was not visible during the site visit) it is unlikely that the geometry of the control has altered since it was installed. The gauge, weir and downstream bridge are all located on a relatively straight section of channel.

2.1.5 Control on stage discharge relationship

Type of section	Weir (believed to be a flat V weir but the profile was not visible during the site visit)
Low flow control(s)	The weir (located immediately downstream of the gauge) will provide a reliable hydraulic control during flows lower than bankfull. 
High flow control(s)	During higher flows a number of other factors will exert an influence on the stage discharge relationship at the gauge location. The main factors are thought to be the weir becoming non modular, the influence of the road bridge/embankment downstream and additional resistance on the floodplain. The R332 highway crosses both the floodplain and river channel approximately 100m downstream of the gauge. The road level is significantly higher than the river/floodplain level and the road is embanked as it crosses both the left and right bank floodplains. The bridge over the river channel is a single span sprung arch which will cause increased headloss as water levels rise. It is anticipated that this bridge will be the dominant hydraulic control during extreme flows but hydraulic modelling would be required to confirm this.

	
Bed slope	The bed slope along this reach has been estimated to be 0.0008
Roughness	In channel hydraulic roughness appears relatively low at the gauge location. However, the floodplains on both banks represent significantly more resistance to flow. In both cases the floodplains consist of isolated dense scrub and sparse woodland. Further upstream of the gauge there is pasture on both banks.

2.1.6 Bypass routes

The formal weir structure will be bypassed as soon as the bankfull level is exceeded. However, the bridge downstream (likely to be the hydraulic control in high flows) cannot be bypassed. Given the proximity of these two structures high flow gaugings may be more reliable if taken at the bridge.

Left bank floodplain (looking downstream)	Right bank floodplain (looking downstream)
	

2.1.7 Additional photographs

Gauging station (viewed from right bank)	Control weir (viewed from right bank)
	
Road embankment	R332 (looking south east)
	
R332 (looking north west)	Road bridge (viewed from control weir)
	

Gauge boards	Looking downstream from road bridge
	
High level gauge board	**Low level gauge board**
	

2.2 Rating details

2.2.1 Check gaugings summary

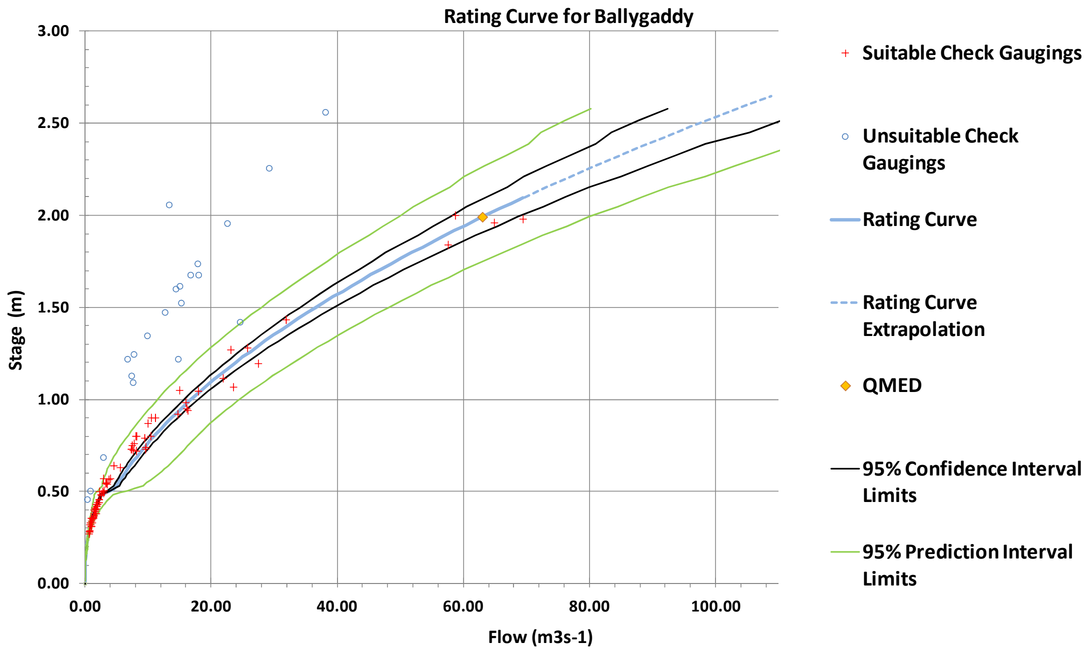
No. of gaugings	82 (after 1974)	Date range	1974 - 2011
Maximum gauged stage (m)	2.00		
Approximate stage corresponding to QMED (m)	1.99	Extrapolation of rating to QMED (m)	None
Maximum observed stage (m)	2.65	Extrapolation to highest flow (m)	0.65
Other comments	Numerous other gaugings are available but these were undertaken prior to 1974 when the weir was installed and are therefore not included in this analysis.		

2.2.2 Details of existing rating

The existing rating is the third rating derived for this gauge and is considered applicable for all data since the weir was installed in 1974. The rating is currently composed of two limbs (although it was supplied as four limbs with two pairs of identical equations).

The parameters of the rating where $Q = C\,(h - e)^{\beta}$ are given below:

Limb No.	C	e	β	Min stage (m)	Max stage (m)
1	16.500	0.005	2.500	n/a	0.490
2	17.000	0.005	1.910	0.490	2.095



In this case unsuitable check gaugings are ones that were undertaken prior to the installation of the weir. They are not used for analysis and are only included on the plot for completeness.

2.2.3 Evaluation of existing rating

Overall agreement with check gaugings	There is currently a very good correlation between the existing rating and the suitable check gaugings above 0.9mAGD; this is significantly below the lowest AMAX value making the rating suitable for use in the study. Between 0.5 and 0.9mAGD the rating appears to overestimate flows and there is a kink in the rating around 0.5mAGD where the rating comes back into line with the check gaugings.
Range of applicability	The existing rating is currently considered applicable to 2.095m. A more conservative estimate may be the highest gauged stage of 2.00m but in either case extrapolation of the existing rating much beyond this will result in much greater uncertainty.
Stability of rating	The current rating at the Ballygaddy gauge is considered to be relatively stable, due to the weir crest providing a formal hydraulic control. Scatter around the rating is most likely to represent errors in the gauging process.
Uncertainty	The 95% confidence interval at QMED is approximately 5.1m^3/s; this represents 16.3% of QMED.

2.3 Rating improvements

A hydraulic model has been developed to extend the high flow rating at Ballygaddy. The following sections describe model development and incorporation of modelled flows into a new rating.

2.3.1 Choice of modelling method

The weir downstream of the gauge provides a good low flow control whereas the road bridge further downstream is likely to be the most significant control at high flows. Hydraulic controls can be accurately represented in a 1D hydraulic model and can be collected efficiently by topographic survey in the form of cross sections. A 1D modelling approach is therefore considered the most appropriate method for representing in-channel flows. However, it is likely that the gauge will bypass at high flows and for this reason careful consideration was given to whether a 1D or 2D model would be most suitable for representing the out-of-bank portion of the model. The road downstream of the gauge is on an embankment which prevents any bypassing and will therefore provide the dominant hydraulic control. This could be represented equally well using either a 1D or a 2D approach, however a 1D model is more efficient to construct, easier to calibrate and does not require additional LIDAR data. For these reasons, 1D modelling software was also used to simulate floodplain flows bypassing the gauge with extended cross sections.

2.3.2 Summary of hydraulic modelling

Overview of model and location

Ballygaddy gauge is within the Clare River HPW for the Tuam AFA. As such it will be included in the 1D-2D (ISIS-TUFLOW) hydraulic model of Tuam. However for the purposes of the rating review a 1D only model has been used with extended cross sections representing floodplain flow.

The model developed for the purposes of updating the rating was built in ISIS (v3.6) and covers almost 500m of the River Clare.

Representation of hydraulic controls

There is a formal channel bed control at the gauge in the form of a weir. At low flows the weir will control water levels at the gauge. The weir is represented as a spill unit in ISIS. Floodplain bypassing of the weir is facilitated in the model with a parallel spill unit (with the channel blocked out). The spill coefficient for the channel spill unit is 2.5 to give the best fit to the check gaugings. This is a higher value than would usually be selected but it is used to give a good match to the high flow gaugings. The spill coefficient applied to the parallel spill is

1.7 (as head loss associated with bypassing is predominantly represented using hydraulic roughness in the adjoining sections).

At higher flows the bridge downstream of the gauge will also exert a control of the water levels. This is modelled using an ARCH bridge unit with skew correction applied in ISIS.

A channel Manning's n value of 0.035 has been adopted for the reach through Ballygaddy gauge in order to provide a rating curve with appropriate curvature to fit the range of check gaugings. Floodplain roughness is set to 0.045 and 0.055 in some areas.

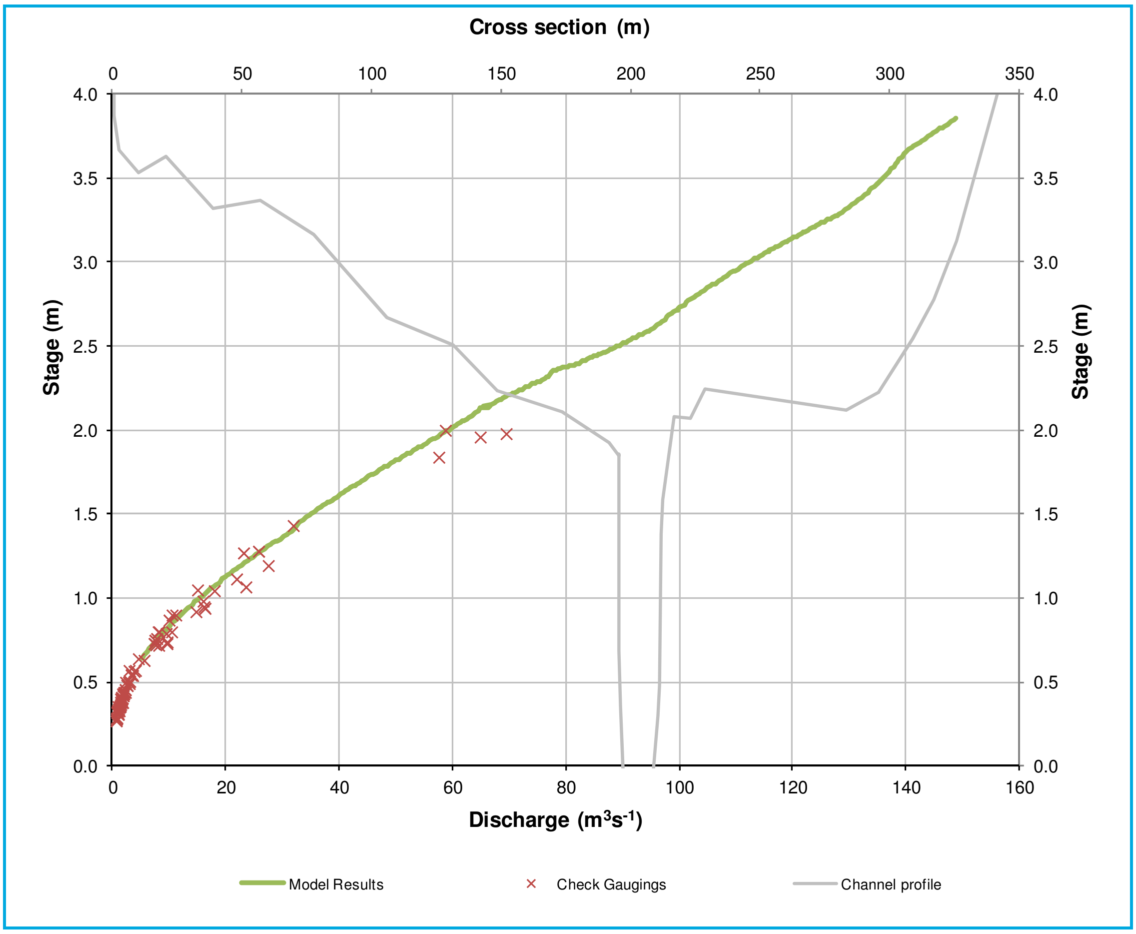
Hydraulic boundaries

The downstream boundary of the model was located approximately 185m downstream of the road bridge; the hydraulic conditions at the model boundary were defined using a normal depth boundary unit.

The model run used to derive the rating curve was a single unsteady run using an estimated hydrograph shape starting at 5m^3/s and peaking at 150m^3/s. The peak flow used gives a level over 1m above the highest recorded level at this site (November 2009).

Conclusions

It is clear from the rating curve graph below that the model is able to do a reasonable job of replicating the hydraulic conditions at the gauge up towards QMED where the highest gaugings are.



2.3.3 Fitting a rating to the modelled results

A stage discharge-rating following a power law form has been parameterised based on the existing rating and the modelled stage-discharge relationship at the measurement section. The rating form applied is $Q=C(h-e)^{\beta}$ where:

h = river stage (m)

Q = river flow (m^3s^{-1})

C, e, β are constants:

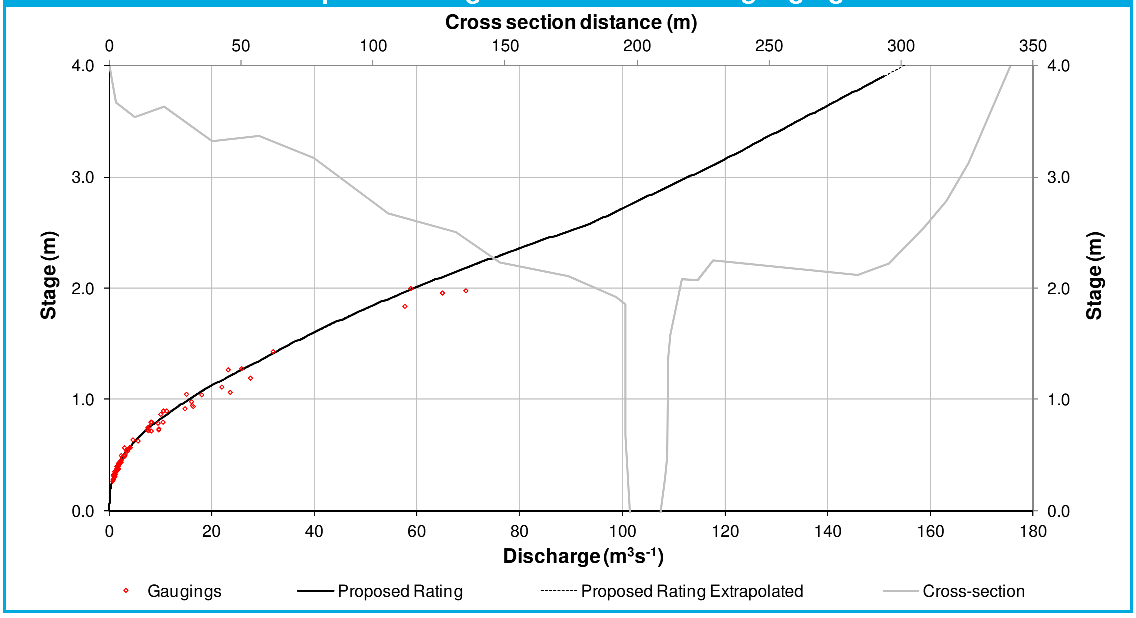
- the coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases.
- the coefficient β is related to the geometry of the channel and
- the coefficient e is related to the elevation of the bed relative to the gauge datum.

In fitting a power law to the modelled ratings, limb or segment breaks have been based on physical interpretation of hydraulic mechanisms and channel geometry, but only where supported statistically (evaluated based on the root mean square error). Fitting has been carried out using bespoke in-house rating curve fitting and evaluation software known as JRacuda. The proposed rating parameters can be seen in the table below with the rating's respective stage discharge pairs available in Section 1.4. The proposed rating form is visible in the graph below.

Limb	C	e	β	SG (min)	SG (max)
1	16.500	0.005	2.500	0.000	0.850
2	15.701	0.005	2.150	0.850	1.300
3	17.289	0.005	1.790	1.300	2.570
4	29.477	0.005	1.225	2.570	3.100
5	34.719	0.005	1.080	3.100	3.900

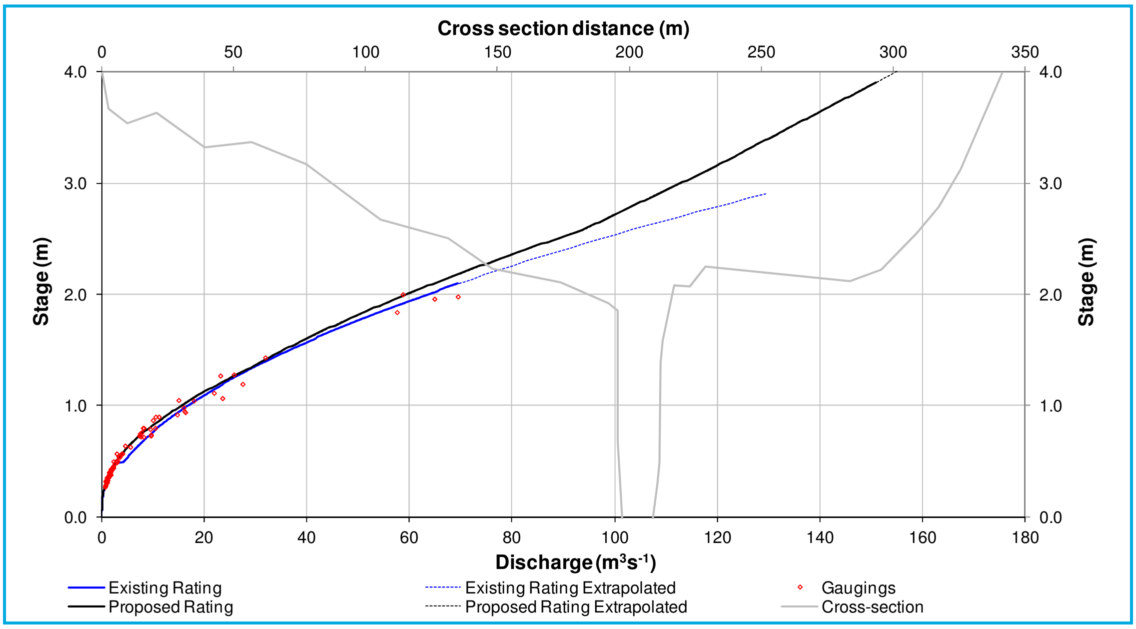
The proposed rating consists of five limbs to describe the hydraulic relationship at Ballygaddy up to a water depth of 3.9m. The proposed rating utilises the existing first limb to describe a greater low flow range up to 0.85m where it merges with the model derived rating. The existing second limb was omitted as it connected poorly with the rating's first limb and deviates from the model results beyond 1.4m when the weir downstream becomes drowned. Its inclusion for such a short stage range would have incurred unnecessary complexity within the final rating which can be adequately described with the five limbs presented in the table above.

Proposed rating with suitable check gaugings



2.3.4 Comparison with existing rating

The graph below compares the existing with the newly proposed rating. The stage discharge relationship is identical below water depths of 0.49m and largely comparable until 1.4m, beyond which the rating curves diverge. This divergence can be associated with non modular flow not being accounted for in the existing rating which results in the existing rating typically estimating a much higher flow for a given stage. At an approximate bank full stage of 2m, the existing rating estimates a flow ~7% greater than the proposed. As the existing rating is extrapolated out to much higher flows, the deviation becomes more pronounced; ~12% at the largest annual maximum event, a water depth of 2.65m, and 55% at depths of 4m. By simulating drowned flow at the weir downstream of the gauging station, the proposed rating will simulate high flow conditions with a greater degree of accuracy. The anticipated change in slope in the rating due to out-of-bank flows is not noticeable given the narrow, relatively constrained floodplain and high flow control exerted by the bridge downstream.



2.3.5 Overall agreement with check gaugings

The proposed rating fits very well to the available flow gaugings. However, the cluster of check gaugings at water depths of approximately 1.9m indicate that the proposed rating may be underestimating flows slightly (~10%). Beyond these gaugings the model results continue to estimate lower flows with greater accuracy compared to those derived from the existing rating; 12% at AMAX1. Extrapolating the existing rating further results in further deviation and there are no check gaugings available with which to calibrate the model or provide any validation during high flow conditions. Despite this, the hydraulic model is thought to provide a reasonable estimate of high flows in the absence of any additional information and it is recommended this rating is used for the study.

2.3.6 Range of applicability

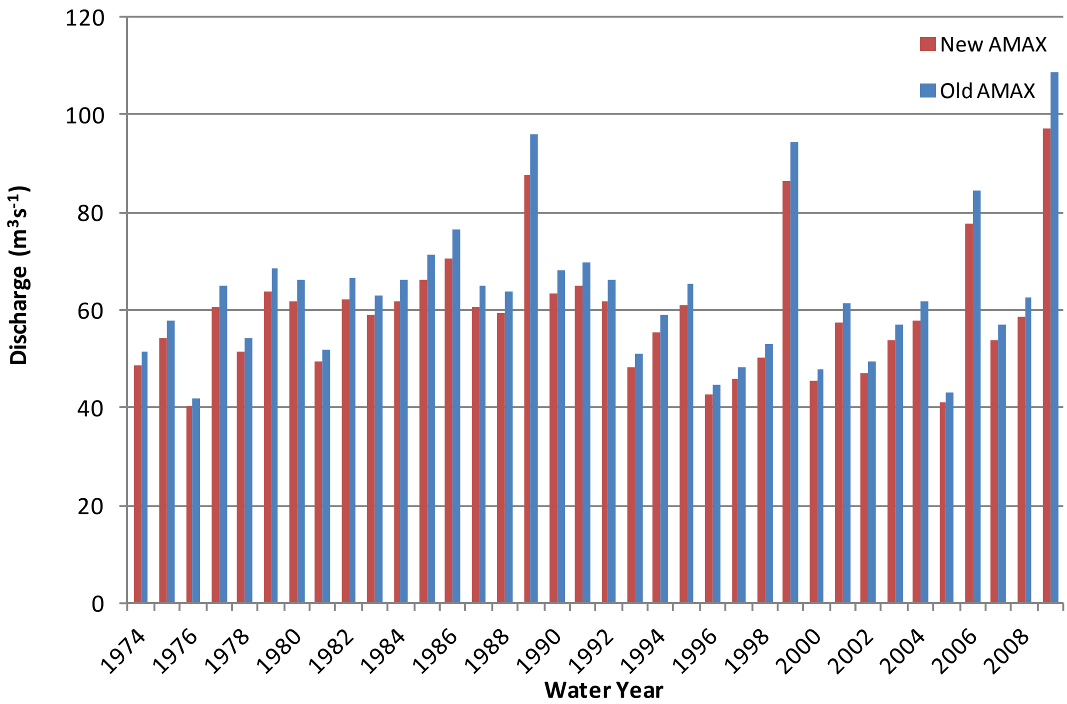
High confidence can be had in the rating across the gauged range, up to water depths of approximately 2m. Beyond this the hydraulic model is anticipated to provide a reasonable representation of the stage discharge relationship as it considers out of bank and drowned flow up to 3.9m, the peak stage value attained during the hydrodynamic simulation. For stage values beyond this there is less certainty on what form and gradient the rating will assume although it is likely that the upper limb will continue as the floodplain is already inundated by this point.

2.4 Proposed rating stage discharge pairs

Stage (m)	Flow (m^3/s)
3.9	150.855
3.8	146.675
3.7	142.504
3.6	138.341
3.5	134.189
3.4	130.045
3.3	125.912
3.2	121.788
3.1	117.646
3.0	113.007
2.9	108.402
2.8	103.833
2.7	99.300
2.6	94.805
2.5	88.822
2.4	82.551
2.3	76.483
2.2	70.621
2.1	64.966
2.0	59.520
1.9	54.286
1.8	49.265
1.7	44.461
1.6	39.876
1.5	35.512
1.4	31.373
1.3	27.371
1.2	23.028
1.1	19.084
1.0	15.533
0.9	12.370
0.8	9.298
0.7	6.644
0.6	4.506
0.5	2.844
0.4	1.618
0.3	0.780
0.2	0.277
0.1	0.046
0.0	0.000

2.5 Impact on QMED and annual maximum series

The graph below compares AMAX values for the period of record derived using both the old and the new rating. The new values are all lower than the old one with the biggest differences occurring at the largest flows. For the median annual maximum flood the reduction is approximately 6% from 63 m^3/s to 59 m^3/s. For the largest annual maximum flood, in the 2009 water year, the flow estimate reduces by approximately 11% from 109 m^3/s to 97 m^3/s. The graph below illustrates what influence changing the rating has on the annual maximum series when applied from 1975.



3 Claregalway

3.1 Station description

3.1.1 Gauge summary

Station name	Claregalway	**Site type**	Velocity-area
Station number	30012	**Watercourse**	Clare River
Grid reference	137302 233237	**Operator**	Galway Country Council

3.1.2 Location

The gauge is located on the right bank of the river immediately downstream of the road bridge.

EN 0021013 © Ordnance Survey Ireland / Government of Ireland

3.1.3 Gauge Datum

Gauge datum (mAOD)	5.724
Means of confirmation (e.g. survey)	Extracted from supplied 15 min level data
Other comments (e.g. gauge boards)	Gauge board located on the downstream face of the bridge.

3.1.4 Description/ other comments

The gauge is located on the right bank of the river just downstream of the road bridge. The road bridge has recently been reconstructed (in 2011-12) resulting in an altered channel section at the gauge location.

3.1.5 Control on stage discharge relationship

<table>
<tr><td>Type of section</td><td>Downstream face of bridge.</td></tr>
<tr><td rowspan="2">Low flow control(s)</td><td>There is a small informal weir downstream of the bridge (pictures below). This control is likely to become non-modular at even moderate flows.</td></tr>
<tr><td></td></tr>
<tr><td rowspan="2">High flow control(s)</td><td>At higher flows the small weir will drown and the open channel geometry downstream will become the hydraulic control. It has also been suggested that from the downstream side of the bridge the water level can also be controlled by elevated levels in Lough Corrib several kilometres downstream.</td></tr>
<tr><td></td></tr>
</table>

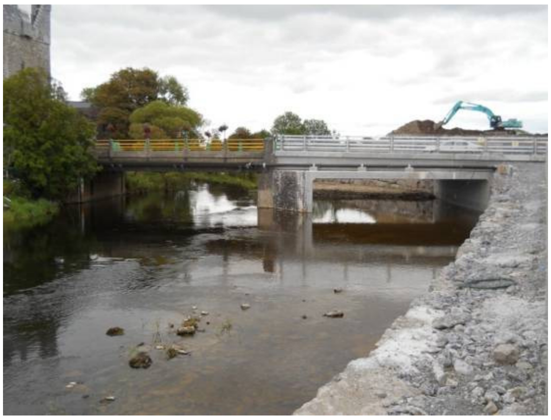
3.1.6 Bypass routes

Out of bank flow occurs on the left bank and this was recorded during the 2009 event. The new flood arch in the bridge should reduce the likelihood of this happening. The bypass route goes through the main Claregalway village and across the main N17 road.

Left bank floodplain upstream of N17	Looking from bridge onto left bank floodplain
	

3.1.7 Additional photographs

Looking downstream from road bridge	Looking upstream from road bridge
	

Looking upstream to gauge and bridge	Gauge on downstream face of bridge
	
Looking to left bank on downstream side of bridge	View from left bank floodplain to wards river bridge (in distance)
	

3.2 Rating details

3.2.1 Check gaugings summary

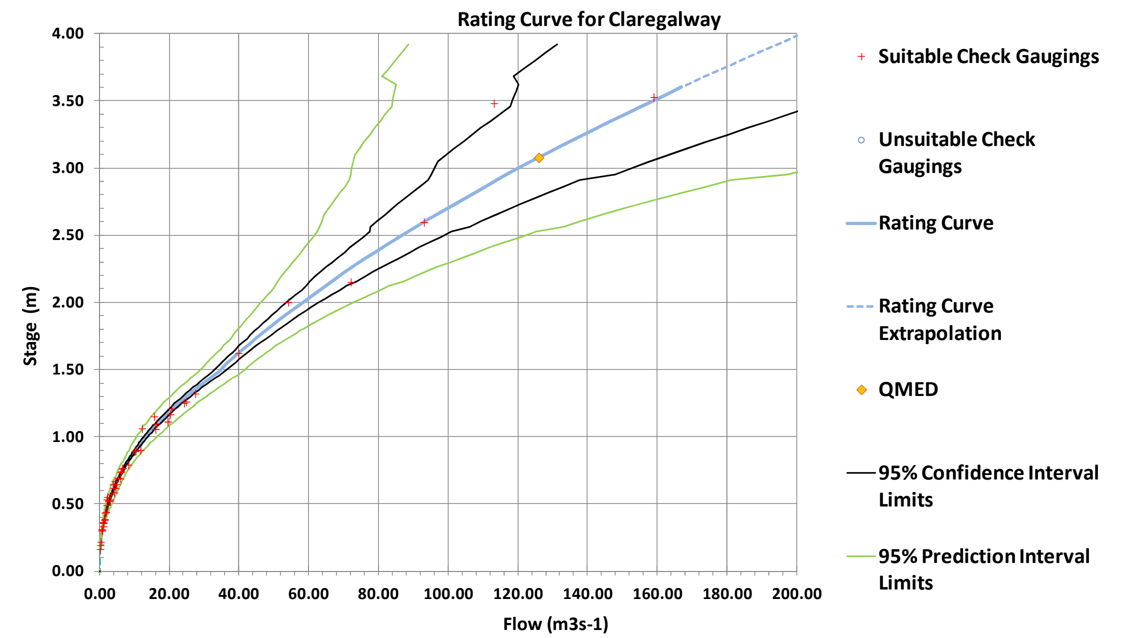
No. of gaugings	52	**Date range**	1989 - 2010
Maximum gauged stage (m)	3.53		
Approximate stage corresponding to QMED (m)	2.87	**Extrapolation of rating to QMED (m)**	n/a
Maximum observed stage (m)	3.58	**Extrapolation to highest flow (m)**	0.05
Other comments	The bridge has recently been rebuilt resulting in a modified section at the gauge location. The existing rating should no longer be applied to levels recorded at the gauge location. There is some concern[1] that the gauge does not reflect upstream water levels during periods of high flows. This will always be the case where water levels are gauged downstream of bridges and will be exaggerated as head loss through the structure increases at high flows. It should not cause any problems with the measurement of flows, but may be a concern if using the water level data to assess the likelihood of flooding upstream of the bridge.		

3.2.2 Details of existing rating

The existing rating is a compound rating comprising two limbs.

The parameters of the rating where $Q = C (h - e)^{\beta}$ are given below:

Limb No.	C	e	β	Min stage (m)	Max stage (m)
1	13.2788	0	2.39968	0.162	1.475
2	16.8162	0	1.79216	1.475	3.526



[1] Ryan Hanley Consulting Engineers (2010) Study to identify practical measures to address flooding on the Clare River.

3.2.3 Evaluation of existing rating

Overall agreement with check gaugings	Despite not being as many check gaugings as there are at some sites there is a good spread with check gaugings being undertaken close to the highest recorded flows. There is generally a very good agreement between the check gaugings and the current rating. However there is one notable high flow gauging which appears inconsistent with the others. This was taken on the 24 November 2009 and there is no comment attached to this gauging indicating any particular problems. Interestingly the highest gauging was taken 3 days prior to this during the same event. Given that the gauging does not appear to significantly affect the rating (only the confidence intervals) it has been retained in our analysis.
Range of applicability	The rating is considered suitable for data recorded between 1989 and 2011. The rating should not be applied to data recorded after 2011 as the channel section and bridge geometry has been modified.
Stability of rating	Generally the stability of the rating appears relatively good. However, there is some concern that when the lake level is high downstream then this may become the hydraulic control.
Uncertainty	Statistical analysis of the supplied data estimates a 95% confidence interval of $31m^3/s$ at QMED, this represents 49% of QMED. As discussed these confidence intervals appear relatively broad because of the effect of a single gauging. However, given there is no reason to dismiss this gauging and it may in fact represent a genuine uncertainty at high flows it is reasonable for it to be retained. At lower flows, there is far greater confidence in the flows derived using this rating.

3.3 Rating Improvements

A hydraulic model has been developed to extend the existing high flow rating at Claregalway. This required the development of a model that represents site conditions at the gauge prior to the bridge modifications. The following sections describe model development and incorporation of modelled flows into a new rating.

3.3.1 Choice of modelling method

The small informal weir downstream of the gauge described above will not provide an effective control at high flows. The downstream channel geometry and water levels in Lough Corrib are the most significant high flow control while the Curraghmore road bridge, approximately 6km downstream of the gauge, may also have some impact on high flows. Hydraulic controls can be accurately represented in a 1D hydraulic model and can be collected efficiently by topographic survey in the form of cross sections. A 1D modelling approach is therefore considered the most appropriate method for representing in-channel flows. However, it is likely that the gauge will bypass at high flows and for this reason careful consideration was given to whether a 1D or 2D model would be most suitable for representing the out-of-bank portion of the model. The channel and floodplain downstream of the gauge could be represented equally well using either a 1D or a 2D approach, however a 1D model is more efficient to construct and easier to calibrate. Floodplain flows were simulated using extended cross sections derived from LIDAR data.

3.3.2 Summary of hydraulic modelling

Source of data used to develop model

As detailed in section 1.2.4, channel geometry prior to the bridge works was required to develop a model for the existing rating. This was achieved by utilising cross section data from the HEC-RAS model supplied for the Clare River flood study (2010)[2].

LIDAR data was used to define floodplain geometry beyond the cross section extents incorporated into the previous HEC-RAS model.

Hydrometric data from Lough Corrib was used to derive a downstream boundary and to guide subsequent sensitivity testing of this parameter.

Model development and representation of hydraulic controls

The geometry of the open channel sections were extracted from the HEC-RAS model and used as the basis for the new model developed in ISIS (v3.4). The cross sections were then extended using LIDAR data. The new model comprises a total of 75 open channel sections, three of which are upstream of the gauge.

Two bridges were included in the model; the Claregalway Bridge, and the Curraghmore Bridge approximately 6km further downstream. Both bridges were represented in the model as USBPR bridge units with flat soffits and parallel spills used to represent both overtopping and bypassing of the structures. The geometries of both bridges were taken directly from the HEC-RAS model.

The downstream boundary of the model was located approximately 8.5km downstream of the gauge. The hydraulic conditions at the model boundary were defined using a stage-time boundary unit with a constant level defined by observed data from the Angligham gauge located on Lough Corrib. Selection and testing of representative downstream water levels at Lough Corrib are described below in section 1.3.3.

The initial estimates of hydraulic roughness applied to the model were taken from the original HEC-RAS model. Although attempts were made to refine these estimates in order to improve model calibration, the initial values provided the best fit to check gaugings. Therefore, the final n values used upstream of Claregalway Bridge were 0.1 and 0.04 respectively for the floodplain and in-channel parts of the cross section. Downstream of Claregalway Bridge, the final n values were 0.15 on the floodplain and 0.045 in the channel.

3.3.3 Downstream boundary sensitivity testing

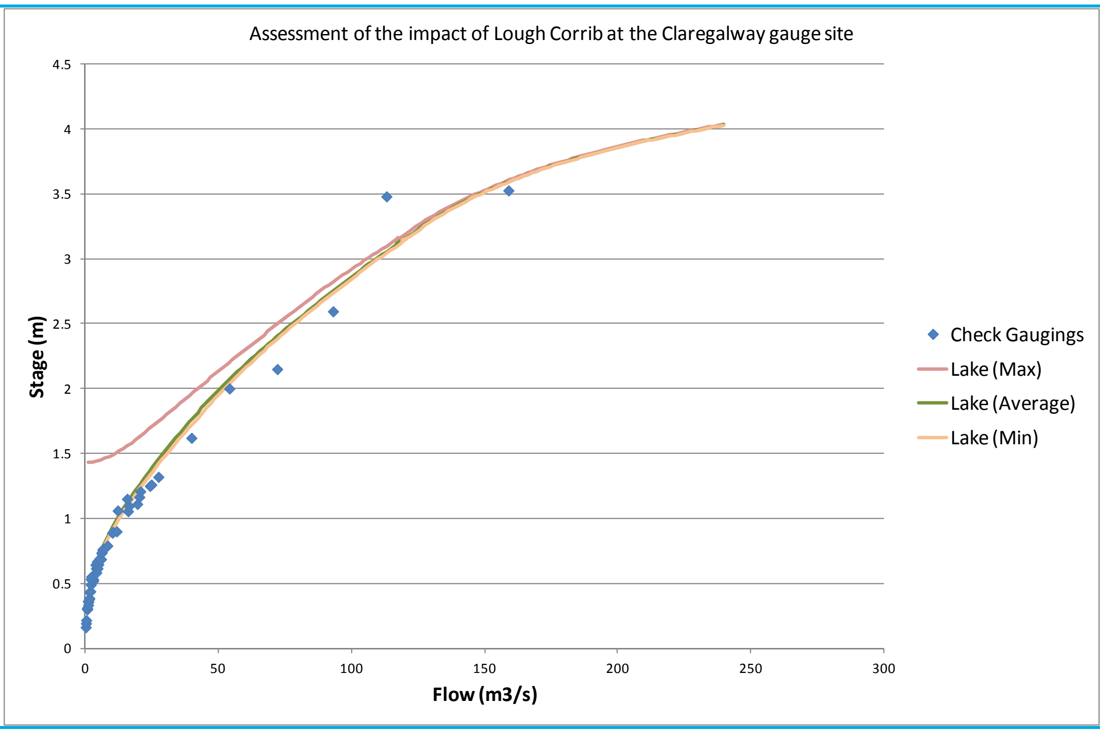
Water level data for the gauge at Angligham on Lough Corrib were received for the period 17/05/2002 to 19/03/2011. The minimum, mean and maximum levels (converted from Poolbeg to Malin Head datum for use in the model, Table 1) were extracted to test the backwater effect from the lake at the Claregalway gauge.

[2] Ryan Hanley Consulting Engineers (2010) Study to identify practical measures to address flooding on the Clare River.

Table 1: Lough Corrib Levels at Angligham gauge

Level	Poolbeg Datum (mOD)	Malin Head Datum (mOD)
Minimum	6.960	4.260
Mean	8.751	6.051
Maximum	9.854	7.154

The graph below shows the impact of varying lake levels at the Claregalway gauge location. The rating is clearly sensitive to lake levels greater than the mean for flows less than 100 m^3/s. Higher flows however are not affected by downstream levels.



To find a representative lake level to apply to the model, levels recorded on Lough Corrib during high flow events in the Claregalway AMAX series were analysed. From the period of overlapping AMAX and lake level data, three events were identified as detailed in Table 2 below, providing levels in the range 6.074 to 6.397mOD for use at the downstream model boundary.

Table 2: Lake levels associated with AMAX events

Date	Claregalway AMAX Level (mOD)	Claregalway Estimated Flow (m^3/s)	Angligham Level (mOD Malin)
11/11/2002	8.338	105	6.397
04/02/2004	8.463	115	6.274
10/01/2005	8.767	141	6.074

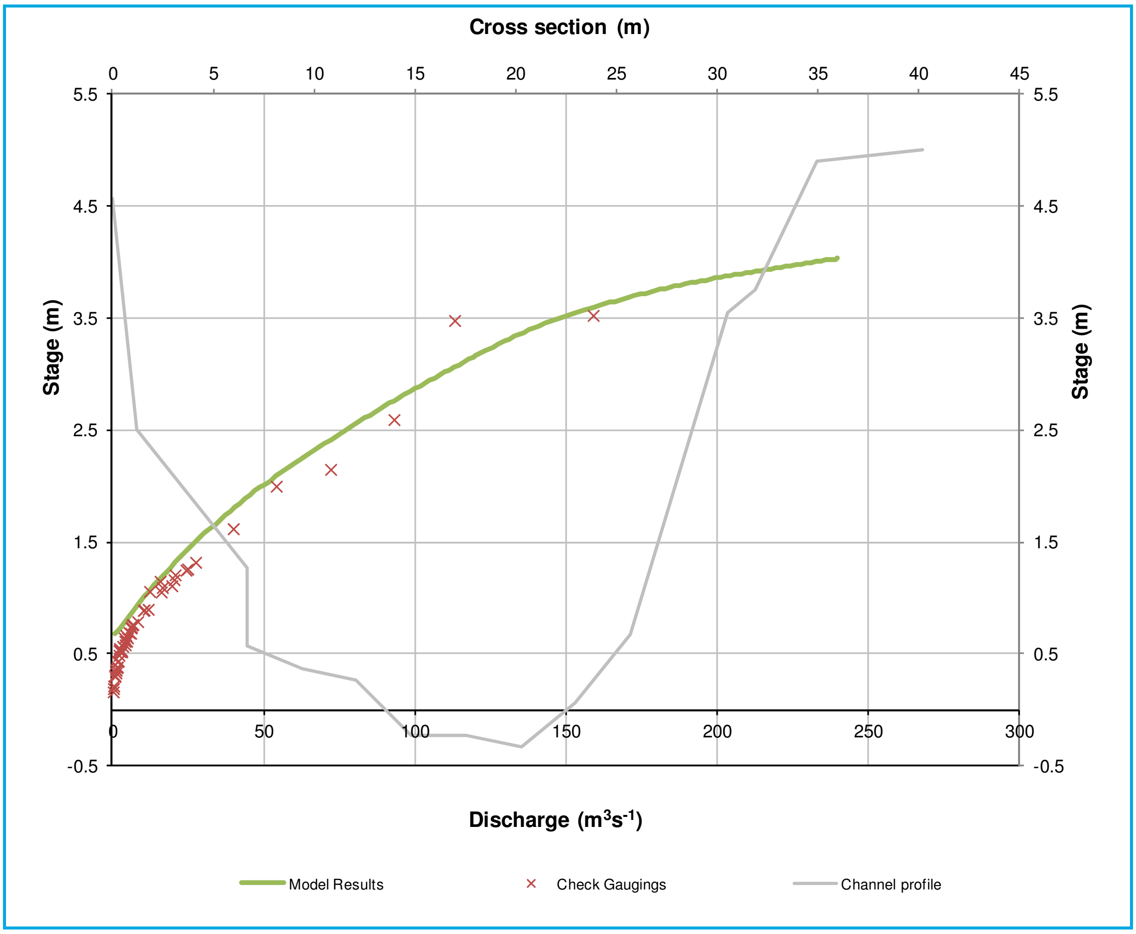
A similar exercise was undertaken to investigate lake levels associated with flow gaugings. From the period of overlapping gauged flows and lake level data, twelve events were identified as detailed in Table 3 below, providing levels in the range 5.877 to 6.577 mOD for use at the downstream boundary. Note that the highest lake level was recorded during a low flow event. Excluding this high level provides a mean lake level during gaugings of 5.988mOD.

Table 3: Lake levels associated with flow gaugings

Date	Claregalway Gauged Level (mOD)	Claregalway Gauged Flow (m^3/s)	Angligham Level (mOD Malin)
23/08/2006	6.087	0.982	6.577
13/07/2006	6.246	2.519	6.001
11/06/2004	6.255	2.964	5.877
18/06/2010	6.256	2.087	5.981
15/06/2010	6.274	2.220	5.962
29/06/2009	6.367	3.852	6.192
29/07/2002	6.391	4.364	5.954
10/03/2005	6.489	6.665	6.001
20/02/2006	6.819	16.417	6.051
25/11/2004	6.984	24.815	6.028
24/11/2009	9.204	113.148	5.914
21/11/2009	9.250	158.978	5.909

It is proposed to utilise the highest lake level identified above (6.397mOD) that is associated with high flow events recorded at Claregalway. Rounded up to 6.4mOD, this provides a representative downstream boundary water level for use in derivation of a rating for Claregalway.

The graph below plots both the modelled results and the supplied check gaugings. The model tends to underestimate discharge in relation to the gaugings, indicating some uncertainty in the ability of the model to replicate hydraulic conditions at the gauge. As the existing rating is to be retained up to a level of 3.73m (see section 3.3.4), this underestimation will not affect the final rating, and it has no effect on the annual maximum series since all observed floods are below 3.73m.



3.3.4 Fitting a rating to the modelled results

A stage discharge-rating following a power law form has been parameterised based on the existing rating and the modelled stage-discharge relationship at the measurement section. The rating form applied is $Q=C(h-e)^{\beta}$ where:

h = river stage (m)

Q = river flow (m^3/s)

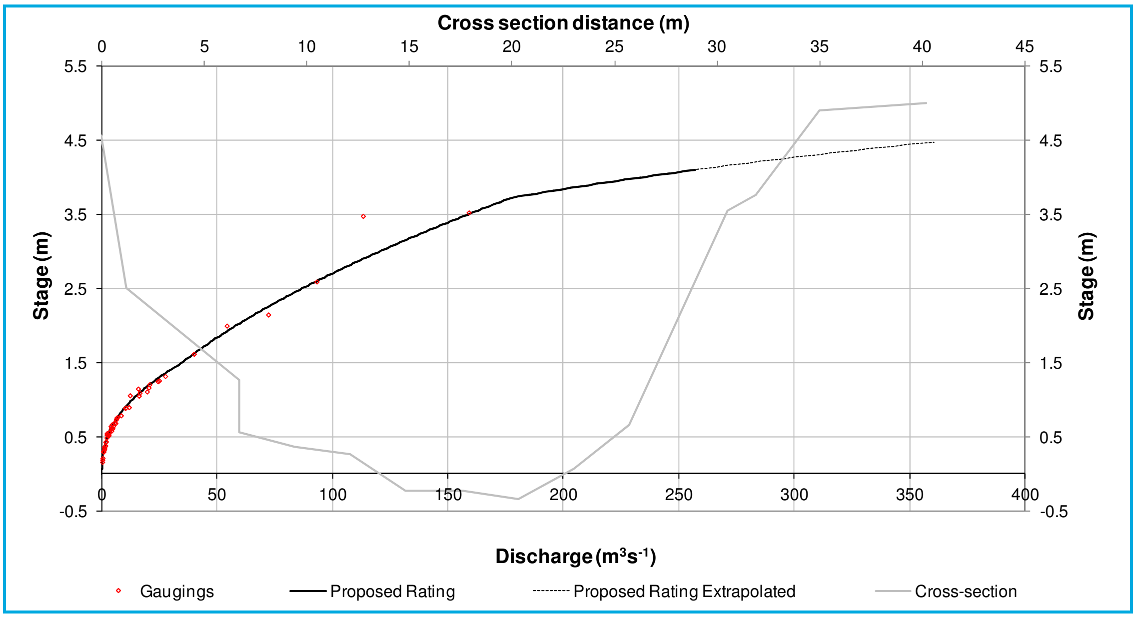
C, e, β are constants:

- the coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases.
- the coefficient β is related to the geometry of the channel and
- the coefficient e is related to the elevation of the bed relative to the gauge datum.

In fitting a power law to the modelled ratings, limb or segment breaks have been based on physical interpretation of hydraulic mechanisms and channel geometry, but only where supported statistically (evaluated based on the root mean square error). Fitting has been carried out using bespoke in-house rating curve fitting and evaluation software known as JRacuda. The proposed rating parameters can be seen in the table below with the rating's respective stage discharge pairs available in Section 3.4. The proposed rating form is visible in the graph below.

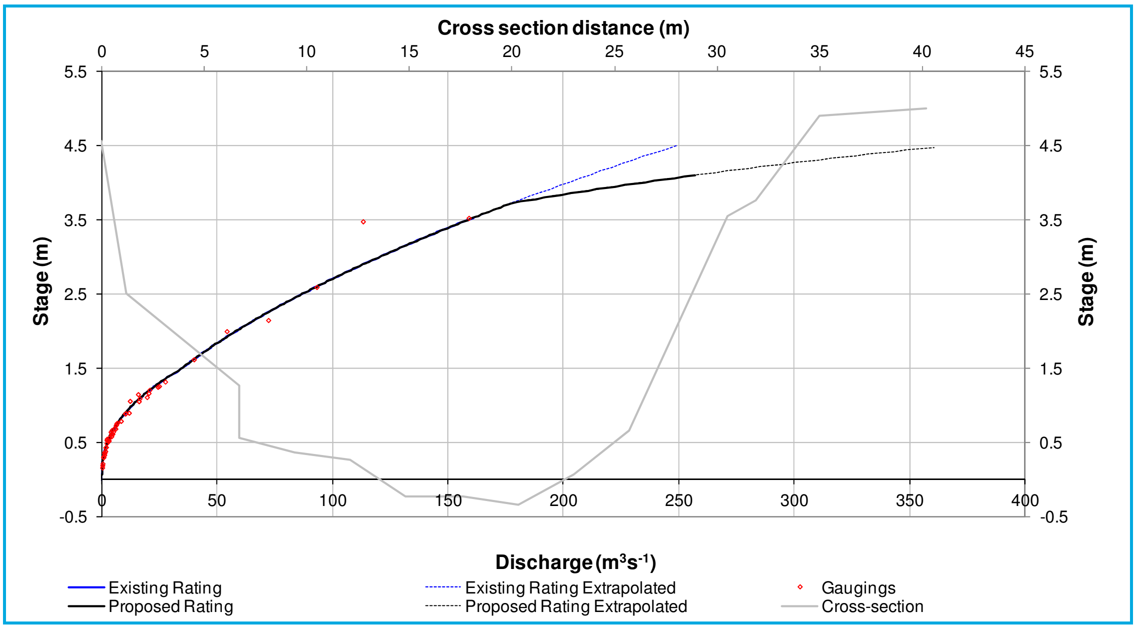
Limb	C	e	β	SG (min)	SG (max)
1	13.279	0.000	2.400	0.162	1.475
2	16.816	0.000	1.792	1.475	3.730
3	1.128	0.000	3.848	3.730	4.100

The proposed rating consists of three limbs to describe the hydraulic relationship at Claregalway up to a water depth of 4.1m. The proposed rating utilises the existing two limbs from the original rating to describe low to medium flows, which have an excellent fit to check gaugings, before using the hydraulic model results to derive the high flow limb.



3.3.5 Comparison with existing rating

The graph below compares the existing with the newly proposed rating. The stage discharge relationship is identical below water depths of 3.73m beyond which a new high flows limb derived from the model results is applied. The proposed high flow limb predicts much larger flows for a given stage than simple extrapolation of the existing rating; 16% and 48% higher flows at depths of 4 and 4.5m respectively. This is a result of the general flattening out of the modelled rating as flows come out of bank. The largest annual maximum flood on record is currently for the water 2009 year, 3.58m.



3.3.6 Overall agreement with check gaugings

The proposed rating fits very well to the available flow gaugings. The two highest gaugings at similar water depths of 3.48m and 3.53m do exhibit quite different flows of 113m^3/s and 160m^3/s respectively; a 40% increase in flow for 1% increase in stage. This uncertainty in check gaugings does reduce, to some extent, confidence in the trajectory of the rating at these depths. However, the hydraulic model is thought to provide a reasonable estimate of high flows in the absence of any additional information.

3.3.7 Range of applicability

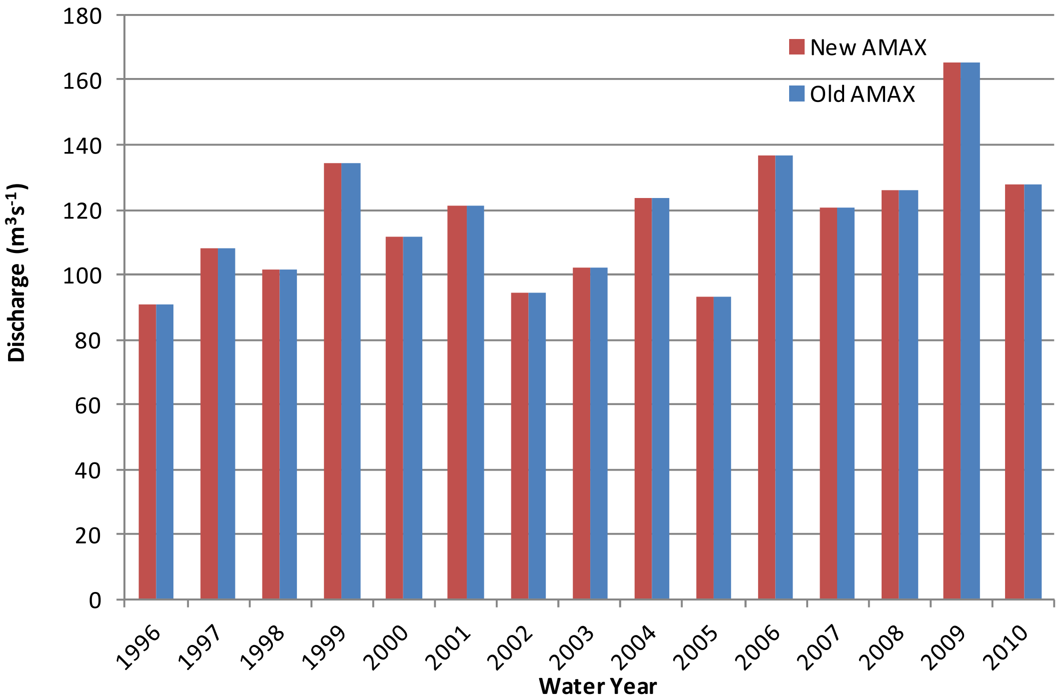
The most significant cause of uncertainty in the rating is lake level in Lough Corrib. The most significant errors will occur at times when the lake level is high (above 6.4mOD) and the flow is below 100m^3/s. However, at times when lake level is below 6.4mOD there is high confidence in the rating up to stages of 4.1m (0.37m higher than the 2009 peak) which corresponds to the highest modelled flow.

3.4 Proposed rating stage discharge pairs

Stage (m)	Flow (m^3/s)
4.1	257.150
4.0	233.844
3.9	212.139
3.8	191.963
3.7	175.403
3.6	166.998
3.5	158.776
3.4	150.738
3.3	142.885
3.2	135.219
3.1	127.740
3.0	120.450
2.9	113.349
2.8	106.440
2.7	99.724
2.6	93.202
2.5	86.876
2.4	80.747
2.3	74.817
2.2	69.088
2.1	63.562
2.0	58.240
1.9	53.125
1.8	48.219
1.7	43.524
1.6	39.043
1.5	34.779
1.4	29.773
1.3	24.922
1.2	20.567
1.1	16.691
1.0	13.279
0.9	10.312
0.8	7.773
0.7	5.642
0.6	3.898
0.5	2.516
0.4	1.473
0.3	0.739
0.2	0.279
0.1	0.053
0.0	0.000

3.5 Impact on QMED and annual maximum series

The proposed rating is identical to the existing until a water depth of 3.73m. All recorded annual maximum flood events fall below this threshold and, as such, there is no change in the annual maximum series (see graph below) or median annual maximum flood flow estimate (121 m^3/s).



4 Claremount

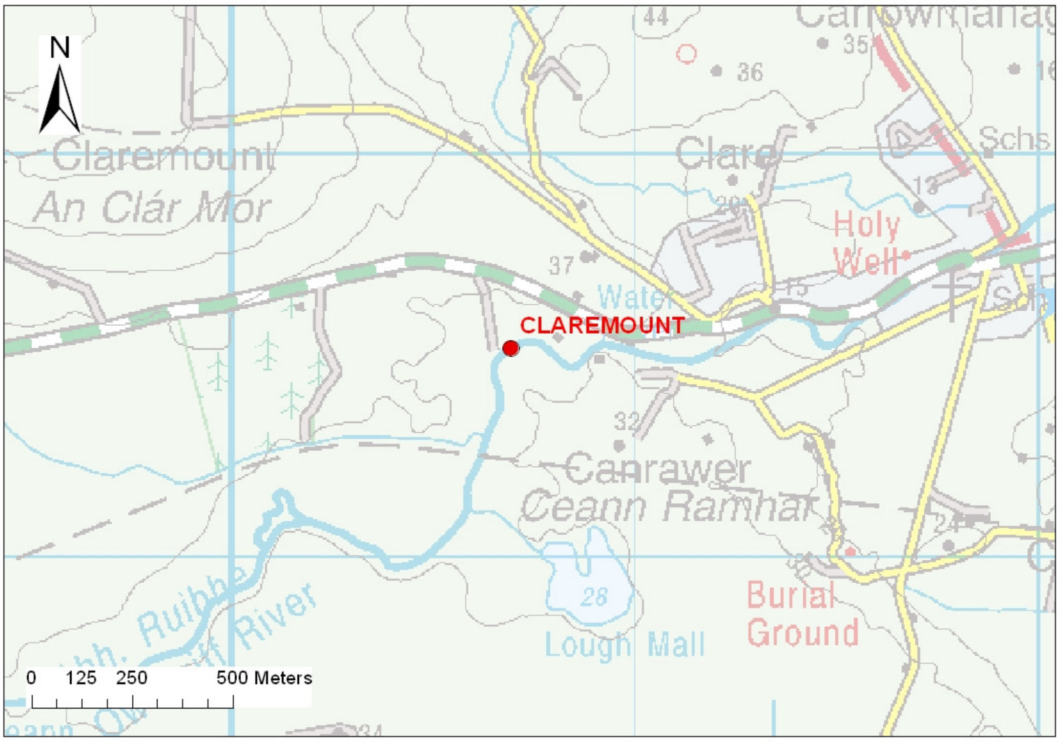
4.1 Station description - from Inception Report

4.1.1 Gauge summary

Station name	Claremount	**Site type**	Velocity-area
Station number	30019	**Watercourse**	Owenriff River
Grid reference	110693 242516	**Operator**	OPW

4.1.2 Location

The gauge is closed and has been removed. It was located upstream of Oughterard, approximately 1.3km upstream of the N59 bridge.



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4.1.3 Gauge Datum

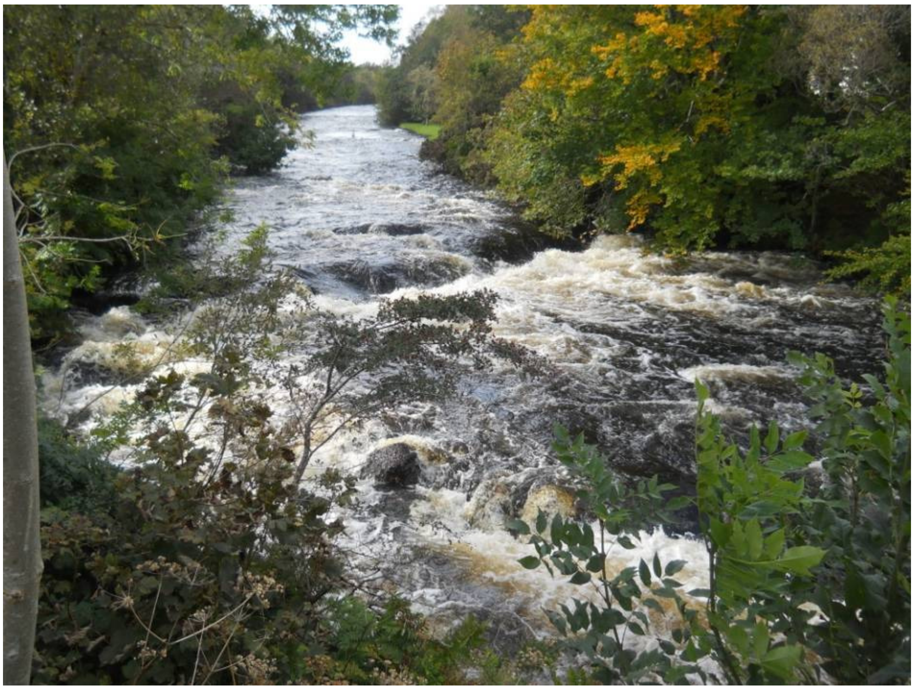
Gauge datum (mAOD)	27.21 to Poolbeg datum
Means of confirmation (e.g. survey)	Supplied by OPW
Other comments (e.g. gauge boards)	Gauge closed, no evidence of it remains.

4.1.4 Description/ other comments

Gauge closed since 2003, no evidence of gauge remains.

4.1.5 Control on stage discharge relationship

Type of section	Open channel.
Low flow control(s)	At low flows the dominant hydraulic control will be river channel. About 400m downstream the river flows over a series of waterfalls and cascades this may exert some hydraulic control at the gauge site. 
High flow control(s)	At higher flows the dominant hydraulic control will be river channel. About 400m downstream the river flows over a series of waterfalls and cascades which still may provide some degree of hydraulic influence at the gauge. Photo below shows view upstream from top of cascades but gauge location is out of sight.



4.1.6 Bypass routes

Out of bank flow will be limited as channel is fairly deeply incised into floodplain. The left bank may offer greater bypassing potential on the floodplain.

View across river from left to right banks	View towards river over the left bank floodplain
	

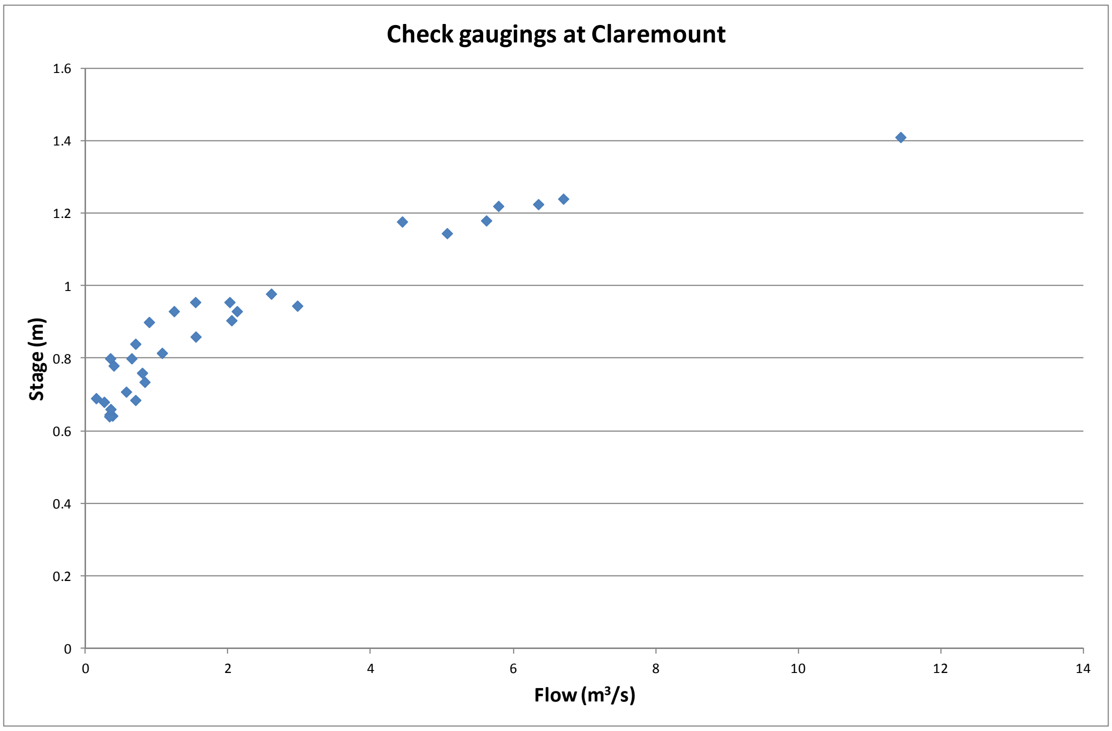
4.1.7 Additional photographs

View over left bank floodplain towards river	View over left bank floodplain towards river
	
Towards cascades 400m downstream of gauge	**Looking upstream from cascades, 400m downstream of gauge**
	

4.2 Rating details

4.2.1 Check gaugings summary

No. of gaugings	31	Date range	1976 - 2001
Maximum gauged stage (m)	1.41		
Approximate stage corresponding to QMED (m)	2.08	Extrapolation of rating to QMED (m)	0.67
Maximum observed stage (m)	2.95	Extrapolation to highest flow (m)	1.54
Other comments	None		



4.2.2 Details of existing rating

The most recent rating is considered applicable for data recorded between 1997 and 2003, when the velocity area station was removed. It was one of five ratings derived for the location from when it was installed in 1976. It was considered not to be applicable for flood flows, and there is currently no series of annual maximum flows.

The parameters for the existing rating where $Q = C\,(h - e)^{\beta}$ are given below:

Limb No.	C	e	β	Min stage (m)	Max stage (m)
1	17.500	0.460	2.705	0.46	1.4

4.3 Rating improvements

In order to improve our confidence in the high flows portion of the rating, in particular at flows higher than those that have been reliably gauged, we have developed a new hydraulic model. The following sections of this report describe how the model was developed and how the results were used to derive an improved rating.

4.3.1 Summary of hydraulic modelling

Claremount is within the Owenriff River HPW for the Oughterard AFA. Therefore it is included in the 1D-2D (ISIS-TUFLOW) hydraulic model of Oughterard. The floodplains are represented in the 2D part of the model to enable detailed representation of floodplain flow. No gauge exists at Claremount now and no evidence of the gauge remains, so the surveyors were not able to provide any information on the gauge to confirm details provided on the OPW Hydrodata website, particularly regarding the specific location and datum level at the site.

There is no formal channel bed control near the gauge and no bridges downstream to provide a control at high flows. The river channel is therefore the primary hydraulic control at the former gauge location. The survey in the area of the Claremount gauge is fairly sparse due to the presence of Freshwater Pearl Mussels. Looking at photos and aerial photos it appears there are various natural bedrock formations that act to control water levels at low flows creating pools and riffles. These controls are not picked up in the survey.

Using the surveyed cross sections only, the modelled levels at the Claremount gauge do not match the check gaugings at low flows, being approximately 300mm lower than the levels

recorded. Altering model parameter values in the reach around the gauge does not improve the match.

This suggests that either there has been change in channel geometry since the gauge was removed (seems unlikely given magnitude of difference), the gauge datum is incorrectly recorded on the OPW Hydrodata website (unable to check this) or more likely the hydraulic control is not fully represented. In order to provide a good fit to the check gaugings the model has had to be adjusted locally with additional bed control and reduced channel roughness. This is justified as the survey is very limited in this area due to FWPMs and the presence of un-surveyed bedrock type controls that are particularly important at low flows and critical to matching the gaugings at very low flows.

An in-channel Manning's n value of 0.020 was initially adopted for the reach through Claremount gauge in order to provide a rating curve with appropriate curvature to fit the range of check gaugings. Channel bank roughness is set to 0.06 given the dense vegetation. Floodplain roughness values are set in relation to the land use based on OSi large scale vector mapping, this is typically 0.05 in the vicinity of the gauge site as there is no detailed vector mapping available.

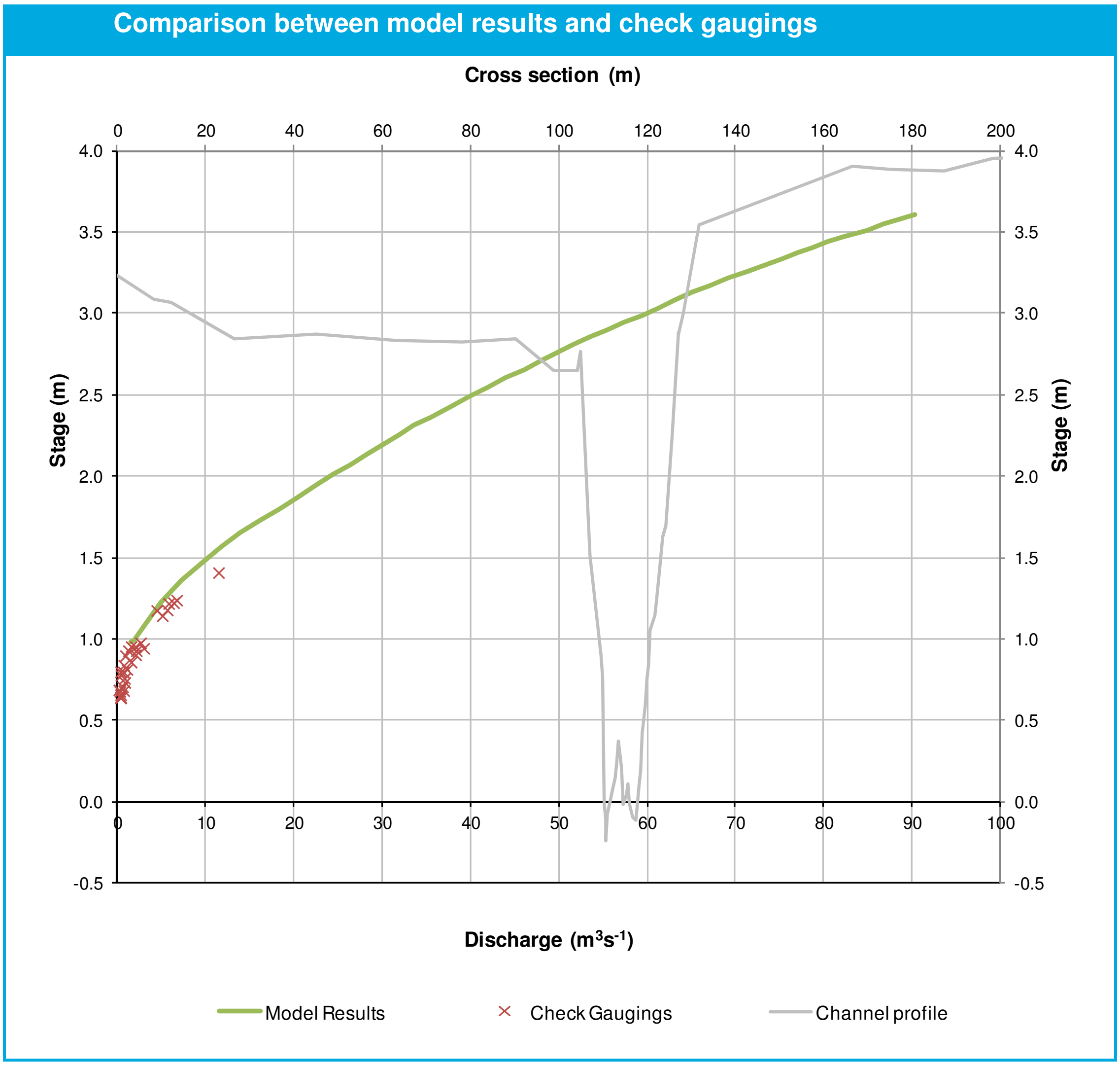
The model run used to derive the rating curve was a single unsteady run using an estimated hydrograph shape starting at 2m^3/s and peaking at 150m^3/s. The peak flow used gives a level well above the highest recorded level at this site. As the gauge is no longer active that is the highest level that needs to be considered.

There is a small period of overlap between the Claremount gauge and the Oughterard DS gauge that replaced it. One AMAX is common between the two on 8 November 2002. Using the modelled ratings for the two gauges gives a flow at Claremount of 34m^3/s and Oughterard DS of 23 m^3/s. The level at the Oughterard DS gauge for this event is within the range of gauged flows whereas at Claremount it is much higher than the gauged flows suggesting that at Oughterard DS there should be greater confidence in the flows for this event.

When comparing the full gauged records, the Claremount gauge shows a steeper gradient than the Oughterard DS gauge with higher flows at higher levels and lower flows and lower levels. The cross over point occurs at approximately 10m^3/s at the Oughterard DS gauge. This highlights an inconsistency between the two gauging stations. (note that the Oughterard DS flows were derived from the hydraulic model rating). Therefore an alternative rating has been developed at Claremount which does not match all the gaugings as well but gives higher flows more consistent with those at Oughterard DS, approx 24m^3/s for the 8 November 2002 event. This model has a channel roughness value of 0.04 and the weir coefficient reduced from 1.7 to 1.4 at the additional bed control near the gauge.

As the gauge no longer exists the purpose of this rating is to refine the recorded AMAX series at this location. The highest recorded AMAX is at 2.95m. On the initial modelled rating fitted to gaugings this corresponds to a flow of 110 m^3/s but on the amended rating curve this level corresponds to a flow of 60m^3/s. The contradiction between the rating based on the check gaugings and that derived to more closely replicate the flows recorded downstream shows the uncertainty at this gauge is high.

The rating at flows between 1.0 to 1.5m stage is not ideal when compared to the check gaugings and further work on this is likely to be fruitless given the gauge is no longer in place. However at all AMAX levels (min AMAX level is 1.81m) the proposed model rating, which has been calibrated towards flows at the downstream gauge, should offer a much improved estimate of flow compared to the original OPW rating. Given the gauge is no longer in place and the new rating is appropriate for all AMAX values we recommend using the new rating through the entire range noting the high uncertainty. Alternatives at mid to lower flows would be difficult to implement and check so long after the gauge has been removed and would likely cause a discontinuity in the rating curve.



4.3.2 Fitting a rating to the modelled results

A stage discharge-rating following a power law form has been parameterised based on the existing rating and the modelled stage-discharge relationship at the measurement section. The rating form applied is $Q=C(h-e)^{\beta}$ where:

h = river stage (m)

Q = river flow (m^3/s)

C, e, β are constants:

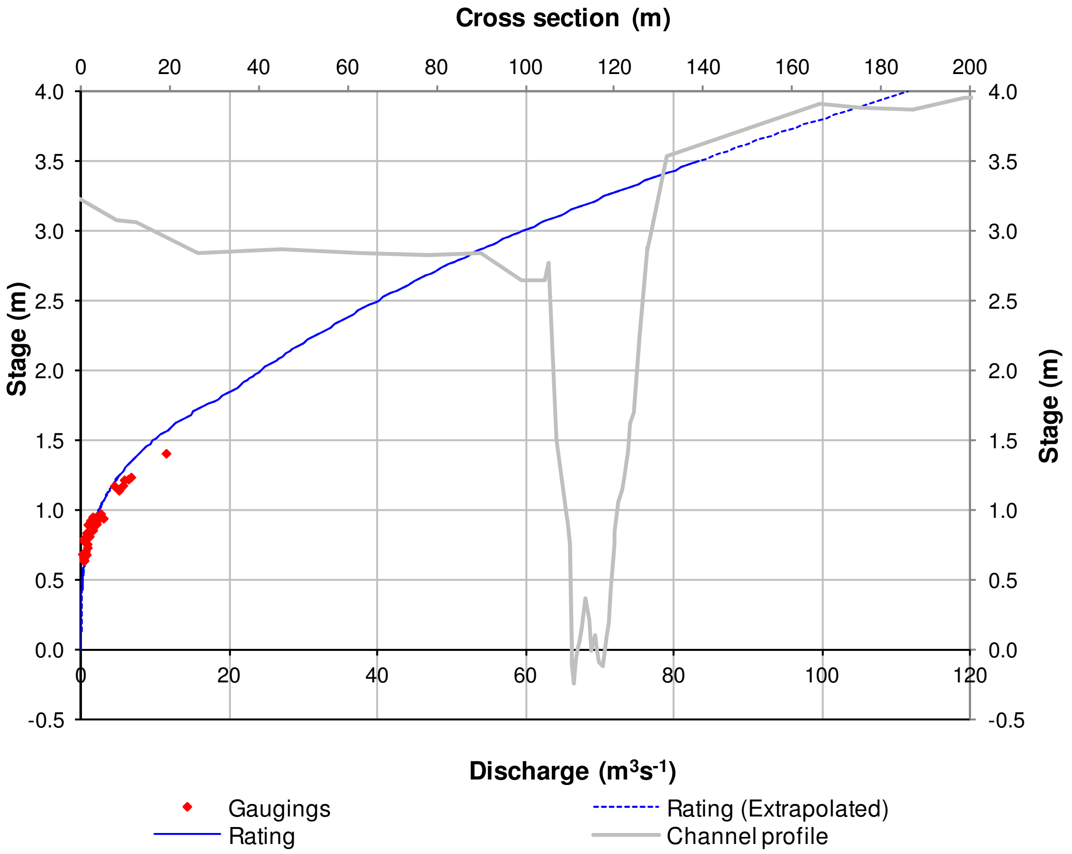
- the coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases.
- the coefficient β is related to the geometry of the channel and
- the coefficient e is related to the elevation of the bed relative to the gauge datum.

In fitting a power law to the modelled ratings, limb or segment breaks have been based on physical interpretation of hydraulic mechanisms and channel geometry, but only where supported statistically (evaluated based on the root mean square error). Fitting has been carried out using bespoke in-house rating curve fitting and evaluation software known as JRacuda. The proposed rating parameters can be seen in the table below with the rating's respective stage discharge pairs available in Section 4.4. The proposed rating form is visible in the graph below.

Limb	C	e	β	SG (min)	SG (max)
1	2.427	0.000	3.460	0.5	1.85
2	5.164	0.000	2.235	1.850	2.550
3	5.423	0.000	2.182	2.550	3.500

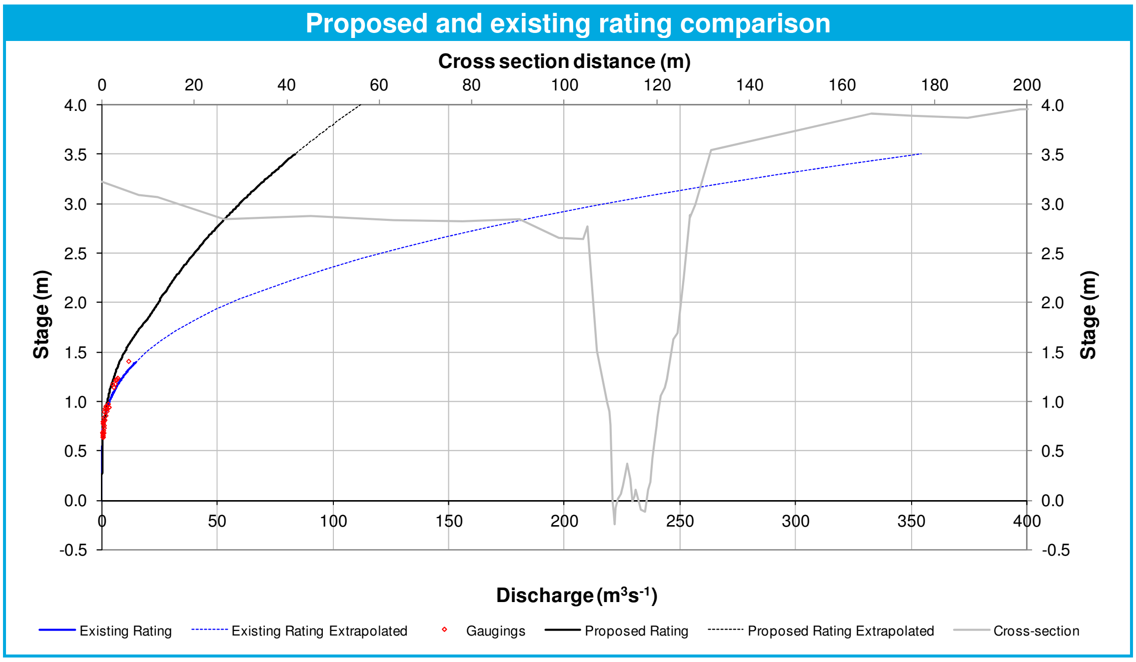
The proposed rating consists of three limbs to describe the hydraulic relationship at Claremount up to a water depth of 3.5m (well above the highest recorded level). All three model limbs have been fitted using results from the hydraulic model. The figure below plots the proposed rating against the available check gaugings.

Proposed rating with suitable check gaugings



4.3.3 Comparison with existing rating

The graph below compares the existing with the newly proposed rating. The stage discharge relationship is comparable below 0.8m beyond which the rating curves diverge. Above approximately 1.5m the ratings diverge significantly with the old extrapolated rating plateauing much more dramatically. Both ratings are illustrated on the graph below.



4.3.4 Overall agreement with check gaugings

The proposed rating fits very well to the available flow gaugings below a stage of approximately 1m. However above this level it was necessary for the rating to diverge slightly from the gauged flows in order to remain consistent with the record of flow downstream at Oughterard. Moreover, as the gauge is no longer in situ (to facilitate accurate survey) and the some of the hydraulic controls have had to be estimated there remains a high degree of uncertainty with the record of flow derived at this gauge.

4.3.5 Range of applicability

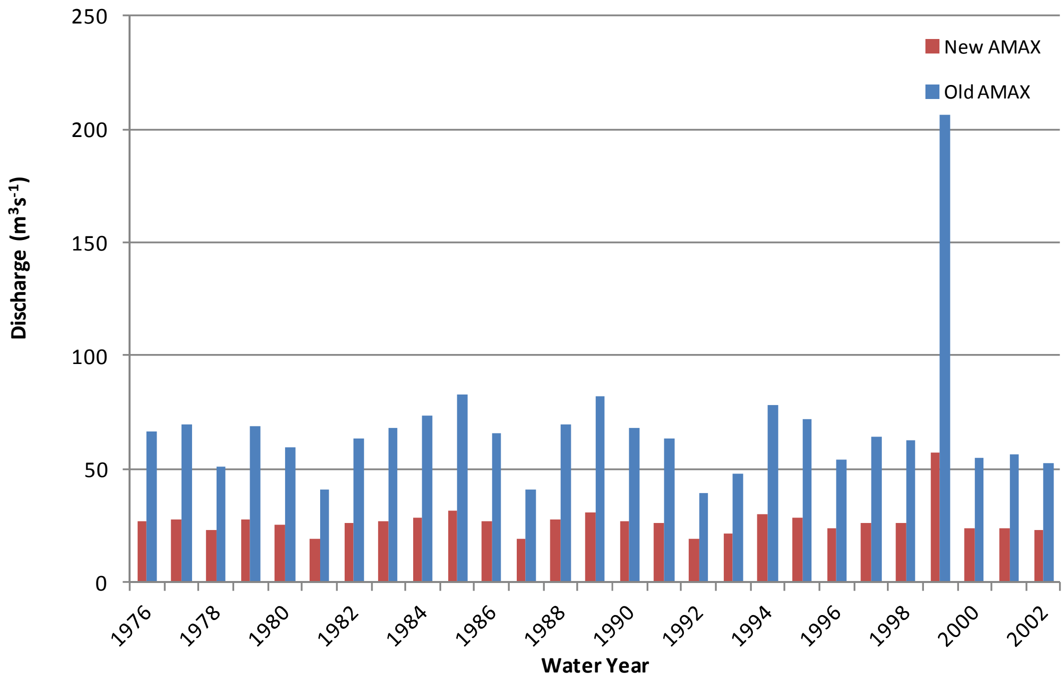
The upper limit of the rating is specified at 3.5m; this refers to the highest flows modelled but is higher than the highest level recorded at the gauge. As the gauge is no longer in use there is no merit to extrapolating the rating any further. The rating does deviate from check gaugings above 1.0m in order to calibrate with the downstream gauge. The low flow rating at this site contains greater uncertainty than would be ideal.

4.4 Proposed rating stage discharge pairs

Stage (m)	Flow (m^3/s)
3.5	83.457
3.4	78.341
3.3	73.400
3.2	68.633
3.1	64.039
3.0	59.617
2.9	55.366
2.8	51.285
2.7	47.372
2.6	43.627
2.5	40.039
2.4	36.547
2.3	33.231
2.2	30.087
2.1	27.116
2.0	24.314
1.9	21.680
1.8	18.550
1.7	15.221
1.6	12.341
1.5	9.871
1.4	7.775
1.3	6.016
1.2	4.561
1.1	3.375
1.0	2.427
0.9	1.686
0.8	1.121
0.7	0.707
0.6	0.414
0.5	0.221

4.5 Impact on QMED and annual maximum series

The revised rating has a significant impact on the derived flow series with all values in the AMAX series showing a dramatic reduction. At the median annual maximum level (2.08m), the existing rating estimates a flow of 65m^3/s compared to 27m^3/s derived using the new rating. This represents a reduction of 59%. At higher flows, the difference is even more pronounced with the highest flow on record being reduced from 206m^3/s to 57 m^3/s (a reduction of 72%). The AMAX series derived using both the old and revised ratings are compared on the graph below.



5 Ballyhaunis

5.1 Station description - from Inception Report

5.1.1 Gauge summary

Station name	Ballyhaunis	**Site type**	Velocity-area
Station number	30020	**Watercourse**	River Dalgan
Grid reference	149621, 279430	**Operator**	Mayo County Council

5.1.2 Location

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5.1.3 Gauge Datum

Gauge datum (mAOD)	74.075 (Poolbeg) or 71.36m (Malin Head)
Means of confirmation (e.g. survey)	Provided in AMAX data. The 0m level on the gauge board was surveyed as 71.44m OD. This is a discrepancy of 0.08m. A review of the modelled output compared to the gaugings shows closer agreement using the original datum than the surveyed datum. The original datum has been used for the analysis.
Other comments	Gauge board located on the gauging station housing structure.

5.1.4 Description/ other comments

The gauging station is located on the right bank, 30m downstream of the road bridge in Ballyhaunis.

5.1.5 Control on stage discharge relationship

Type of section	Upstream face of bridge.
Low flow control(s)	Control weir downstream of the gauge will provide a reliable hydraulic control at low flows. This weir was installed in 1987. 
High flow control(s)	At high flows the hydraulic controls at the gauge location will be more complex. These will include possible backwater effects from downstream structures and channel geometry (once the weir becomes non modular), out of bank geometry, hydraulic resistance and a possible influence from the upstream bridge exacerbating bypassing (see below). 
Bed slope	Channel gradient at the gauge location has been estimated from 1:50,000 mapping to be approximately 0.002m/m

Roughness	Artificial concrete channel - Very smooth channel bed
	

5.1.6 Bypass routes

There will be little by passing of the gauge at low to moderate flows. However once the banks are overtopped at the gauge location the control structure will be bypassed. At even higher flows it is possible that backing up behind the upstream bridge may result in some flow running down the road and bypassing the gauge location altogether.

Looking at right bank of river downstream (picture taken from road bridge)	
	
Right bank of river downstream (picture taken from road bridge)	
	
From the left bank, looking towards the upstream face bridge.	From the left bank, looking towards the upstream face bridge.
	

From the left bank, looking upstream (taken 20m downstream of road bridge)	From the left bank, looking downstream. (taken 200m downstream of road bridge)
	

5.1.7 Additional photographs

Looking at stilling well, gauge board and small upstream weir	
	
River channel ~100m downstream of gauge	River channel ~150m downstream of gauge
	

River channel ~200m downstream of gauge	River channel ~210m downstream of gauge
	

5.2 Rating details

5.2.1 Check gaugings summary

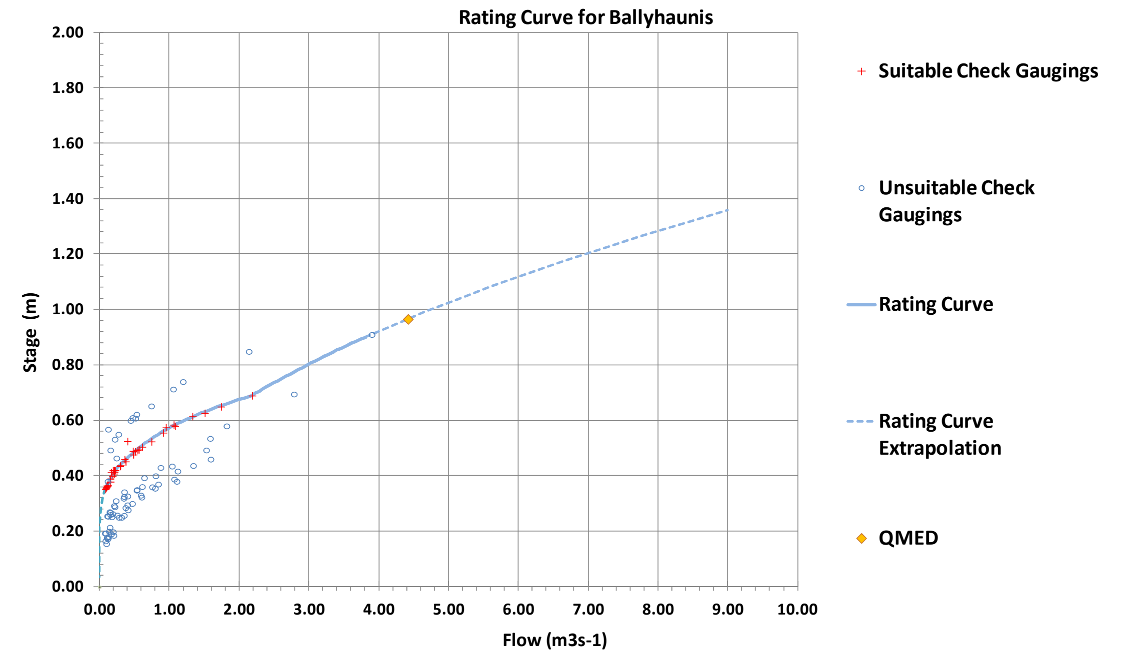
No. of gaugings	111 (39 since 1987)	Date range	1975 - 2011
Maximum gauged stage (m)	0.69		
Approximate stage corresponding to QMED (m)	0.95	Extrapolation of rating to QMED (m)	0.26
Maximum observed stage (m)	1.24	Extrapolation to highest flow (m)	0.55
Other comments	There are only two suitable check gaugings falling within the range of the upper limb of the current rating.		

5.2.2 Details of existing rating

The current rating (supplied by Mayo County Council) is a compound rating comprising four limbs. The rating is considered suitable for all data recorded since the existing control weir was installed in 1987.

The parameters for the existing rating where $Q = C (h - e)^{\beta}$ are given below:

Limb No.	C	e	β	Min stage (m)	Max stage (m)
1	98.9212	0	6.74913	0.352	0.369
2	23.5435	0	5.30887	0.369	0.481
3	10.4604	0	4.20168	0.481	0.688
4	4.75044	0	2.08889	0.688	0.910



Check gaugings undertaken prior to the control weir being installed are displayed above as unsuitable gaugings and have not been included in any analysis.

5.2.3 Evaluation of existing rating

Overall agreement with check gaugings	Generally, there is a very good agreement between the current rating and the check gaugings which have been undertaken since the control weir was installed in 1987. However, on the upper limb of the rating there are only two suitable gaugings and the rating has obviously been defined to intersect them. The result is no scatter around the upper portion of the rating and so it is very hard to define the associated uncertainty. For this reason confidence intervals have not been provided on the figure above.
Range of applicability	The current rating should only be applied to data collected after the existing control weir was installed in 1987. It is also only currently recommended to be used for levels lower than 0.91m.
Stability of rating	Following the installation of the control weir the rating appears very stable.
Uncertainty	As discussed above it is not possible to estimate uncertainty at high flows without additional data. This data could either be in the form of additional high flow gaugings or a separate rating derived independently using a hydraulic model.

5.2.4 Recommendations for rating improvement

Additional high flow gaugings and hydraulic modelling would help improve the high flows rating at this station. For the purposes of improving the rating at this location a 1D hydraulic model is probably sufficient. This is due to the constrained river channel and little possibility for extensive bypassing. The model should extend as far downstream as the railway bridge and should include all significant hydraulic structures. This section of watercourse forms part of an HPW and will also be being modelled for this purpose.

5.3 Rating improvements

In order to improve our confidence in the high flows portion of the rating, in particular at flows higher than those that have been reliably gauged, we have developed a new hydraulic model. The following sections of this report describe how this model was developed and how the model results were used to derive an improved rating.

5.3.1 Choice of modelling method

The Ballyhaunis Gauge is located on the Dalgan River within a narrow concrete lined channel. Out of bank the local topography rises steeply on both banks confining flow to the channel and the local floodplain. There are two small weirs located upstream and downstream of the gauge, the latter of which is the control for the gauge at low flows.

Downstream of the gauge the local topography flattens out and higher flows will overtop the banks on both sides of the watercourse. It is this channel and floodplain that will become the control as the small local weir is drowned out.

To accurately model the floodplain downstream of the gauge site and to provide consistency with the final AFA model the preferred approach for the rating review is to model this gauge using a 1D 2D hydraulic model.

5.3.2 Summary of hydraulic modelling

Overview of model and location

The Ballyhaunis gauge is located within the Ballyhaunis AFA and as such will be included in the 1D 2D hydraulic model of the AFA. The rating review model extends the full length of the AFA from approximately 1km upstream of the gauge site to where the Dalgan passes beneath the railway line downstream of the AFA.

A single tributary, the Curries watercourse, discharges into the Dalgan within the AFA upstream of the gauge site. This has been included in the model but will not have any impact on the rating.

The rating review model is a fully linked 1D 2D model of the system described above.

Representation of channel controls

The main control for the gauge is a small weir located immediately downstream of the gauge. This has been represented as parallel weirs for in bank and out of bank flow to appropriately represent the change in hydraulic roughness between these two areas of the active channel.

Further downstream is a 35m long culvert that runs parallel to the railway line. This starts to have an influence at higher flows causing areas of the floodplain between the culvert and the gauge site to become active. The structure has been modelled as a rectangular culvert.

Comparison and use of gauge datum

The datum provided with the gaugings is 71.365 OD. The 0m level on the gauge board as surveyed was 71.44m OD. This is a discrepancy of 0.08m. A review of the modelled output compared to the gaugings shows closer agreement using the original datum than the surveyed datum. The original datum has been used for the analysis.

Roughness values used

A channel Manning's n value of 0.020 has been applied for the channel bed of the Dalgan River in the vicinity of the gauge site where the channel is concrete lined. The left bank and right banks are covered in bushes and trees through this reach and Manning's n has been set as 0.07 in these areas.

Floodplain roughness values are set in relation to the land use based on OSi large scale vector mapping.

Model Run

The model run used to derive the rating curve was a single unsteady run using an estimated hydrograph shape starting at 0.05m^3/s and peaking at 15m^3/s. The maximum AMAX flow at

the gauge from the existing rating is 7.5m^3/s. There is little attenuation between the top of the model and the gauge site and the peak flow at the gauge site is also 15m^3/s.

Further Discussion

Preliminary model runs were completed with out of bank flows at the gauge represented in 2D. This was found to produce a negative water surface at the gauge where flow was bypassing the gauge cross section in the 1D model via the 2D model. To prevent this, the channel and floodplain in the vicinity of the gauge site have been modelled in 1D only with the representation of the floodplain in 2D starting downstream. Flows through this reach are constrained within a narrow reach and representation in 1D is appropriate.

Conclusions

The hydraulic model provides a reasonable fit to the gaugings. It does not fully mirror the observed gaugings at low flows despite the location of a control weir immediately downstream.

The modelled rating does tie into the single high flow gauging and there is little reason to doubt the extrapolation of the rating curve beyond the observed data.

5.3.3 Fitting a rating to the modelled results

A stage discharge-rating following a power law form has been parameterised based on the existing rating and the modelled stage-discharge relationship at the measurement section. The rating form applied is $Q=C(h-e)^{\beta}$ where:

h = river stage (m)

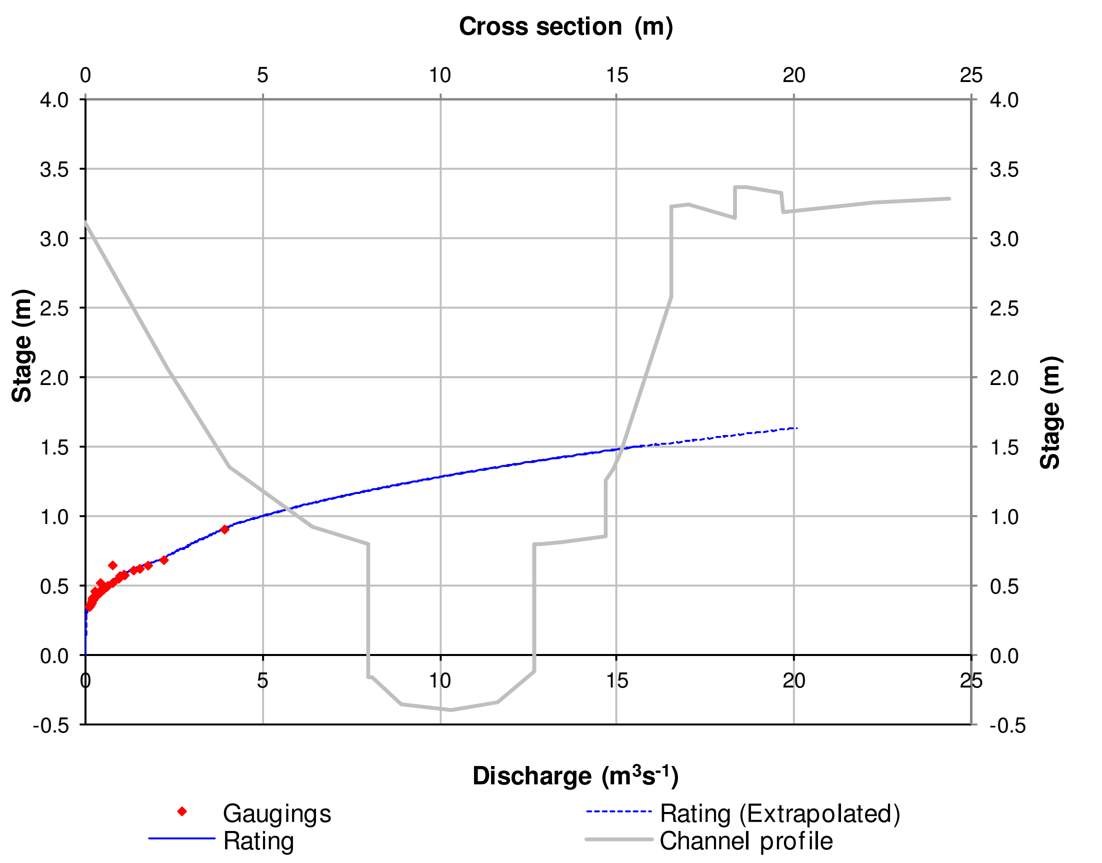
Q = river flow (m^3/s)

C, e, β are constants:

- the coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases.
- the coefficient β is related to the geometry of the channel and
- the coefficient e is related to the elevation of the bed relative to the gauge datum.

In fitting a power law to the modelled ratings, limb or segment breaks have been based on physical interpretation of hydraulic mechanisms and channel geometry, but only where supported statistically (evaluated based on the root mean square error). Fitting has been carried out using bespoke in-house rating curve fitting and evaluation software known as JRacuda. This has only been carried out for the rating above a stage of 0.95m with the existing rating used for levels below this. The proposed rating parameters can be seen in the table below with the rating's respective stage discharge pairs available in Section 1.4. The proposed rating form is visible in the graph below.

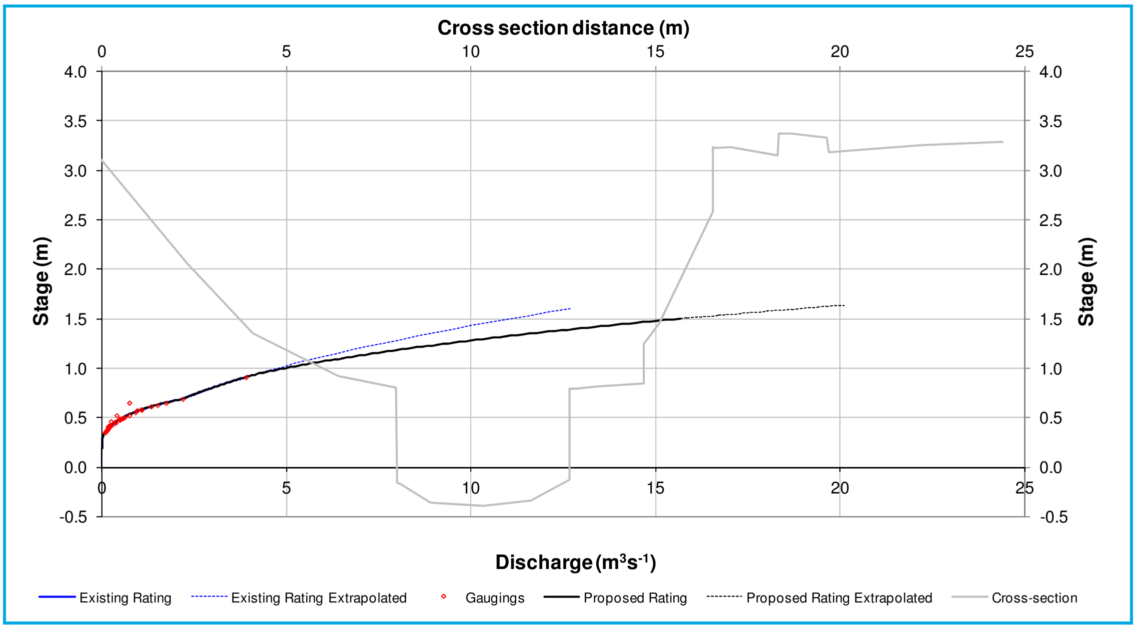
Limb	C	e	β	SG (min)	SG (max)
1	98.921	0.000	6.749	0.000	0.37
2	23.544	0.000	5.309	0.370	0.48
3	10.460	0.000	4.202	0.480	0.69
4	4.750	0.000	2.089	0.690	0.950
5	4.962	0.000	2.835	0.950	1.500



The proposed rating consists of five limbs to describe the hydraulic relationship at Ballyhaunis to a water depth of 1.5m. It utilises the existing rating in its entirety before merging with the model derived rating above 0.91m to describe higher flow conditions (see graph below).

5.3.4 Comparison with existing rating

The graph below compares the existing with the newly proposed rating. The stage discharge relationship is identical below water depths of 0.91m (the median annual flood water level is estimated at 0.95m) beyond which the proposed rating and extrapolation of the existing rating diverge. Extrapolation of the existing rating beyond 0.91m estimates a much lower flow for a given stage; a 19% reduction for the largest annual maximum flood. The modelled rating has been preferred in this instance as it incorporates the changes to the channel geometry with increasing water levels.



5.3.5 Overall agreement with check gaugings

As the proposed rating adopts the existing theoretical rating parameters, there is an excellent fit to the available gaugings. Beyond the maximum gauged flow of $3.9m^3/s$ there is no information with which to calibrate and subsequently validate the rating during these flow conditions. However, the hydraulic model is thought to provide a reasonable estimate in the absence of any additional information.

5.3.6 Range of applicability

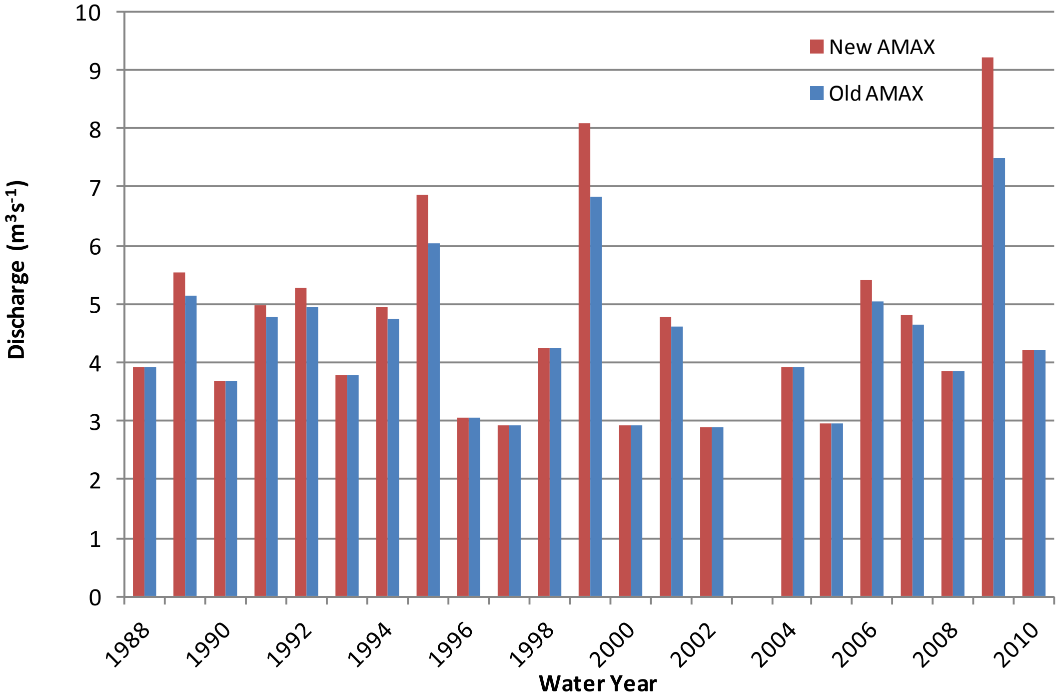
The upper limit of the rating is specified at 1.5m, the peak stage value attained during the hydrodynamic simulation. For stage values beyond this there is less certainty on what form and gradient the rating will assume although it is likely that the upper limb will continue as the non modular flow is already accounted for and flows will continue to be contained.

5.4 Proposed rating stage discharge pairs

Stage (m)	Flow (m^3/s)
1.5	15.663
1.4	12.880
1.3	10.440
1.2	8.320
1.1	6.501
1.0	4.962
0.9	3.812
0.8	2.980
0.7	2.255
0.6	1.223
0.5	0.568
0.4	0.182
0.3	0.029
0.2	0.002
0.1	0.000
0.0	0.000

5.5 Impact on QMED and annual maximum series

The proposed rating increases the estimates of high flows beyond 0.91m only. As a result, there is no change in the estimate of the median annual maximum flood (4.23 m^3/s) using suitable data from 1987, the period for which the existing rating is currently valid. For the largest annual maximum flood in the 2009 water year, the flow estimate increases by approximately 22% from 7.5 m^3/s to 9.2 m^3/s. The graph below illustrates what influence changing the rating has on the annual maximum series when applied across the period of record.



6 Oughterard D/S

6.1 Station description

6.1.1 Gauge summary

Station name	Oughterard D/S	**Site type**	Velocity-area
Station number	30101	**Watercourse**	Owenriff River
Grid reference	112237 243154	**Operator**	OPW

6.1.2 Location

The gauge is located on the left bank of the river immediately upstream of the road bridge.

EN 0021013 © Ordnance Survey Ireland / Government of Ireland

6.1.3 Gauge Datum

Gauge datum (mAOD)	9.24
Means of confirmation (e.g. survey)	Supplied by OPW
Other comments (e.g. gauge boards)	Gauge board located on the upstream face of the bridge.

6.1.4 Description/ other comments

The gauge is located on the left bank of the river just upstream of the road bridge.

6.1.5 Control on stage discharge relationship

<table>
<tr><td>Type of section</td><td>Upstream face of bridge.</td></tr>
<tr><td rowspan="2">Low flow control(s)</td><td>At low flows the dominant hydraulic control will be the channel geometry and bridge abutments.</td></tr>
<tr><td></td></tr>
<tr><td rowspan="2">High flow control(s)</td><td>At higher flows the control on water level will be the bridge. As water levels rise the bridge will exert a greater influence on the water levels as the opening is surcharged.</td></tr>
<tr><td></td></tr>
</table>

6.1.6 Bypass routes

The gauge will not be bypassed until the road levels are overtopped and these are elevated well above the floodplain level. However, the relatively low bridge soffit is likely to cause a restriction to flow during flood events and will probably result in considerable head loss.

No historical information was available regarding bypassing of this bridge. However, our modelled calibration runs indicate that the 1999 event bypassed the bridge on left bank flowing over the road (not over the bridge deck) but 2011 event did not (the model indicates that bypassing will start at approximately 58m^3/s. It is likely that only events greater than the 1% AEP will result in significant bypassing.

From the bridge, looking towards the left bank floodplain.	From the bridge looking towards the right bank floodplain
	

6.1.7 Additional photographs

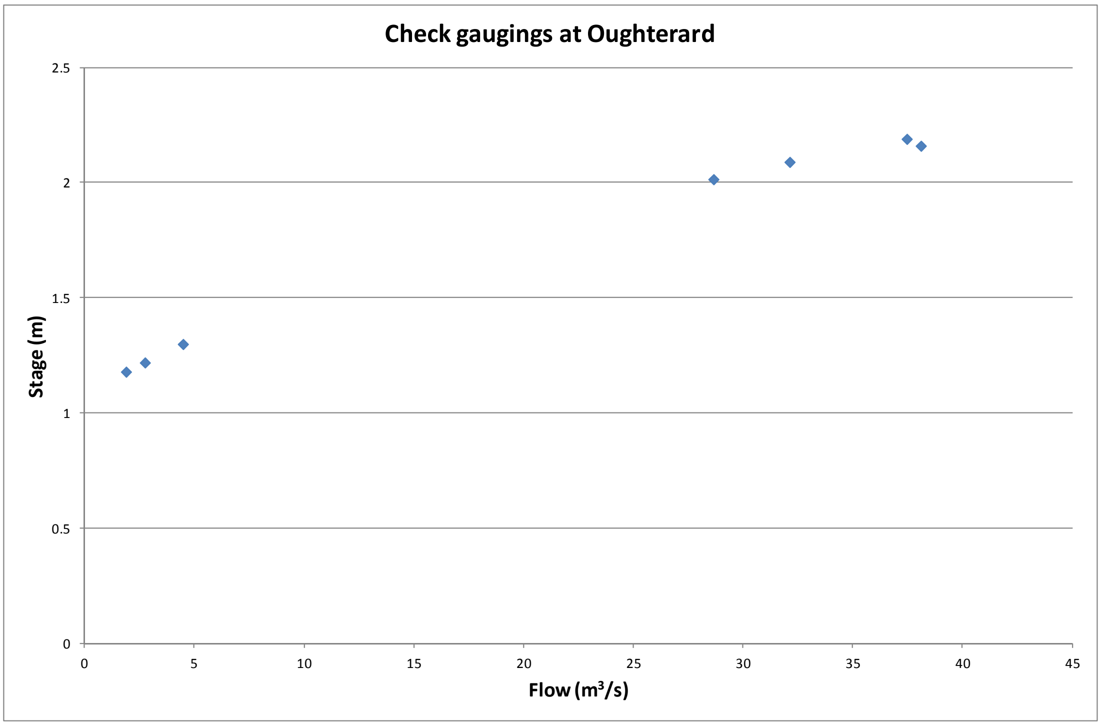
Looking downstream from road bridge	Looking upstream from road bridge
	
Looking upstream on left bank	View onto left bank away from channel - road behind wall at top right
	
View across channel from left to right bank upstream of gauge	View downstream towards gauge at bridge
	

View from road bridge along road on left bank


6.2 Rating details

6.2.1 Check gaugings summary

No. of gaugings	7	Date range	2001 - 2011
Maximum gauged stage (m)	2.19		
Approximate stage corresponding to QMED (m)	1.91	Extrapolation of rating to QMED (m)	n/a
Maximum observed stage (m)	2.2	Extrapolation to highest flow (m)	n/a
Other comments	No rating has been developed at this site.		



6.2.2 Details of existing rating

No rating was supplied for this site.

6.3 Rating improvements

In order to derive a rating suitable for the estimation of peak flow from recorded AMAX values of stage, we have developed a new hydraulic model. The following sections of this report describe how this model was developed and how the model results were used to derive an improved rating.

6.3.1 Summary of hydraulic modelling

Oughterard DS is within the Owenriff River HPW for the Oughterard AFA. As such it is included in the 1D-2D (ISIS-TUFLOW) hydraulic model of Oughterard. The floodplains are represented in the 2D part of the model to enable detailed representation of floodplain flow.

There is no formal channel bed control near the gauge but a bridge is situated immediately at the gauge location (gauge is located on the upstream face). At low flows the channel and bridge abutments will be the hydraulic control and these appear to be adequately represented in the survey data. At higher flows the bridge will become a greater influence on water levels. Gaugings extend to nearly 40m^3/s which is greater than QMED and are accurately represented by the model.

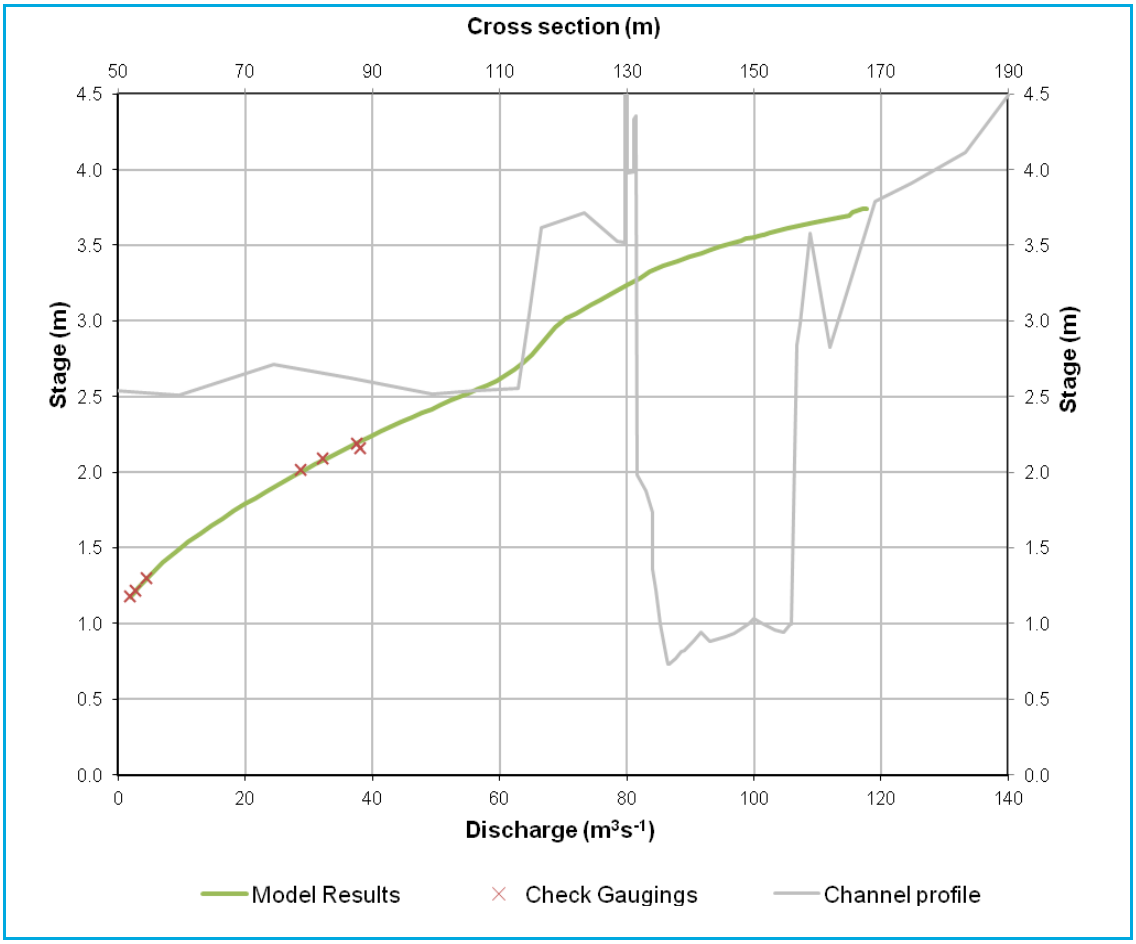
A channel Manning's n value of 0.035 has been adopted for the reach through Oughterard DS gauge in order to provide a rating curve with appropriate curvature to fit the range of check gaugings. Channel bank roughness is set to 0.055 given the moderate vegetation cover. Floodplain roughness values are set in relation to the land use based on OSi large scale vector mapping.

The model run used to derive the rating curve was a single unsteady run using an estimated hydrograph shape starting at 2m^3/s and peaking at 120m^3/s. The peak flow used gives a level approximately 1.5m above the highest recorded level at this site.

There is a small period of overlap between the Claremount gauge and the Oughterard DS gauge that replaced it. One AMAX is common between the two on 8 November 2002. Using the modelled ratings for the two gauges gives a flow at Claremount of 23.37m^3/s and Oughterard DS of 23 m^3/s. The level at the Oughterard DS gauge for this event is within the range of gauged flows whereas at Claremount it is higher than the gauged flows suggesting that at Oughterard DS there should be greater confidence in the flows for this event.

The only other water year with data at both gauges is 2001 but the AMAX level did not occur on the same date for the two gauges. This was because two events of very similar magnitude occurred in the same water year; both events record a very similar stage at both gauges but their relative magnitudes differ between the two events meaning that different events are recorded as the AMAX value. Using the modelled ratings the earlier (3rd December) event reached 23.96m^3/s at Claremount and 22.98m^3/s at Oughterard. The latter event (10th March) is estimated to be 23.88m^3/s at Claremount and 22.66m^3/s at Oughterard. In both cases, the flows are comparable at the two gauges but slightly lower at Oughterard.

It is clear from this rating curve graphs that the model is able to do a good job of replicating the hydraulic conditions at the gauge up to high flows (gauged to nearly 40m^3/s which is greater than QMED).



6.3.2 Fitting a rating to the modelled results

A stage discharge-rating following a power law form has been parameterised based on the existing rating and the modelled stage-discharge relationship at the measurement section. The rating form applied is $Q=C(h-e)^{\beta}$ where:

h = river stage (m)

Q = river flow (m^3/s)

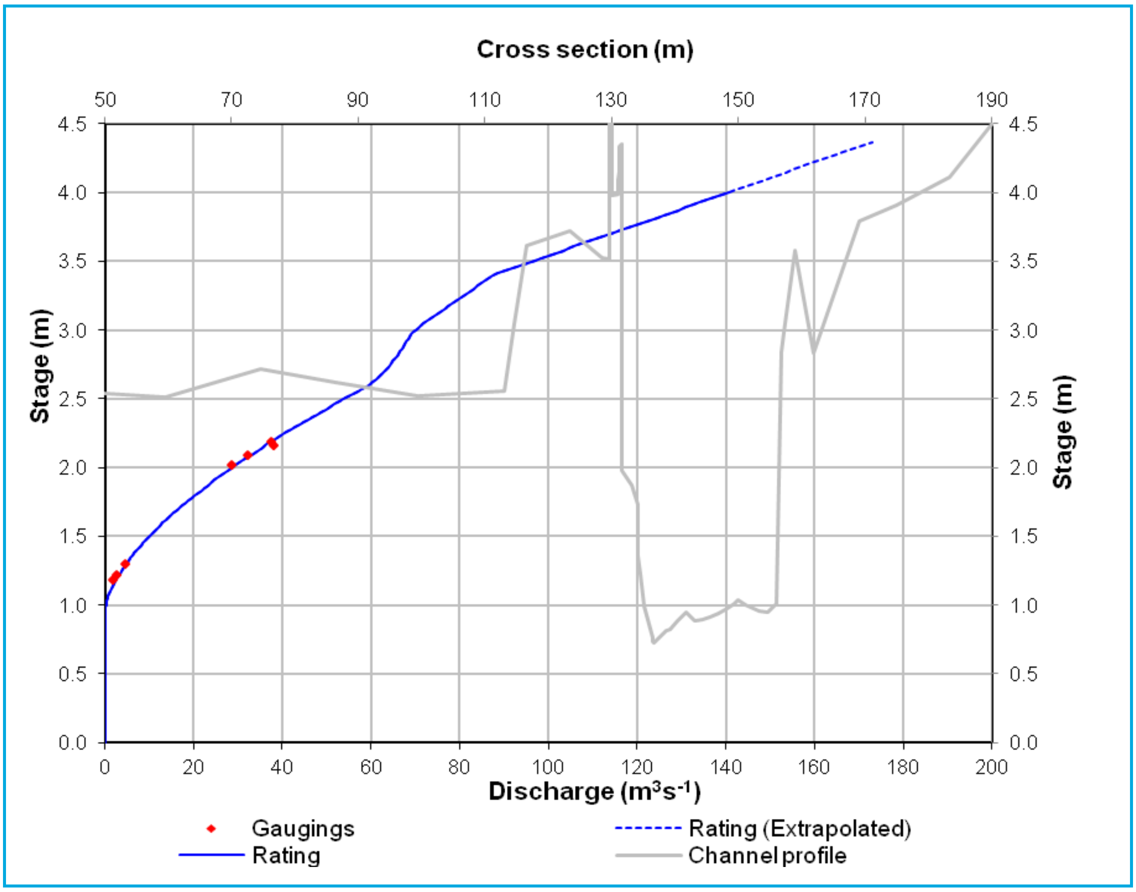
C, e, β are constants:

- The coefficient C increases as river cross-sectional area and slope increase, but decreases as roughness increases.
- The coefficient β is related to the geometry of the channel and
- The coefficient e is related to the elevation of the bed relative to the gauge datum.

In fitting a power law to the modelled ratings, limb or segment breaks have been based on physical interpretation of hydraulic mechanisms and channel geometry, but only where supported statistically (evaluated based on the root mean square error). Fitting has been carried out using bespoke in-house rating curve fitting and evaluation software known as JRacuda. The proposed rating parameters can be seen in the table below with the rating's respective stage discharge pairs available in Section 1.4. The proposed rating form is visible in the graph below.

Limb	C	e	β	SG (min)	SG (max)
1	26.457	0.948	1.624	0.000	2.570
2	74.548	2.421	0.131	2.570	2.980
3	9.601	0.000	1.808	2.980	3.410
4	89.700	2.425	0.989	3.410	4.000

The proposed rating consists of four limbs to describe the hydraulic relationship at Oughterard Gauge up to a water depth of 4m. As no current rating exists for the station, the proposed rating was developed entirely from the calibrated hydraulic model results (see graph below). The concave curvature associated with limb two can be attributed to the surcharging of the arch bridge immediately downstream of the gauging station. As increasing bypassing occurs across the floodplain, the rating curve steadily plateaus as demonstrated in limbs three and four. It is noted that limb 2 is effectively a connecting limb allowing the rating to transition between two hydraulic controls, as a result the limb is fitted mathematically and the resulting parameters are typical (for example a very low value of β). As such the parameters for this limb should not be viewed as having a direct physical meaning.



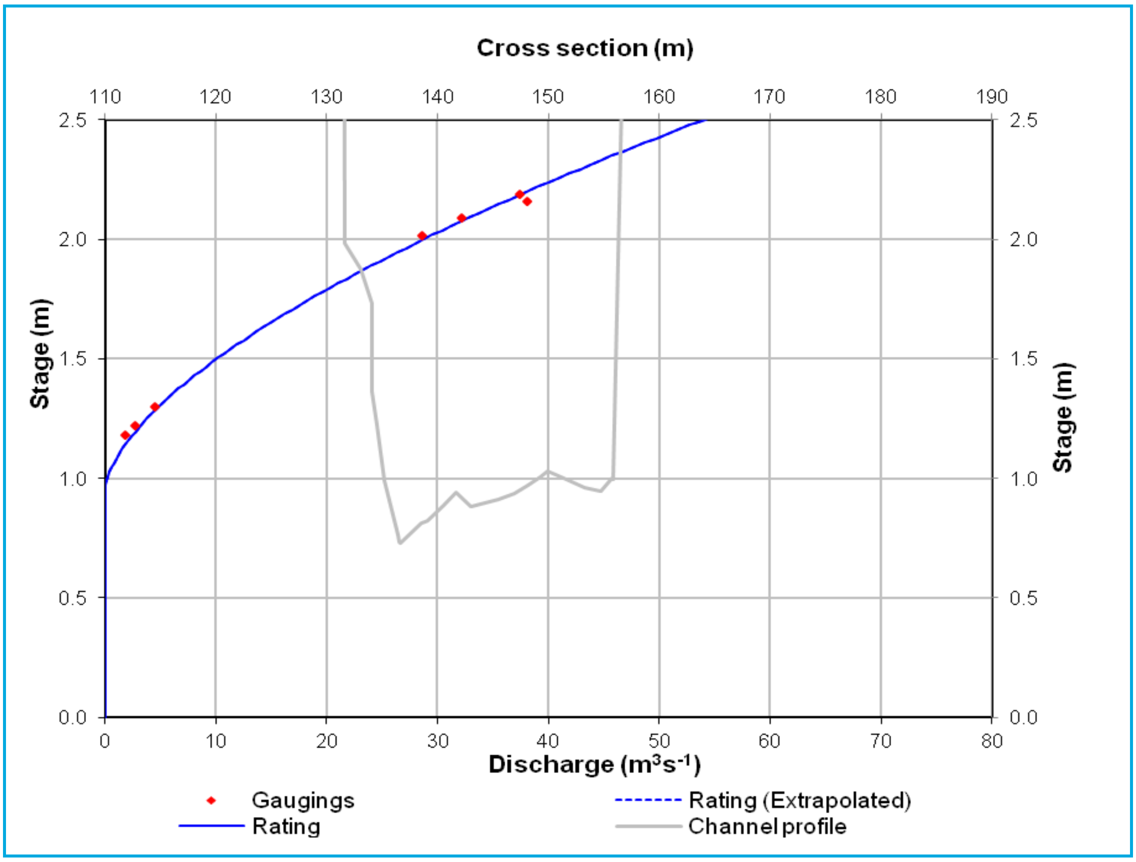
6.3.3 Comparison with existing rating

No rating is currently available for Oughterard.

6.3.4 Overall agreement with check gaugings

The proposed rating fits very well to the available flow gaugings (see graph below). It should be noted that these are all well within bank and there are no check gaugings available beyond 2.22m with which to calibrate the model or provide any validation during high flow conditions.

However, the hydraulic model is thought to provide a reasonable estimate in the absence of any additional information.



6.3.5 Range of applicability

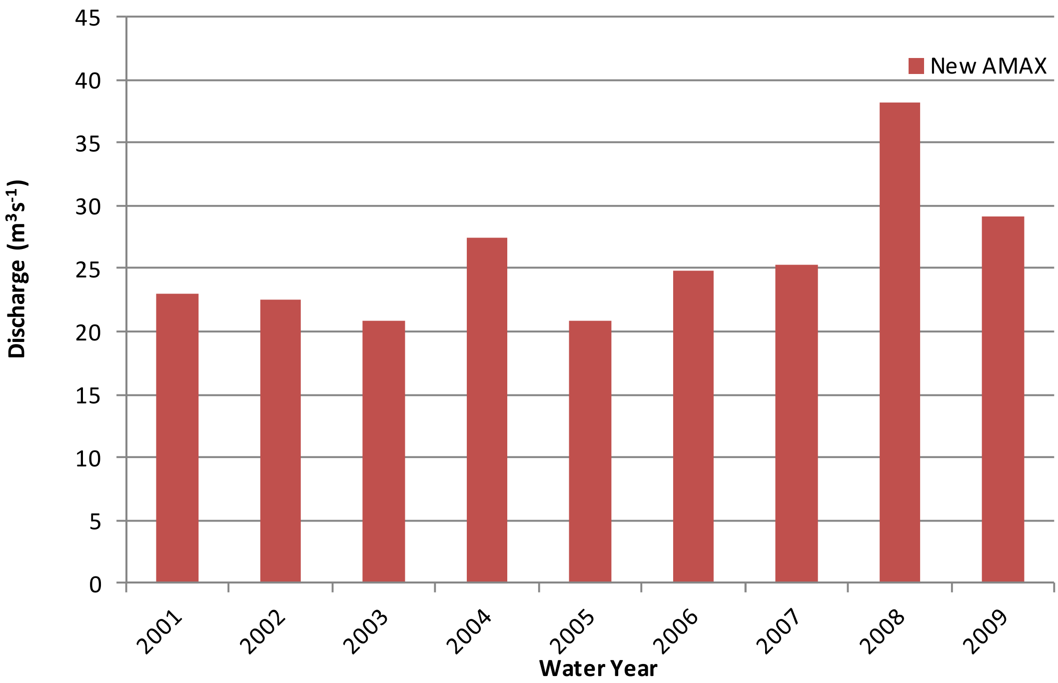
The upper limit of the rating is specified at 4m, the peak stage value attained during the hydrodynamic simulation. For stage values beyond this there is less certainty on what form and gradient the rating will assume although it is likely that the upper limb will continue as the floodplain is already inundated by this point and the floodplain appears relatively well contained.

6.4 Proposed rating stage discharge pairs

Stage (m)	Flow (m^3/s)
4.0	140.573
3.9	131.743
3.8	122.906
3.7	114.062
3.6	105.211
3.5	96.351
3.4	87.747
3.3	83.136
3.2	78.637
3.1	74.250
3.0	69.976
2.9	67.696
2.8	65.650
2.7	63.068
2.6	59.506
2.5	54.019
2.4	48.481
2.3	43.176
2.2	38.111
2.1	33.292
2.0	28.727
1.9	24.426
1.8	20.397
1.7	16.654
1.6	13.209
1.5	10.080
1.4	7.286
1.3	4.854
1.2	2.821
1.1	1.241
1.0	0.217

6.5 Impact on QMED and annual maximum series

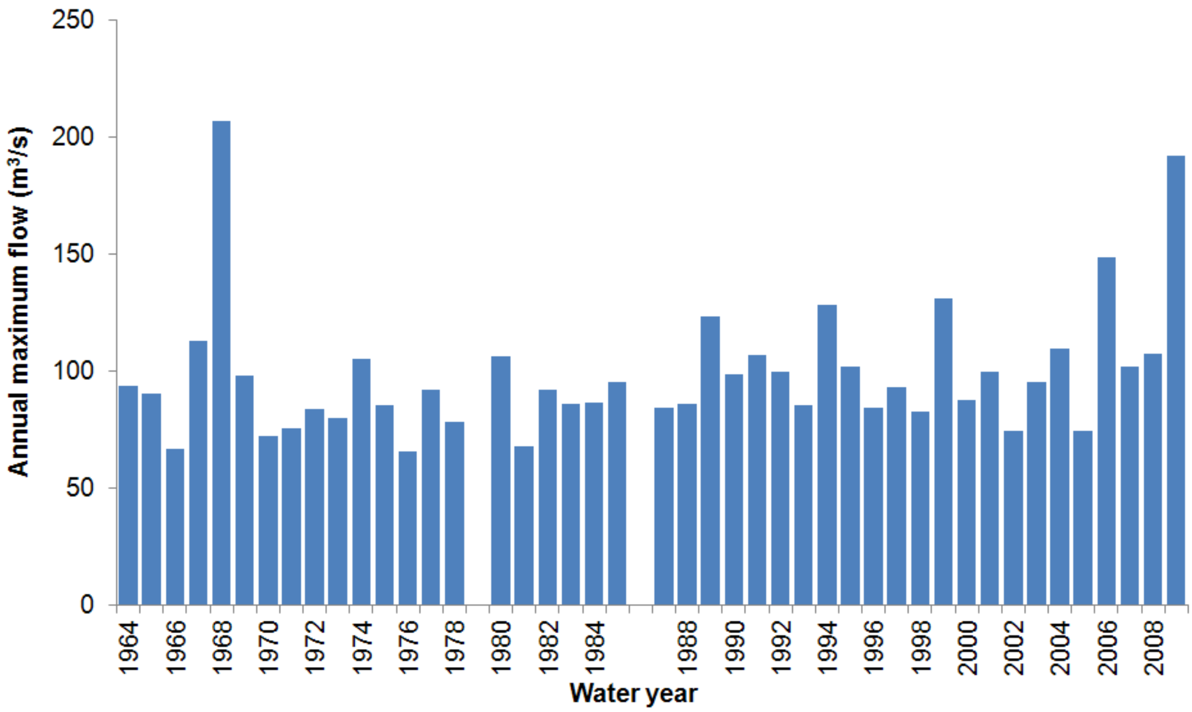
Comparison could not be drawn with existing annual maximum flows as no formal rating currently exists. The proposed rating estimates the mean annual flood at 25 m^3/s, **with the largest annual maximum flow estimated at** 38 m^3/s **for the 1968 event.** The graph below illustrates the annual maximum series as a result of applying the proposed rating to the period of record.



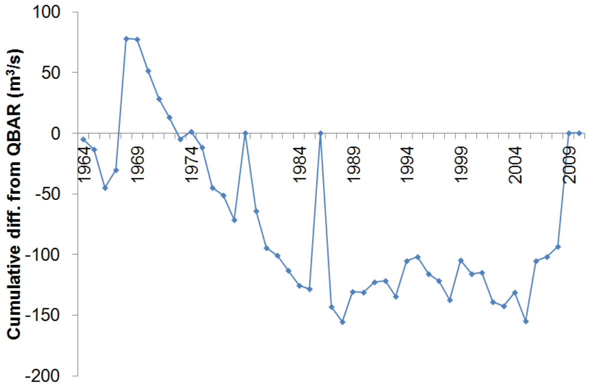
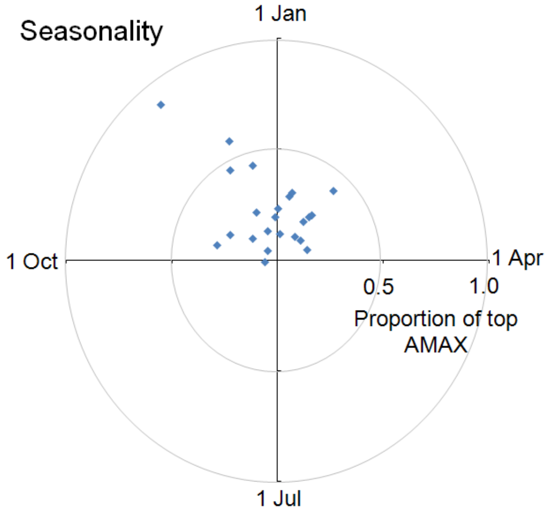
B Flood peak analysis

Flood frequency analysis summary sheet

Station 30004	Clare @ Corrofin
Analysis of original flood peak series, from inception report	
Note: after the inception phase, the decision was taken to exclude pre-1972 flow data (for reasons given below) and so the findings of the inception report have been superseded. They are included below for consistency with the information presented at other gauges.	



Top ranking floods:			QMED (m^3/s): 92.7
Rank	**Date**	**Flow (m^3/s)**	**AEP (%) from single-site analysis**
1	02 November 1968	206.7	0.2
2	21 November 2009	192.0	0.5
3	30 November 1999	148.4	4.3

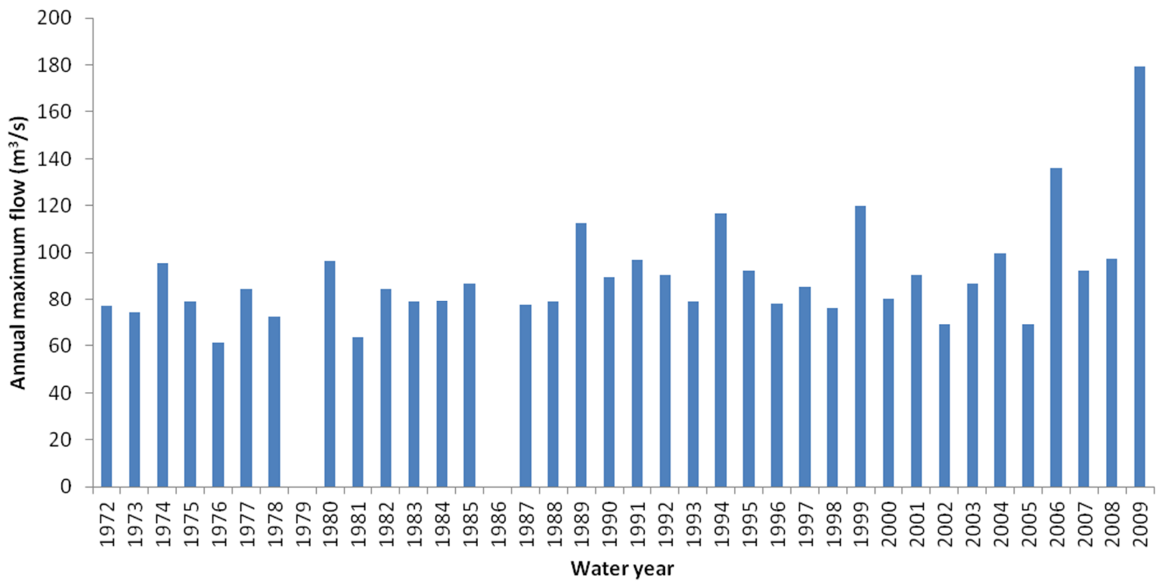
Tests for stationarity: Mann-Kendall test: significant increasing trend 	

There is a seasonal bias in the supplied AMAX data for this site, with nearly all significant floods occurring between October and April. The largest flood on record (1968) has a growth factor of approximately 2.2 whilst that of the second largest is 2.1. Statistical analysis indicates a significant increasing long term trend is present in this dataset whilst visual inspection identifies a step change in the late 1960s resulting from the extreme event of 1968.

Notes: Annual maxima flows and levels were sourced directly from the Flood Studies Update Programme for 1964-2004. Annual maxima levels supplied by OPW were extended using the current rating to calculate the annual maxima flows 2005-2009.

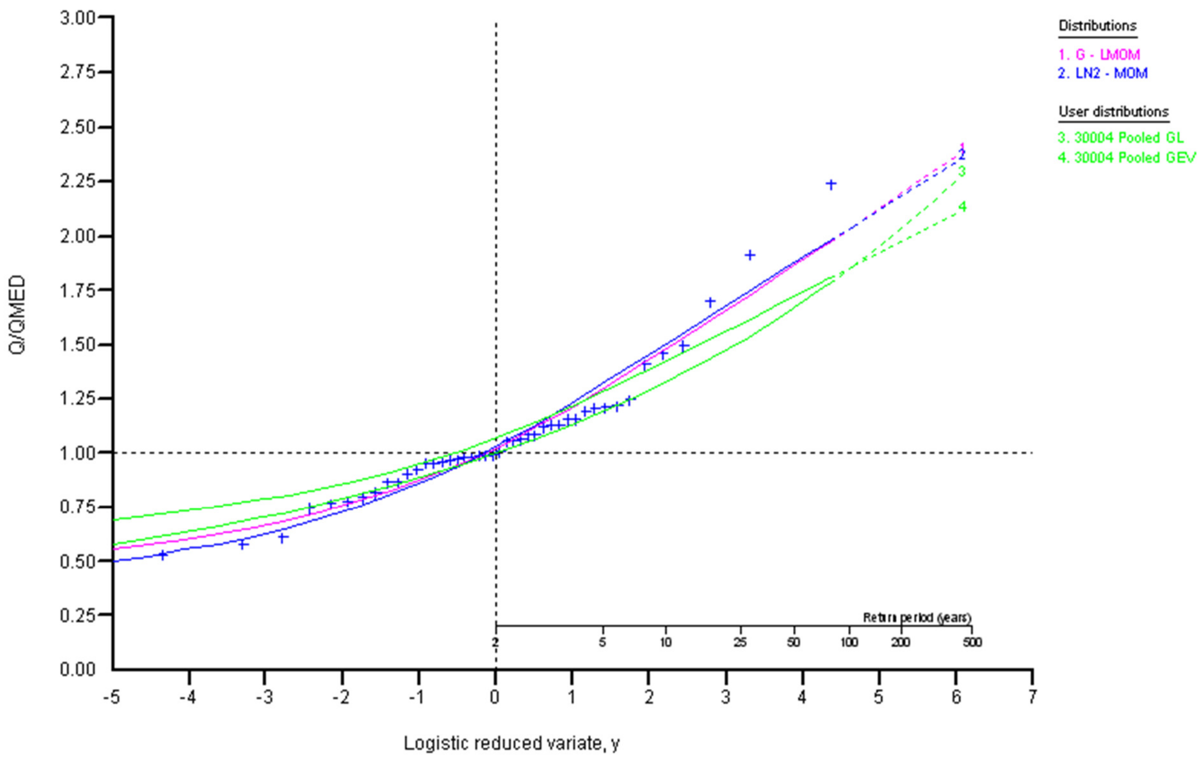
Analysis of revised flood peak series, after rating review

Note: The revised series excludes pre-1972 data which had been derived using a different rating from the rest of the dataset, not covered by the rating review. The pre-1972 rating was based on check gaugings up to $40m^3/s$, which meant that the 1968 flood (estimated as over $200m^3/s$) was based on a very large amount of extrapolation. Comments from OPW indicated that flood peaks were thought to be overestimated.



QMED (m^3/s): 84.703

Flood frequency analysis – comparison of single-site and pooled growth curves



Distribution	Location	Scale	Shape	100-year growth factor
Single-site Gumbel (L-moments)	0.938	0.238	n/a	2.034
Single-site LN2 (moments)	0.030	0.293	n/a	2.035
Pooled GL (L-moments)	1.000	0.124	-0.167	1.850
Pooled GEV (L-moments)	1.000	0.189	0.005	1.790

Comments on growth curves

There is little difference between the single-site curves. If the analysis had been carried out before November 2009, the single-site curves would have been less steep. The shallower pooled curves may under-estimate the November 2009 flood AEP – although even the single-site curves give an AEP of under 0.5%.

The historical review carried out in the inception phase found that the Nov 2009 flood was probably only the second largest flood on the Clare River in the last 90 years, but this was based on a comparison with the magnitude of the 1968 flood, which is now thought to have been over-estimated. There is no other record of flood peaks on the lower Clare River that extends back to 1968, so it is difficult to assess the relative magnitudes of the 1968 and 2009 floods.

Recommended growth curve

The single-site Gumbel curve is recommended as the preferred growth curve for design flood estimation. It gives a more realistic probability for the 2009 flood and is very similar to the pooled growth curve that is recommended further upstream on the Clare at Ballygaddy.

Recommended design flows[1] (Single-site Gumbel)								
AEPs	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.1%**
Flow (m^3/s)	84.7	109.7	124.9	139.4	158.2	172.3	204.9	218.9
Growth factor	1.0	1.3	1.5	1.6	1.9	2.0	2.4	2.6

Composition of pooling group

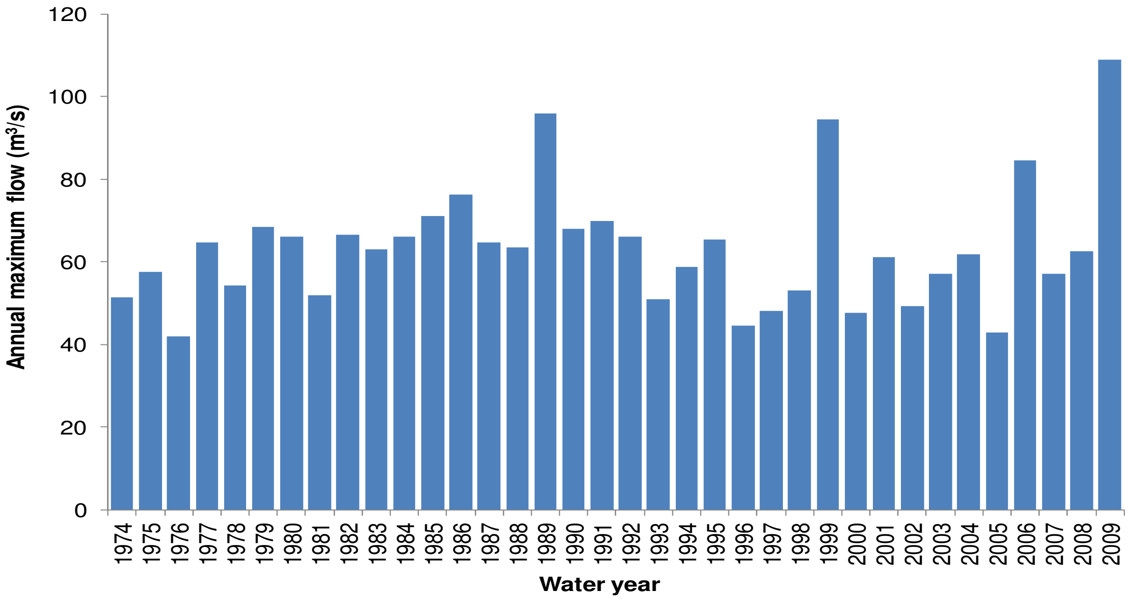
The stations in the pooling group have been selected as the most similar catchments in Ireland and Northern Ireland according to three descriptors at the subject site: AREA (699km^2), SAAR (1104mm) and BFIsoil (0.60).

ID	Rank	Watercourse	Location	Years
30004	1	Clare	Corrofin	44
26002	2	Suck	Rookwood	58
15004	3	Nore	McMahons Bridge	56
35005	4	Ballysadare	Ballysadare	62
26005	5	Suck	Derrycahill	58
30007	6	Clare	Ballygaddy	36
16008	7	Suir	New Bridge	56
26007	8	Suck	Bellagill Bridge	58
29011	9	Dunkellin	Kilcolgan	27
14005	10	Barrow	Portarlington	53

[1] *Final design flows have been developed from the recommended design flows at gauging station presented here but these have been further modified in some areas through regional smoothing of the QMED adjustment factor. In addition, for all HEPs the flood growth curve was extended for AEPs lower than 1% using ratios from FSR rainfall-runoff method growth curves. Please refer to Appendix F Design flows for the final design flows derived following these additional modifications.*

Flood frequency analysis summary sheet

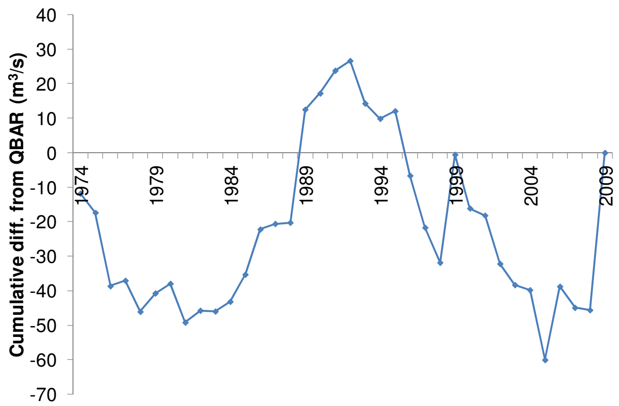
Station 30007	Clare @ Ballygaddy
Analysis of original flood peak series, from inception report	

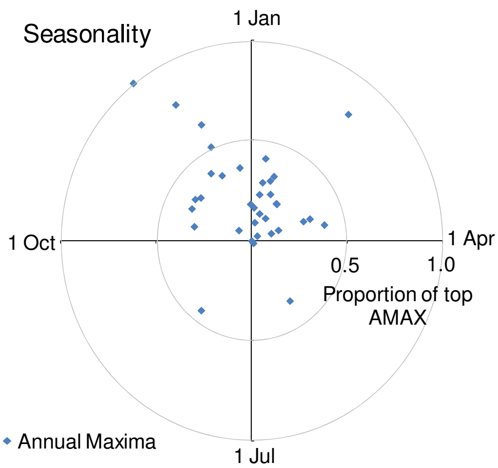


Top ranking floods:			QMED (m^3/s): 63.0
Rank	**Date**	**Flow (m^3/s)**	**AEP (%) from single-site analysis**
1	21 November 2009	108.8	1.0
2	07 February 1990	96.0	3.0
3	29 November 1999	94.5	3.5

Tests for stationarity:

Mann-Kendall test: no significant trend



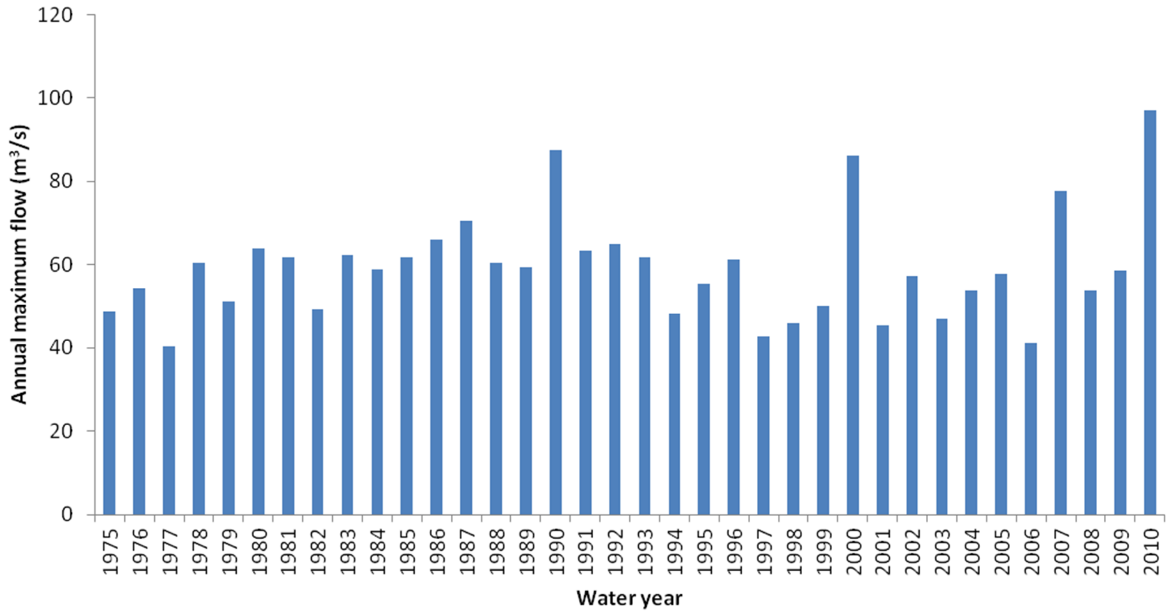


There is little variation in most AMAX flows, which generally are between 40 and 70m^3/s. This may reflect both the maritime climate and the considerable attenuation of peak flows due to the floodplain storage available in the upstream catchment. November 2009 was the highest on record but not by a great deal. None of the floods could be regarded as outliers. There is a distinct concentration of large floods in late November to early December, but February 1990 was an exception. This was not a

snowmelt event – weather at this time was mild and very stormy. There is no evidence of a long term trend within this dataset.

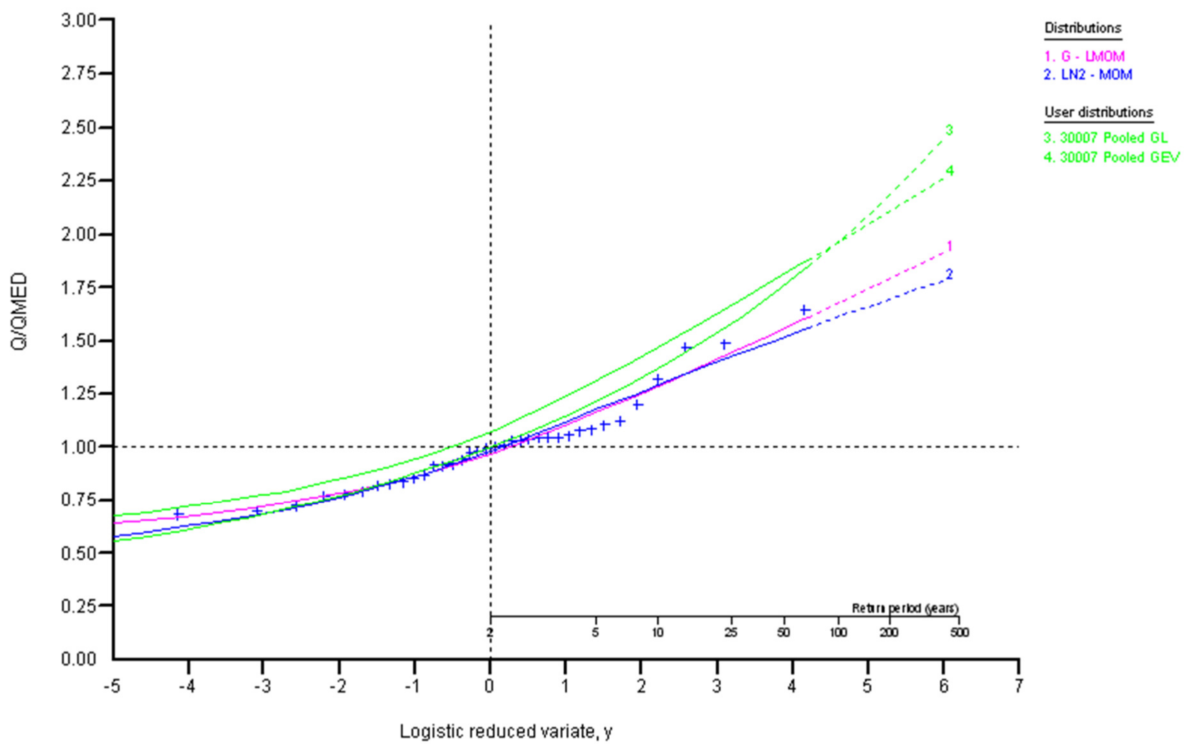
Notes: Annual maxima have been sourced directly from OPW.

Analysis of revised flood peak series after rating review



QMED (m^3/s): 58.801

Flood frequency analysis – comparison of single site and pooled growth curves



Distribution	Location	Scale	Shape	100-year growth factor
Single-site Gumbel (L-moments)	0.947	0.263	n/a	1.677
Single-site LN2 (moments)	-0.018	0.212	n/a	1.610
Pooled GL (L-moments)	1.000	0.143	-0.184	2.030
Pooled GEV (L-moments)	1.000	0.215	-0.022	1.960

Comments on growth curves

Pooled curves have a similar shape, becoming steeper at longer AEPs. The single-site curves are shallower than the pooled ones.

Recommended growth curve

From the information shown on the graph above it is difficult to choose between the growth curves. There is a theoretical preference for pooled growth curves as they bring in data from a wider group of stations, helping to overcome the possible under-estimation of the growth curve which may arise from the limited period of record available at Ballygaddy. Therefore the pooled GL curve is recommended as the preferred growth curve for design flood estimation.

Recommended design flows[1] (Pooled GL)

AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m^3/s)	58.8	72.1	81.6	91.7	106.7	119.6	134.3	176.2
Growth factor	1.0	1.2	1.4	1.6	1.8	2.0	2.3	3.0

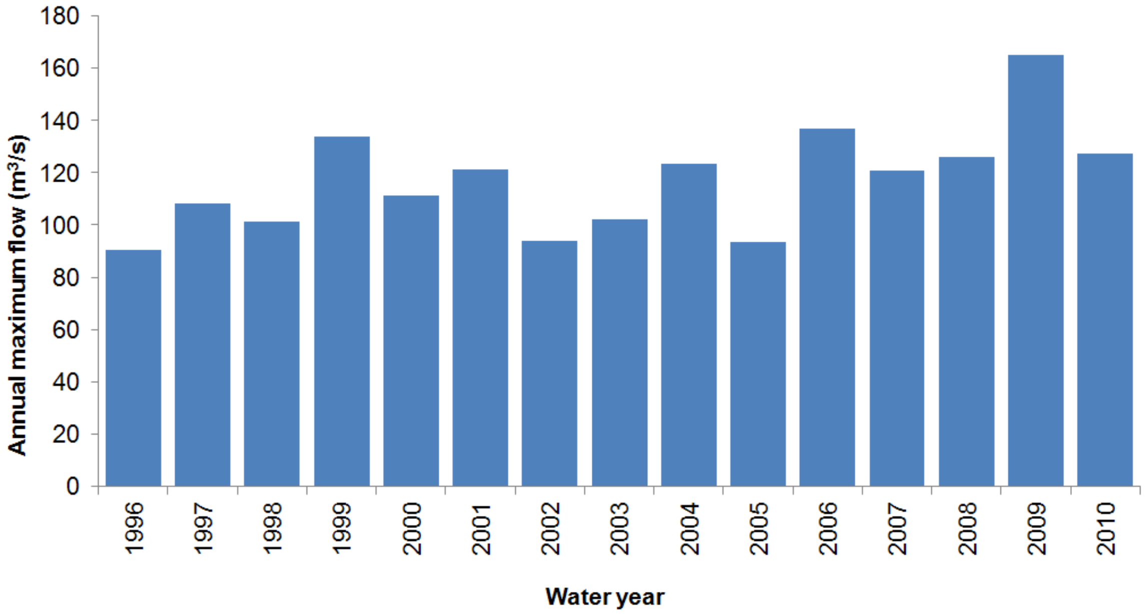
Composition of pooling group

The stations in the pooling group have been selected as the most similar catchments in Ireland and Northern Ireland according to three descriptors at the subject site: AREA (470km^2), SAAR (1115mm) and BFIsoil (0.65).

ID	Rank	Watercourse	Location	Years
30007	1	Clare	Ballygaddy	36
29011	2	Dunkellin	Kilcolgan	27
25029	3	Nenagh	Clarianna	38
29007	4	L. Cullaun	Craughwell	27
26002	5	Suck	Rookwood	58
15004	6	Nore	McMahons Bridge	56
30004	7	Clare	Corrofin	44
35005	8	Ballysadare	Ballysadare	62
16010	9	Anner	Anner	56
26108	10	Boyle	Boyle Abbey Bridge	20
25030	11	Graney	Scarriff Bridge	53
14005	12	Barrow	Portarlington	53

Flood frequency analysis summary sheet

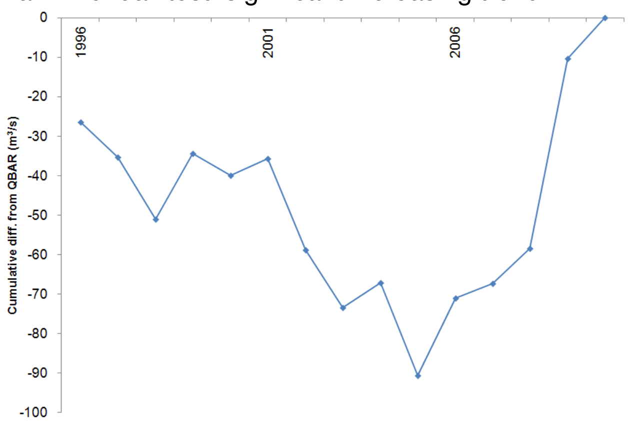
Station 30012	Clare @ Claregalway
Analysis of original flood peak series, from inception report	



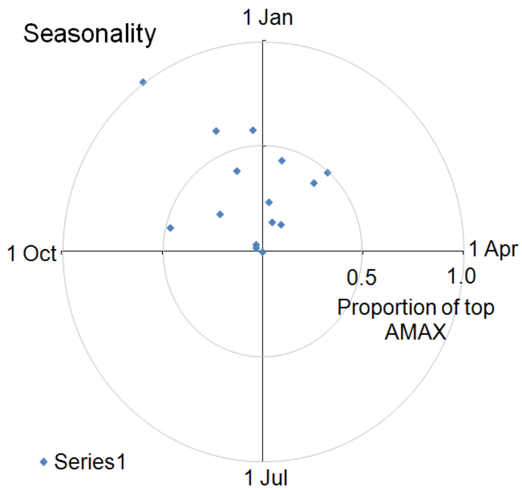
Top ranking floods: QMED (m^3/s): 120.8

Rank	Date	Flow (m^3/s)	AEP (%) from single-site analysis
1	22 November 2009	165.3	2.9
2	07 December 2006	136.7	15.7
3	24 December 1999	133.8	18.4

Tests for stationarity:
Mann-Kendall test: significant increasing trend



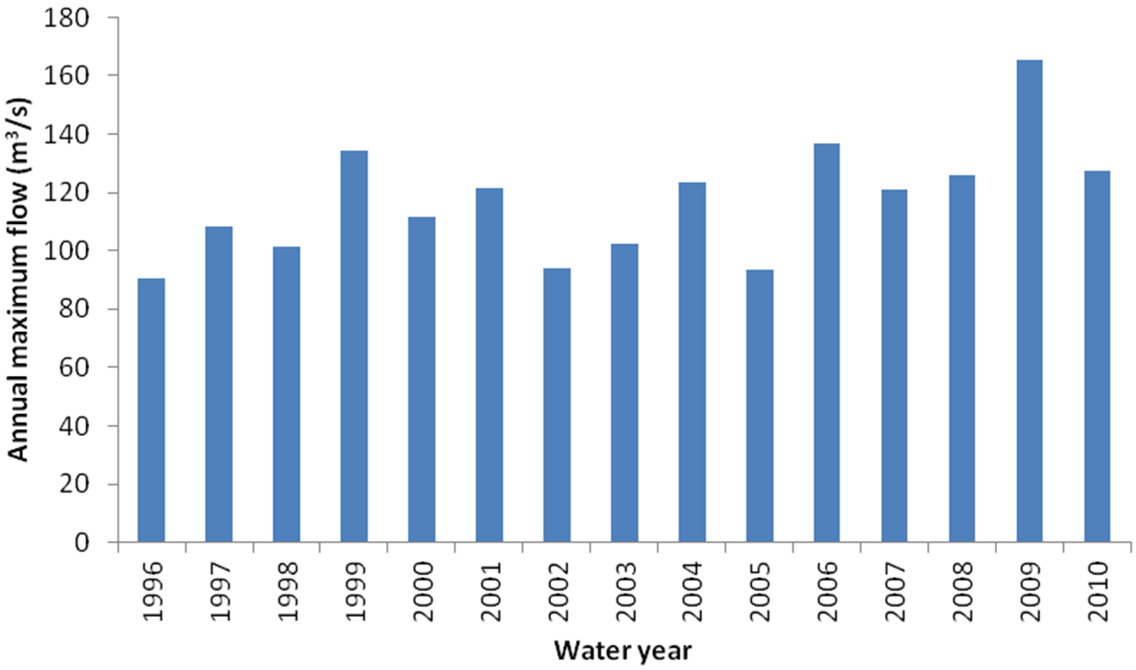
Seasonality



The majority of the 15 years of AMAX flood events have occurred in the winter months, potentially as a result of prolonged rainfall and saturated soils. The relatively narrow range of AMAX flows appears to reflect the good fit between the rating and gauged records. A significant increasing trend is present within this dataset.

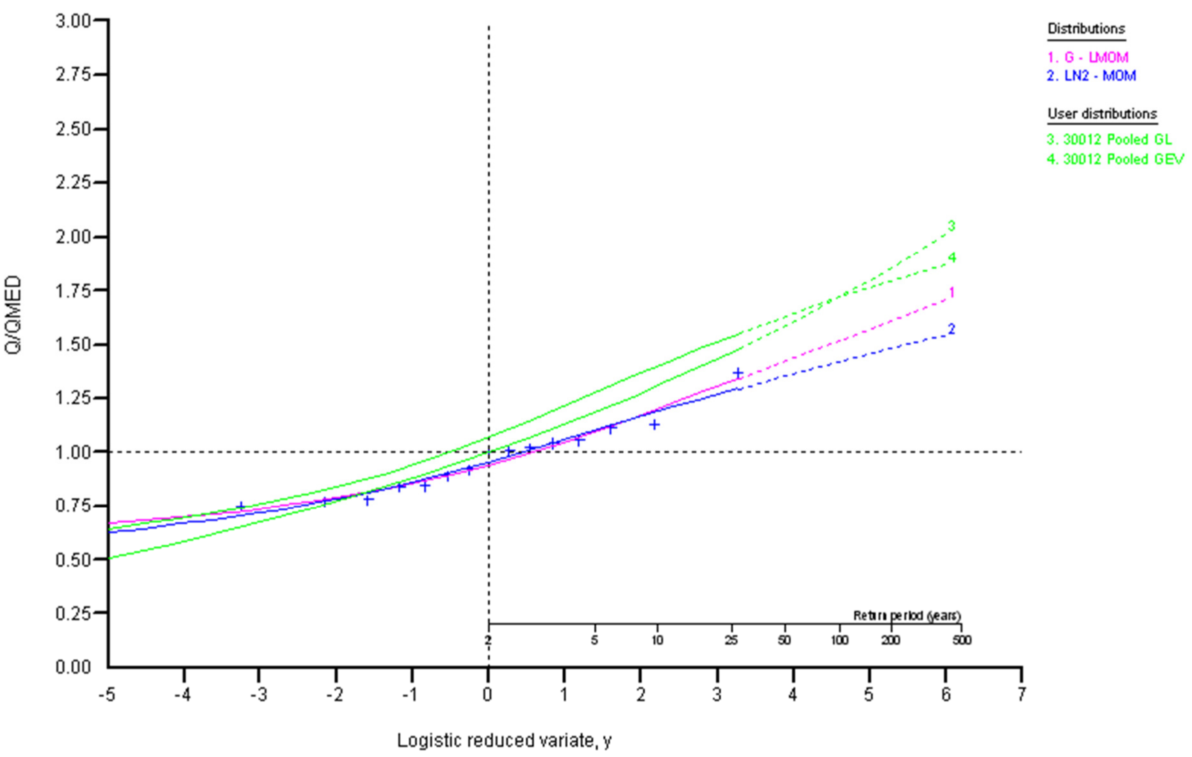
Notes: Annual maxima have been extracted from continuous data supplied by EPA.

Analysis of revised flood peak series, after rating review



QMED (m^3/s): 120.81

Flood frequency analysis – comparison of single-site and pooled growth curves



Distribution	Location	Scale	Shape	100-year growth factor
Single-site Gumbel (L-moments)	0.891	0.136	n/a	1.518
Single-site LN2 (moments)	-0.045	0.170	n/a	1.419
Pooled GL (L-moments)	1.000	0.124	-0.098	1.720
Pooled GEV (L-moments)	1.000	0.201	0.115	1.650

Comments on growth curves

The single-site curves are similar, with the Gumbel slightly steeper at higher return periods. This fits the AMAX series from the rating review better than the LN2 curve. The pooled curves are steeper than the single-site curves.

Recommended growth curve

The pooled GL distribution is recommended as preferable to the single-site curve as the short AMAX record may not give a reliable growth curve.

In addition, there is some concern that the flow measurements may be affected by downstream levels on Lough Corrib. Peak flows at Claregalway seem surprisingly low in comparison with upstream flows at Corrofin: for example the peak of the 2009 flood is 179m^3/s at Corrofin and 165m^3/s at Claregalway. This is despite the fact that QMED increases by 42% between Corrofin and Claregalway. The result is that the single-site growth curve at Corrofin is much steeper than that at Claregalway, with a 1% AEP growth factor of 2.03.

Since the single-site growth curve has been preferred at Corrofin, it may be preferable to apply this curve also at Claregalway, to promote spatial consistency (avoiding a drop in the growth factor over a short reach of river). The design flows below are therefore calculated using the single-site curve from Corrofin.

Recommended design flows[1] (Single-site Gumbel from Corrofin)

AEPs	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.1%**
Flow (m^3/s)	120.8	156.4	178.1	199.0	225.7	245.7	268.3	360.3
Growth factor	1.0	1.3	1.5	1.6	1.9	2.0	2.2	3.0

Composition of pooling group

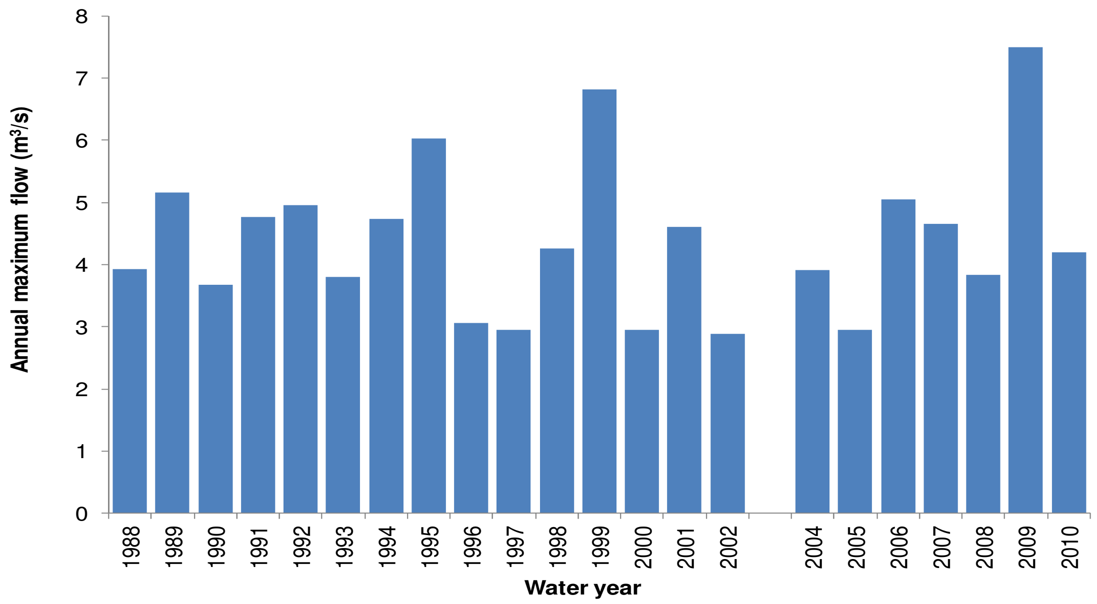
The stations in the pooling group have been selected as the most similar catchments in Ireland and Northern Ireland according to three descriptors at the subject site: AREA (1073 km^2), SAAR (1099 mm) and BFIsoil (0.54).

ID	Rank	Watercourse	Location	Years
30012	1	Clare	Claregalway	15
26007	2	Suck	Bellagill Bridge	58
30004	3	Clare	Corrofin	44
26002	4	Suck	Rookwood	58
203012	5	Ballinderry	Ballinderry Bridge	40
16009	6	Suir	Caher Park	57
16008	7	Suir	New Bridge	56
26005	8	Suck	Derrycahill	58
15004	9	Nore	McMahons Bridge	56
24082	10	Maigue	Islandmore	33
24001	11	Maigue	Croom	34

Flood frequency analysis summary sheet

Station 30020	Dalgan @ Ballyhaunis

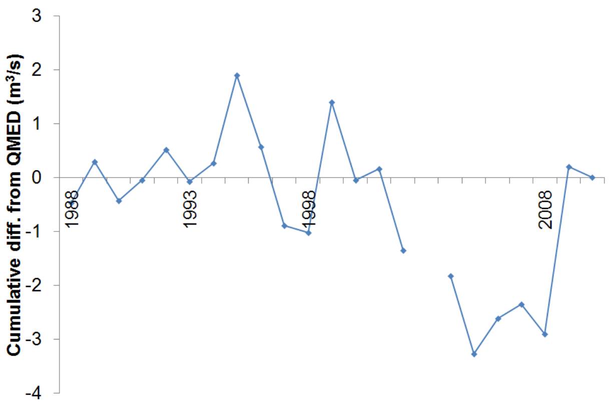
Analysis of original flood peak series, from inception report

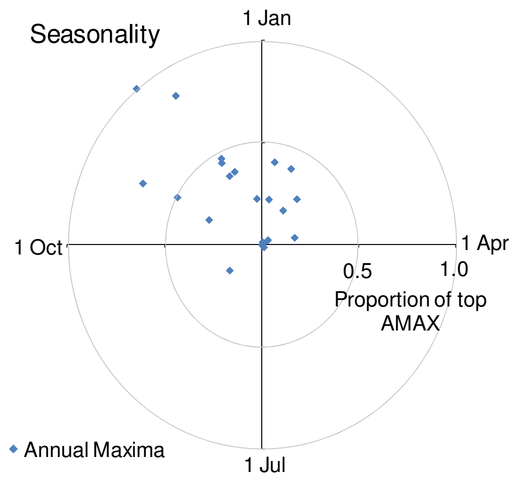


Top ranking floods: QMED (m^3/s): 4.2

Rank	Date	Flow (m^3/s)	AEP (%) from single-site analysis
1	19 November 2009	7.5	2.3
2	28 November 1999	6.8	4.6
3	26 October 1995	6.0	10.0

Tests for stationarity:
Mann-Kendall test: no significant trend

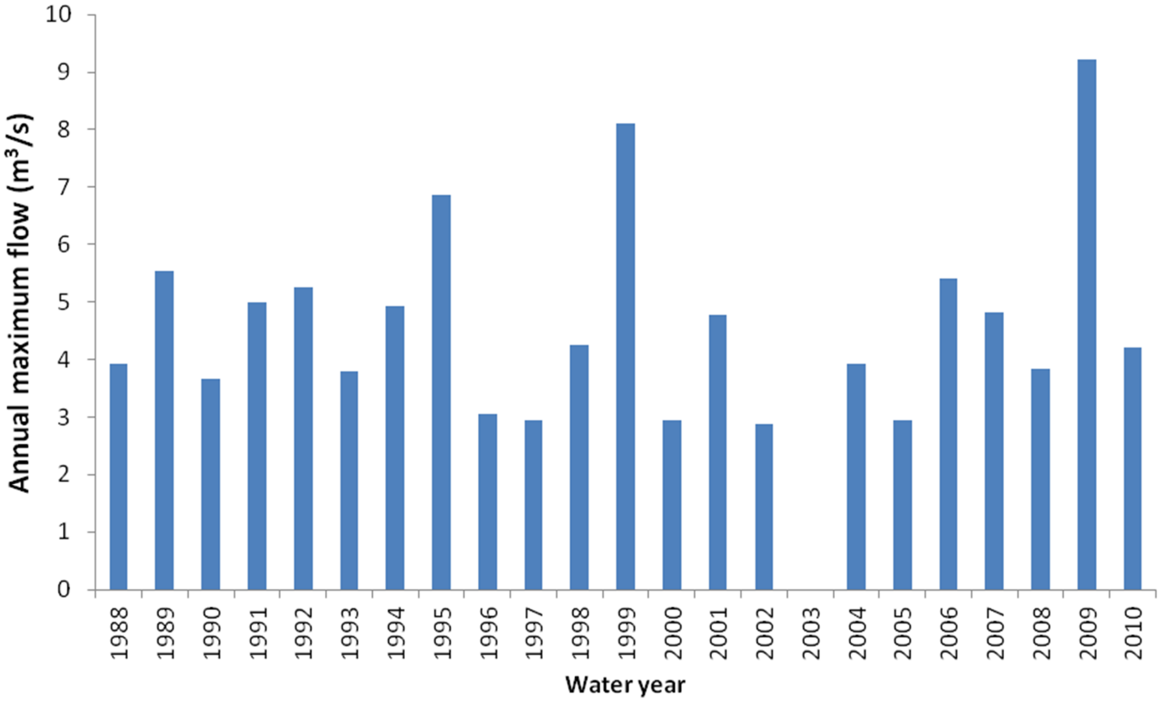




There is a slight seasonal bias in the supplied AMAX data for this site, with nearly all significant floods occurring between October and April. However, annual peak flows have occurred throughout the year, with those of 2005 and 2008 occurring in May and August respectively. The largest flood on record (2009) has a growth factor of approximately 1.8. Statistical analysis indicates no significant long term trend is present in this dataset whilst visual inspection identifies no sudden step changes.

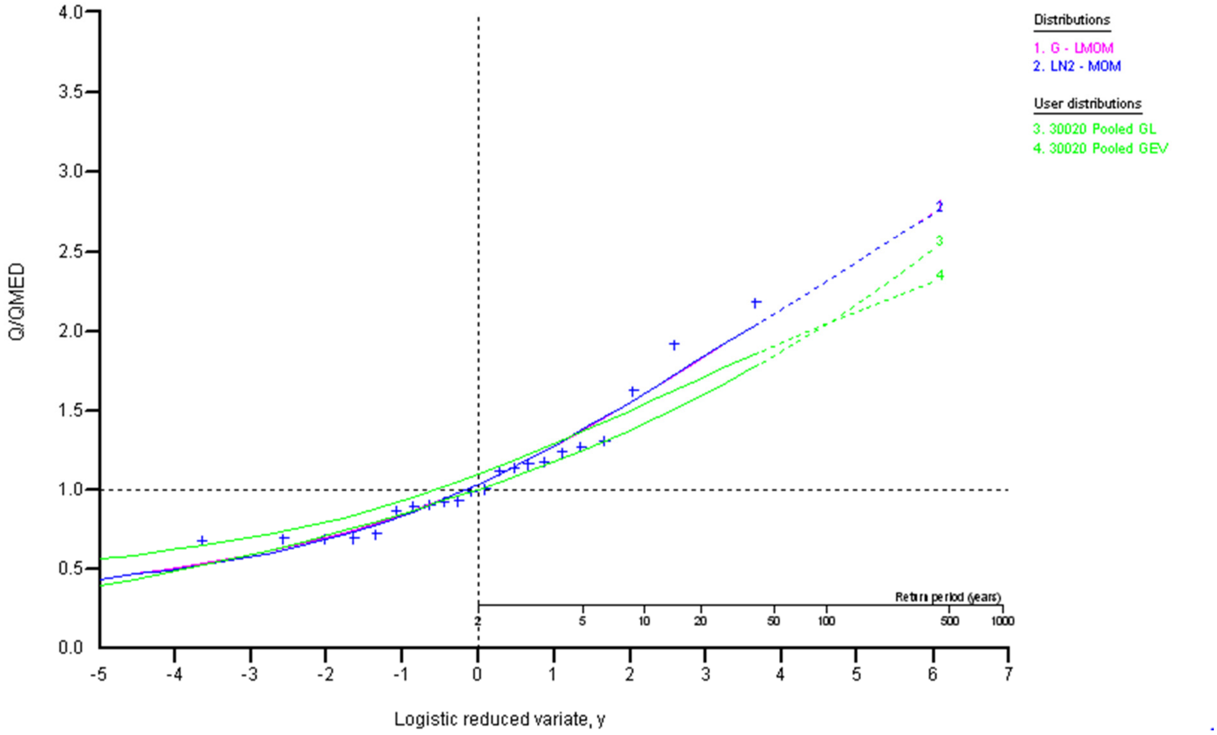
Notes: Annual maxima have been extracted from continuous data supplied by EPA, no data was available for 2003 due to a logger malfunction.

Analysis of revised flood peak series, after rating review



QMED (m^3/s): 4.230

Flood frequency analysis – comparison of single-site and pooled growth curves



Distribution	Location	Scale	Shape	100-year growth factor
Single-site Gumbel (L-moments)	0.924	0.303	n/a	2.317
Single-site LN2 (moments)	0.035	0.346	n/a	2.316
Pooled GL (L-moments)	1.000	0.162	-0.148	2.060
Pooled GEV (L-moments)	1.000	0.251	0.035	1.970

Comments on growth curves

There is little difference between the single-site curves. The single-site curves are steeper than pooled curves. This is likely a result of the November 2009 flood and the two other top ranking floods steepening the single-site curves. The November 2009 event has a 2% AEP using the single site analysis compared to a 0.5% AEP using the pooled analysis. Whilst the 0.5% AEP produced from the pooled analysis does seem high, the 2009 event is known to have been significant compared to historical events. Furthermore, if the 2009 event is extreme, as the period of record available increases at the gauge site, the single site analysis will tend towards the pooled analysis and the estimated frequency of the event will decrease below a 2% AEP. This is supported by the gauged record at the Ballygaddy gauge, located downstream on the same watercourse, where the 2009 event is the largest in close to 40 years of data and the single site analysis accordingly estimates the event to have a 1% AEP.

There are only 22 years of data available at the gauge and it would not be appropriate to extrapolate this data up to an event with a 2% AEP. The short data period means that the pooled analysis is the preferred approach to develop the design flows and over time as the significance of the 2009 events is clarified it would be expected that the single-site curves will become similar to the pooled curves.

Recommended growth curve

Given the shortness of the flood peak record, the pooled growth curve is recommended. The GL is preferred given its similarity to the single-site curve for low AEPs.

Recommended design flows[1] (Pooled GL)

AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m^3/s)	4.2	5.3	6.0	6.7	7.7	8.5	9.3	11.7
Growth factor	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.8

Composition of pooling group

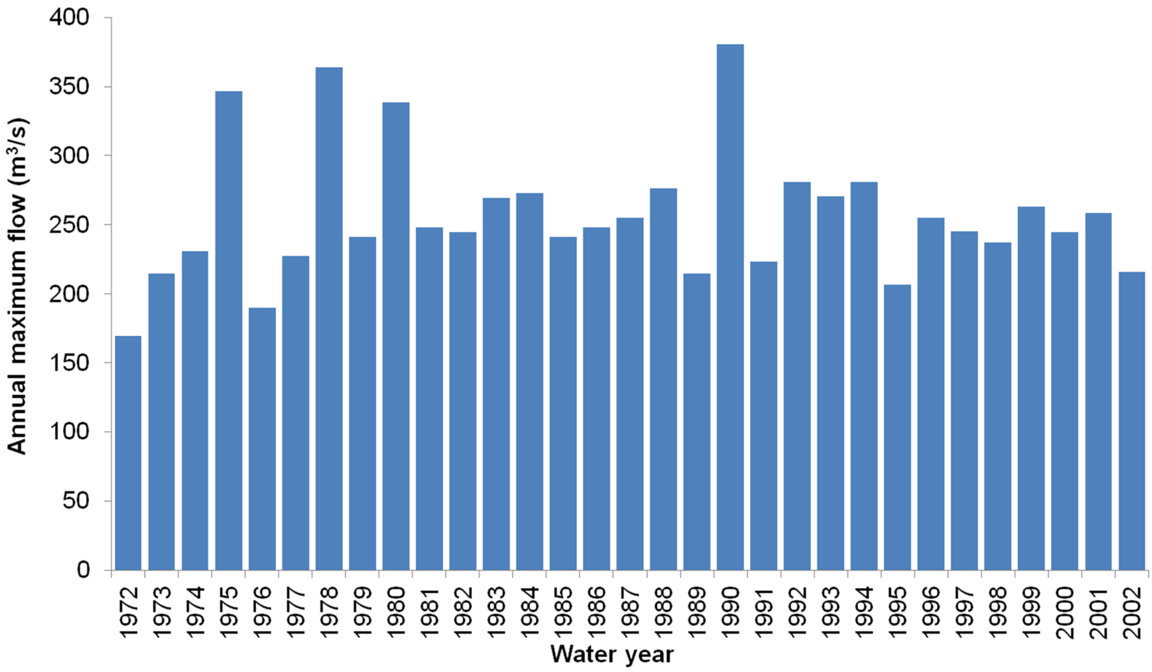
The stations in the group have been selected as the most similar catchments in Ireland and Northern Ireland according to three descriptors at the subject site: AREA (21.4km^2), SAAR (1191mm) and BFIsoil (0.68).

ID	Rank	Watercourse	Location	Years
30020	1	Dalgan	Ballyhaunis	22
19020	2	Owennacurra	Ballyedmond	28
29010	3	Aggard	Aggard Bridge	28
25034	4	L. Ennel Trib	Rochfort	26
24022	5	Mahore	Hospital	20
26018	6	Owenure	Bellavahan	54
25040	7	Bunow	Roscrea	19
29004	8	Clarinbridge	Clarinbridge	37

25044	9	Kilmastulla	Coole	40
25027	10	Ollatrim	Gourdeen Bridge	48
26022	11	Fallan	Kilmore	38
06070	12	Muckno L	Muckno	27
6012	13	Fane	Clarebane	40
34018	14	Castlebar	Turlough	34
25014	15	Silver	Millbrook Bridge	55

Flood frequency analysis summary sheet

Station 30061	Corrib @ Wolfe Tone Bridge



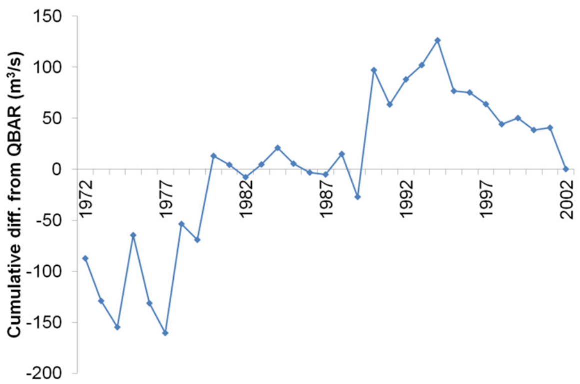
Top ranking floods:

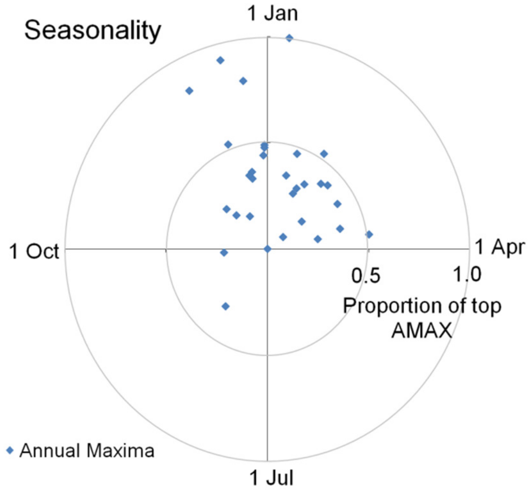
QMED (m^3/s): 248.0

Rank	Date	Flow (m^3/s)	AEP (%) from single-site analysis
1	05 January 1991	380.8	2.2
2	15 December 1978	363.6	3.6
3	02 December 1975	346.8	5.5

Tests for stationarity:

Mann-Kendall test: no significant trend



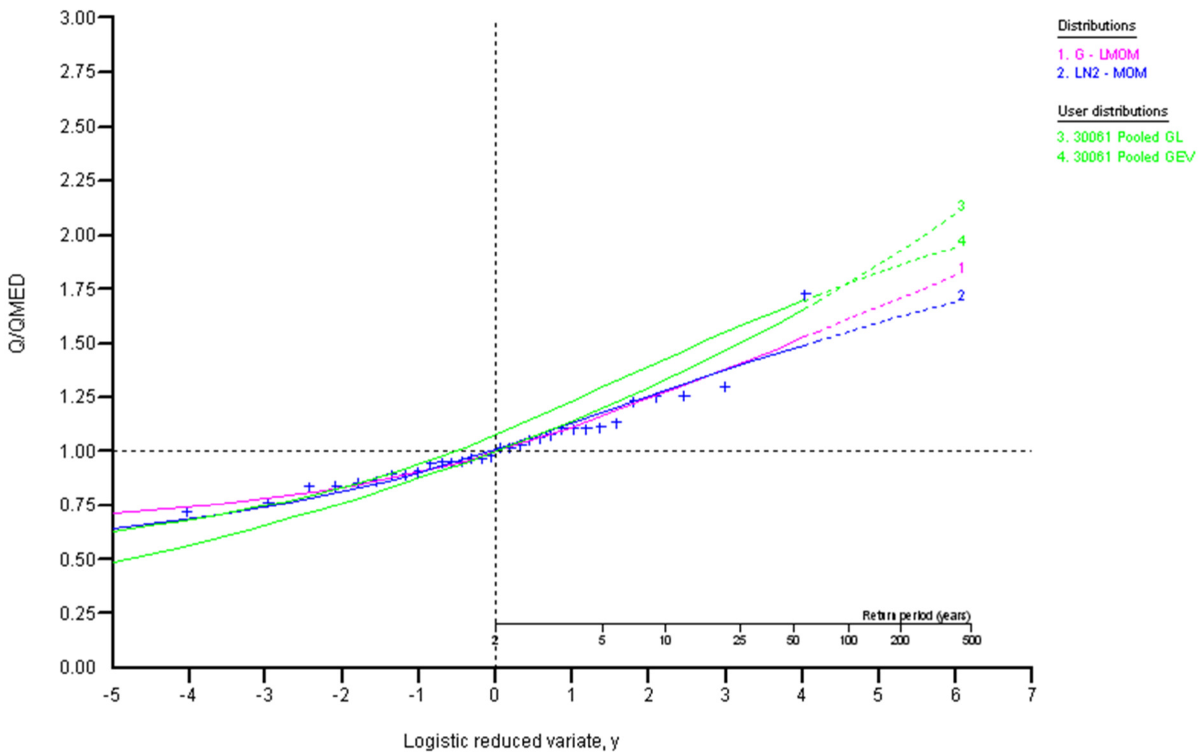


The period of record at this site is limited to 32 water years as there are no recorded gaugings dated pre -1972. In addition, this gauging station was moved in 2004 from a location upstream of the bridge to downstream of the structure. Since relocation, no stage-discharge relationship has been discerned form the gauged data and therefore no AMAX flow estimates are available from mid-2004.

The majority of annual maximum flows have occurred between November and April, with one event occurring in August 1985. The largest event has a growth factor of 1.7 whilst that of the second largest event is 1.3, indicating a large spread in the magnitude of the high extreme annual maximum flows. There is no evidence of a significant trend within this data series.

Notes: Annual maxima have been sourced directly from OPW. A rating review was not undertaken for this site.

Flood frequency analysis – comparison of single-site and pooled growth curves



Distribution	Location	Scale	Shape	100-year growth factor
Single-site Gumbel (L-moments)	0.945	0.144	n/a	1.609
Single-site LN2 (moments)	0.011	0.184	n/a	1.550
Pooled GL (L-moments)	1.000	0.132	-0.104	1.780
Pooled GEV (L-moments)	1.000	0.212	0.105	1.700

Comments on growth curves

There is little difference between the single-site curves. The single-site curves are shallower than the pooled. This is despite the January 1975 flood which did not cause the single-site curves to steepen significantly.

The Corrib catchment at Galway is unusual as it is very large and subject to extremely large flood attenuation due to the size of Loughs Corrib and Mask immediately upstream. For this reason it is difficult to have much confidence that many of the gauges in the pooling group are representative of the hydrological response of the Corrib catchment. The top-ranking gauges in the group are both on the Moy, which has a similar size to the Corrib and is also subject to major influence from loughs, although a substantial part of the Moy catchment does not drain through the loughs.

On the other hand, the single-site growth curve is based on only 31 years of data from Galway, years which exclude 1968 and 2009 during which the highest floods on record occurred upstream on the Clare River. The period of record at Galway also excludes the years with the highest recorded levels

of Lough Corrib (2008 and 2009). Furthermore, there are doubts over the quality of the flood peak data at Wolf Tone Bridge given the tidal influence and the poor correlation between annual maximum flows at Wolf Tone Bridge and annual maximum levels on Lough Corrib.

According to the single-site growth curves, the January 1975 flood had an AEP of between 0.5% and 0.02%. This is considered likely to be an underestimate of the probability of the event given evidence of higher flows and levels at upstream gauges in recent years and the results from the historic review which found evidence of at least one other severe flood on the River Corrib, in 1852 (see the inception report). The pooled curves appear more realistic, giving a return period of just over 50 years to the 1975 flood.

Recommended growth curve

On balance, the pooled analysis is considered more realistic. The GL curve is recommended as the preferred growth curve, since it avoids the upper bound that occurs for the GEV curve.

Recommended design flows[1] (Pooled GL)

AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m^3/s)	248.0	296.8	328.8	360.8	405.0	440.9	479.1	579.0
Growth factor	1.0	1.2	1.3	1.5	1.6	1.8	1.9	2.3

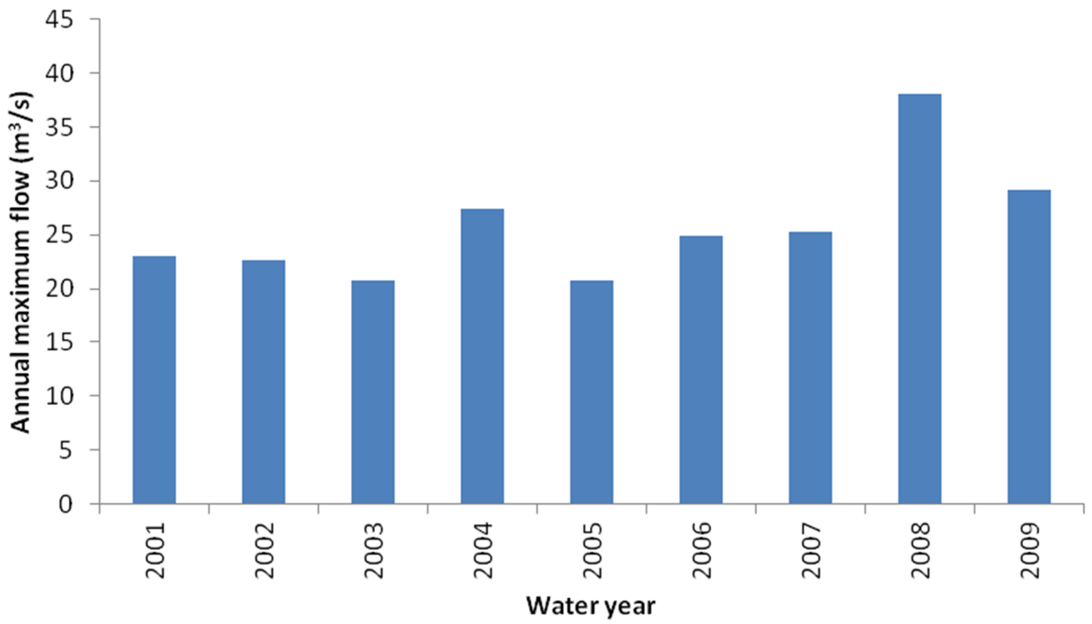
Composition of pooling group

The stations in the pooling group have been selected as the most similar catchments in Ireland and Northern Ireland according to three descriptors at the subject site: AREA ($3136km^2$), SAAR (1422mm) and BFIsoil (0.81).

ID	Rank	Watercourse	Location	Years
34001	1	Moy	Rahans	37
34003	2	Moy	Foxford	36
12002	3	Slaney	Enniscorthy	31
12001	4	Slaney	Scarawalsh	55
27002	5	Fergus	Ballycorey	56
35073	6	Lough Gill	Lough Gill	30
16011	7	Suir	Clonmel	57
26108	8	Boyle	Boyle Abbey Bridge	20
35012	9	Garvogue	New Bridge	10
16009	10	Suir	Caher Park	57
35005	11	Ballysadare	Ballysadare	62
25017	12	Shannon	Banagher	39
26005	13	Suck	Derrycahill	58

Flood frequency analysis summary sheet

Station 30101	Owenriff @ Oughterard
Analysis of flood peak series, after rating review (no rating existed before the CFRAM)	



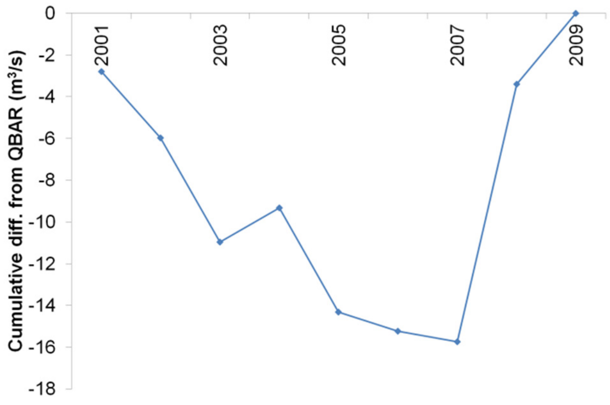
Top ranking floods:

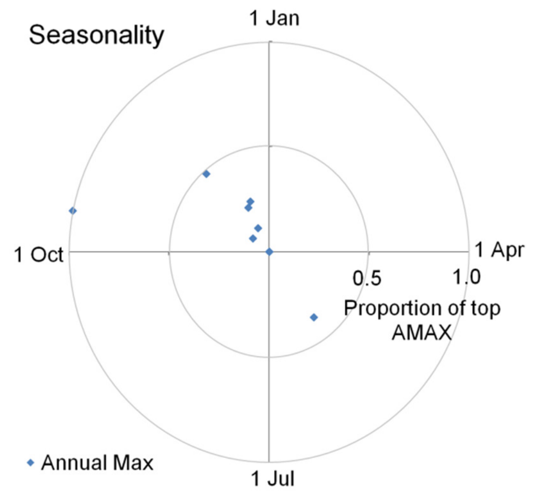
QMED (m^3/s): 28.8

Rank	Date	Flow (m^3/s)
1	10 October 2008	38.1
2	19 November 2009	29.2
3	25 May 2005	27.4

Tests for stationarity:

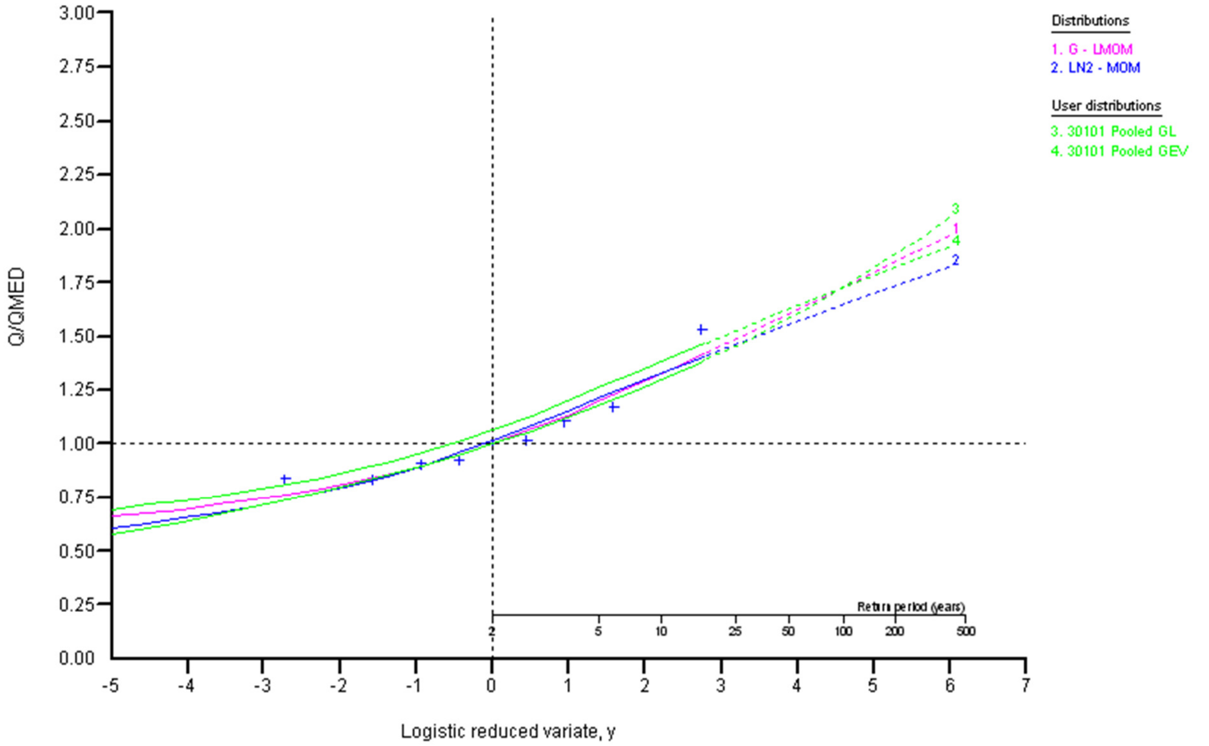
Mann-Kendall test: n/a – record too short.





A rating equation was derived for the nine year stage data series at Oughterard. The highest annual maximum flow occurred in the October 2008 event. All but one AMAX flow occurred between October and January, the other AMAX flow occurred in May 2005.

Flood frequency analysis – comparison of single-site and pooled growth curves



Distribution	Location	Scale	Shape	100-year growth factor
Single-site Gumbel (L-moments)	0.938	0.172	n/a	1.728
Single-site LN2 (moments)	0.015	0.208	n/a	1.648
Pooled GL (L-moments)	1.000	0.124	-0.134	1.790
Pooled GEV (L-moments)	1.000	0.195	0.057	1.720

Comments on growth curves

There is little difference between the single-site and the pooled GEV curves. The pooled GL curve is a little higher than the other three curves and may overestimate design flows for moderate return periods. The single-site Gumbel and pooled GEV curves are most similar as the pooled GL curve is a little lower at longer AEPs.

Recommended growth curve

The pooled GL curve is recommended as the growth curve for design flow estimation. At this gauge it gives almost identical results to the single site Gumbel curve. The pooled analysis is preferable at this site due to the short nine year gauged record.

Recommended design flows[1] (Pooled GL)

AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m^3/s)	28.8	34.3	38.1	41.9	47.0	51.7	56.6	69.6
Growth factor	1.0	1.2	1.3	1.5	1.6	1.8	2.0	2.4

Composition of pooling group

The stations in the group have been selected as the most similar catchments in Ireland and Northern Ireland according to three descriptors at the subject site: AREA (63.5km^2), SAAR (1918mm) and BFIsoil (0.486).

ID	Rank	Watercourse	Location	Years
30101	1	Owenriff	Oughterard	9
30019	2	Owenriff	Claremount	27
32006	3	Carrowbeg	Coolloughra	6
35028	4	Bonet	New Bridge (Manorhamilton)	20
31002	5	Cashla	Cashla	26
32012	6	Newport	Newport Weir	31
33070	7	Carrowmore L.	Carrowmore	28
201008	8	Derg	Castlederg	34
35002	9	Owenbeg	Billa Bridge	25
25158	10	Bilboa	Cappamore	18
27070	11	L. Inchiquin	Baunkyle	29
27003	12	Fergus	Corrofin	48
34018	13	Castlebar	Turlough	34
236007	14	Sillees	Drumrainey Bridge	28
34007	15	Deel	Ballycarroon	61
32011	16	Bunowen	Louisburgh Weir	27
203039	17	Clogh	Tullynewey	28
203033	18	Upper Bann	Bannfield	34

C Historical flood chronology

Flood chronology

This appendix provides results from analysis of flood history for UoM 30 and 31. Historic flood records were collected from sources such as local newspapers, previous studies, OPW's National Flood Hazard Mapping website, publications on flood history and other relevant websites. Dates and magnitude of more recent events were obtained from hydrometric records. The information was reviewed in order to provide qualitative and, where possible, also quantitative information on the longer-term flood history in the area. Further details relating to the specific flood history of individual AFAs are provided in the relevant Flood Risk Review Reports[1].

The table below gives a chronology of flood events, including information on their impacts.

Date	Catchment/River	Details
October 1816	Co. Galway	The whole countryside from Ballinasloe, Co Galway, to Moate, Co Westmeath, was under water (Hickey, 2010).
24 December 1852	River Corrib	Within the last week immense quantities of rain have fallen and such a body of water has not been seen rolling down the river Corrib for the last 50 years. The great increase of water is not so much owing to the great falls of rain, as to the opening of Lough Mask into Lough Corrib, under the late drainage operation (Freemans Journal, 1763 – 1924).
September 1927	Bunnakill	"Phenomenal rain", land slides (Irish Independent, 1905 – 2011).
10 February 1928	North Co. Galway to Headford	Flooding worst in memory (Irish Independent, 1905 – 2011)
	River Clare	Ballindooley was reported as a lake, water at level with the highway (N84) that stands from 3 feet and more above the lowland. River Clare has expanded 10x its normal width (Irish Independent, 1905 – 2011).
20 January 1932	North Galway	Worst flooding for many years (Irish Independent, 1905 – 2011).
13 December 1948	Claremorris	Over a foot of flood water was present within houses (Independent, 1905 – 2011).
23 September 1950	Galway and Tuam	The worst storm for 40 years (Tuam Herald, 1837 – 2000).
18 December 1954	Caherlistrane	Flooding worst in last 54 years, road to Beaghmore impassable since November (Connacht Tribune, 1909-2011).
	Killour	Flooding worst in 102 years (Connacht Tribune, 1909-2011).
	South-east Galway	Previous high water marks exceeded (Connacht Tribune, 1909-2011).

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

1968	River Clare	Worst flooding in North Galway for over 45 years. (Connacht Tribune, 1909-2011). This is also the highest annual maximum recorded at the Corrofin gauge (30004) on the River Clare, about 35km upstream of Galway.
February 1990	Claregalway River Corrib - Clare	Flooding was recorded by residents in the area of Lakeview estate
1991	Claregalway	Flooding was recorded by residents in the area of Lakeview estate
17th January 1995	Galway City	Flooding of houses, shops and streets occurred on 17th Jan 1995 in Galway City as a result of heavy rainfall and storm coupled with very high tide.
20th January 1995	Galway City	Flooding attributable mainly to melting snow and heavy rainfall. The main areas affected were Quay Street, Flood St, The Docks area and Lower Salthill.
January 1995	Tuam	Extensive land flooding in the Tuam area in 1995.
	Claregalway	Flooding was recorded by residents in the area of Lakeview estate
	Two Mile Ditch	The N17 was closed.
September 1998	Maam, Co. Galway	September 1998 there was minor flooding to 2 properties in Maam.
November 1999	Owenriff River	Significant flood affecting properties in Oughterard in 1999, with more than 19 properties impacted.
	Tuam	Extensive land flooding in the Tuam area in 1999 from the River Nanny.
	Claregalway	Flooding was recorded by residents in the area of Lakeview estate
	Two Mile Ditch	The N17 at Two mile ditch caused a road closure.
	Ballyhaunis	Flooding of a Joinery on the River Clare
February 2002	Claregalway	Flooding was recorded by residents in the area of Lakeview estate
	Galway City	Flooding in Galway City around the Spanish Arch, Fr Griffin Road, and Claddadh from Heavy rain, gale force winds and high time.
	Tuam	Reported land flooding in the Tuam area in 2002.
January 2005	Galway City	Flooding in Galway City along Quay st, Flood St, Spanish Arch, the Docks and Lower Salthiss due to high tides, wind direction and heavy rain.
	Tuam	Extensive land flooding in the Tuam area in 1999 from the River Nanny.

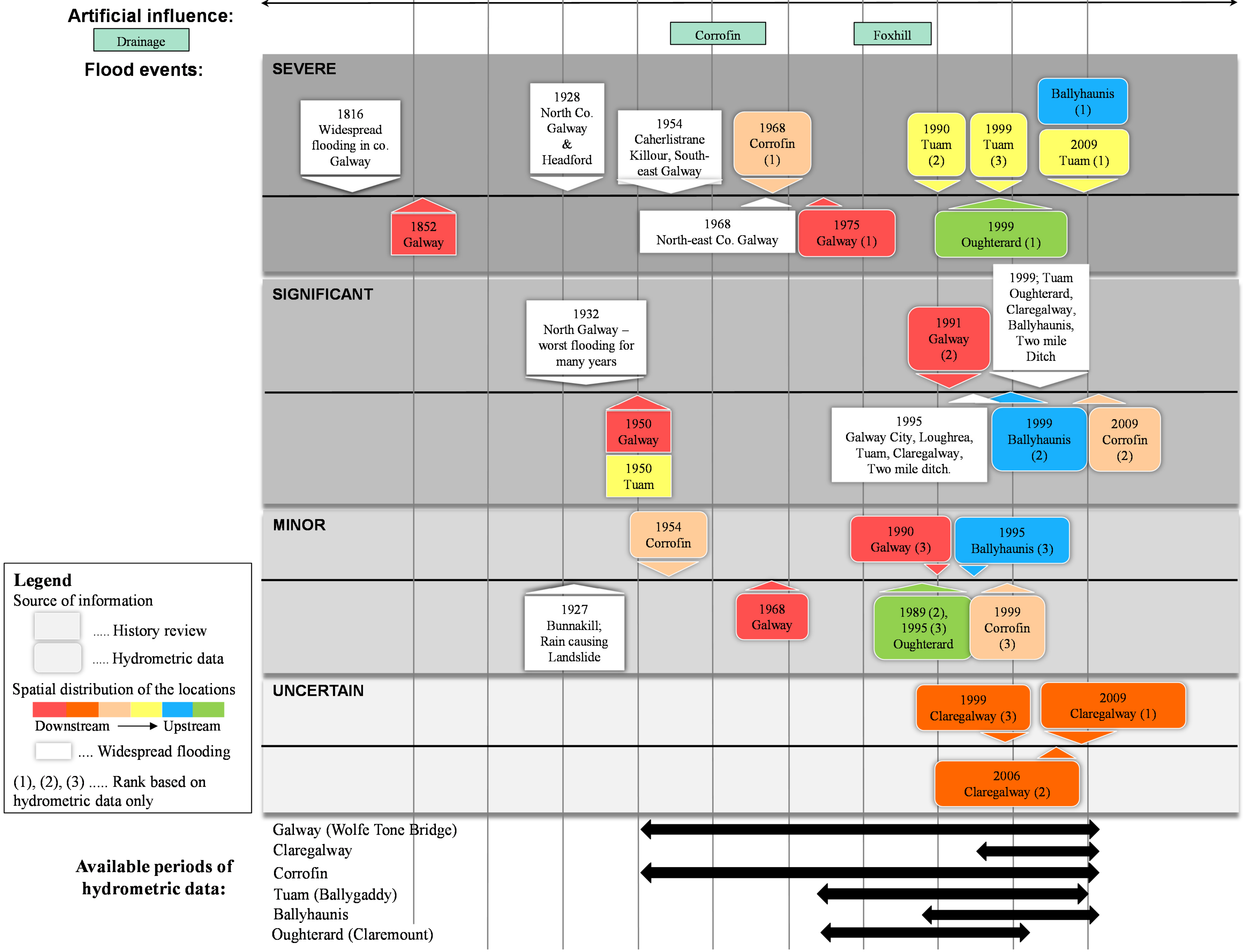
	Claregalway	Flooding was recorded by residents in the area of Lakeview estate and Friary Estate
23 November 2009	River Clare	There was also flooding of land along the River Clare banks at this time
	Abbert River, Corrib.	Extensive flooding in areas around Corrofin (Bullaun, Annagh, Ballglunin, Ballybanagher). Flooding in Ardskeamore near Corrofin was mainly due to groundwater flooding.
	Two Mile Ditch	The N17 at Two mile ditch caused a road closure.
	Ballyhaunis	Flooding of properties from the River Daigan in Ballyhaunis.

Based on the outcomes of the analysis, a flood history time line was produced. The time line provides a comprehensive overview of the main flooding events by putting together key events extracted from the available hydrometric data (usually limited to the top three events indicated by rank 1-3), and the events identified in the collated information on historic flooding. The time line sheet also includes locations of the flood events and indicates spatial distribution of these locations (i.e. downstream or upstream along a watercourse).

Four levels of flood severity are used in the table, namely “Severe”, “Significant”, “Minor” and “Unknown” classifications. These are indicative only and are based on the available quantitative and qualitative flood history information. The table below provides details of the classification.

Flood severity classification	AEP (from available data)	Flood severity from historic information
Severe	< 4%	Greatest flood in more than 25 years and/or widespread flooding covering area
Significant	4% - 10%	Widespread flooding
Minor	> 10%	Other
Uncertain	N/A	Other

UoM 30

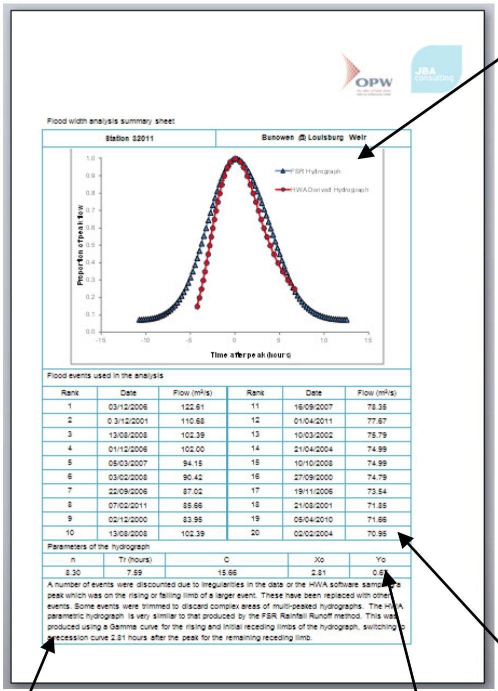

Artificial influence:
Drainage
Flood events:
<1850
1900
1925
1950
1960
1970
1980
1990
2000
2010
Corrofin
Foxhill
SEVERE
1816 Widespread flooding in co. Galway
1928 North Co. Galway & Headford
1954 Caherlistrane Killour, South-east Galway
1968 Corrofin (1)
1990 Tuam (2)
1999 Tuam (3)
Ballyhaunis (1)
2009 Tuam (1)
1852 Galway
1968 North-east Co. Galway
1975 Galway (1)
1999 Oughterard (1)
SIGNIFICANT
1932 North Galway – worst flooding for many years
1991 Galway (2)
1999; Tuam Oughterard, Claregalway, Ballyhaunis, Two mile Ditch
1950 Galway
1950 Tuam
1995 Galway City, Loughrea, Tuam, Claregalway, Two mile ditch.
1999 Ballyhaunis (2)
2009 Corrofin (2)
MINOR
1954 Corrofin
1990 Galway (3)
1995 Ballyhaunis (3)
1927 Bunnakill; Rain causing Landslide
1968 Galway
1989 (2), 1995 (3) Oughterard
1999 Corrofin (3)
UNCERTAIN
1999 Claregalway (3)
2009 Claregalway (1)
2006 Claregalway (2)
Legend
Source of information
..... History review
..... Hydrometric data
Spatial distribution of the locations
Downstream Upstream
.... Widespread flooding
(1), (2), (3) ..... Rank based on hydrometric data only
Available periods of hydrometric data:
Galway (Wolfe Tone Bridge)
Claregalway
Corrofin
Tuam (Ballygaddy)
Ballyhaunis
Oughterard (Claremount)

D Hydrograph width analysis

Introduction to Flood width analysis summary sheets

This appendix summarises the analysis of the widths of observed flood hydrographs. The results of this will be used in the next stage of the study to derive design flood hydrographs.

Information provided in the summary sheets



Flood width analysis summary sheet

Station 32011	Bunowen @ Louisburg Weir

Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	03/12/2006	122.61	11	16/09/2007	78.35
2	0 3/12/2001	110.68	12	01/04/2011	77.67
3	13/08/2008	102.39	13	10/03/2002	75.79
4	01/12/2006	102.00	14	21/04/2004	74.99
5	05/03/2007	94.15	15	10/10/2008	74.99
6	03/02/2008	90.42	16	27/09/2000	74.79
7	22/09/2006	87.02	17	19/11/2006	73.54
8	07/02/2011	85.66	18	21/08/2001	71.85
9	02/12/2000	83.95	19	05/04/2010	71.66
10	13/08/2008	102.39	20	02/02/2004	70.95

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
8.30	7.59	15.66	2.81	0.67

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is very similar to that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 2.81 hours after the peak for the remaining receding limb.

Commentary

Notes on the analysis.

Flood hydrograph plot

The plot shows characteristic flood hydrographs, i.e. hydrographs that are standardised to peak at 1.0 and plotted so that the time origin is at the peak.

The "HWA derived hydrograph" is a mathematical function fitted to a set of median hydrograph widths from a large number of observed floods. HWA is Hydrograph Width Analysis, a computer program developed within work package 3.1 of the FSU research.

The "FSR hydrograph" is derived from the Flood Studies Report rainfall-runoff method, with model parameters estimated solely from catchment descriptors.

In comparing the two hydrographs it is important to be aware that the FSR hydrograph has the potential to be adjusted in order to give a better fit with the shape of observed events. This would be accomplished by estimating the time to peak parameter via a lag analysis.

List of flood events

These are the events from which the HWA hydrograph was derived. The events initially selected for analysis were the highest 20 floods on record. This list was then refined to exclude events with missing data or events with multiple peaks which could not easily be separated, and other events were added to maintain a total of 20. As recommended in FSU WP3.1, some events were trimmed to discard complex areas of multi-peaked hydrographs.

These 20 hydrographs were analysed to calculate their width at a range of percentiles of the peak flow. The median width was then calculated at each percentile, thus producing a derived hydrograph shape.

Parameters of the fitted hydrograph

This table lists the parameters of the mathematical function fitted to the derived flood hydrograph. Use of a parametric approach is recommended in FSU WP3.1 for studies with multiple flow estimation points such as CFRAMS. The parameters are:

n: Shape parameter of gamma function

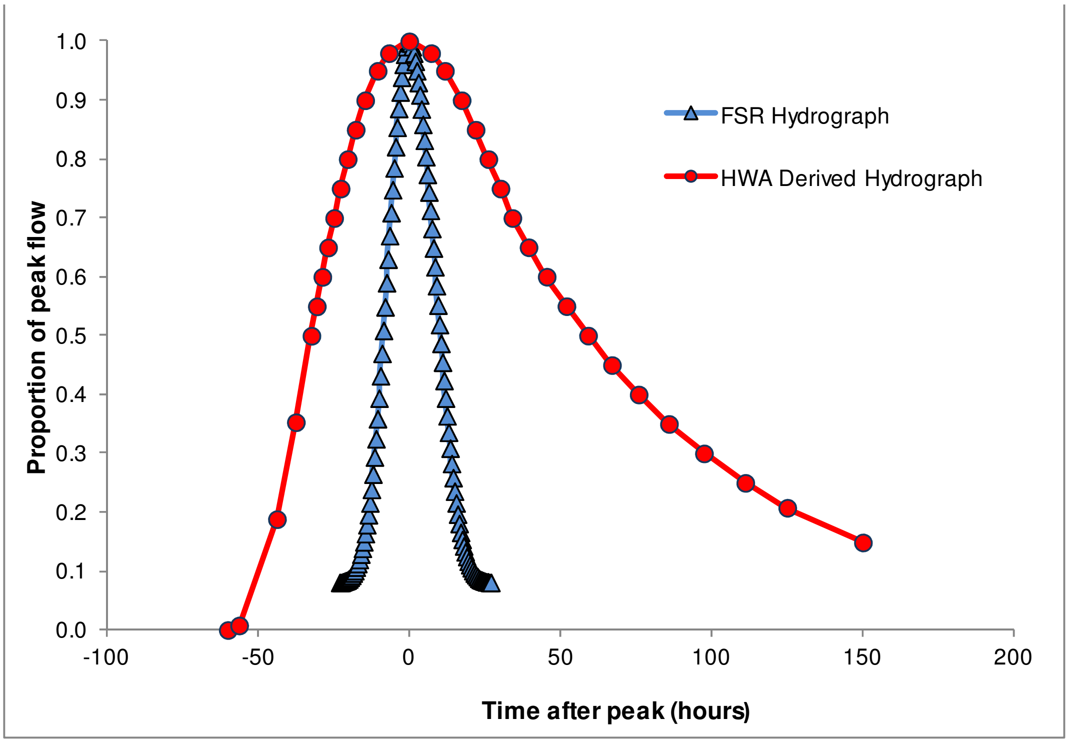
Tr: Translation (location) parameter of gamma function

C: Parameter of the exponential function which is used to describe the recession part of the flood hydrograph

X_0,Y_0: Co-ordinates for the transition between the gamma and exponential functions. X_0 is the time after the peak (in hours) and Y_0 is the normalised flow at this time.

Flood width analysis summary sheet

Station 29001	**Raford @ Rathgorgin (116km²)**



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	20/11/2009	23.25	11	25/12/1968	16.44
2	29/12/2007	20.37	12	09/09/1974	16.38
3	25/08/2009	18.92	13	27/10/2008	16.17
4	08/10/1964	18.75	14	07/02/1990	15.96
5	10/10/1967	18.42	15	10/11/1977	15.8
6	09/12/2007	18.17	16	10/12/1983	15.72
7	02/12/1973	17.11	17	13/12/1964	15.54
8	01/01/2010	16.99	18	23/01/1975	15.17
9	27/11/2009	16.8	19	01/02/2009	14.73
10	03/12/2007	16.61	20	28/12/1978	14.68

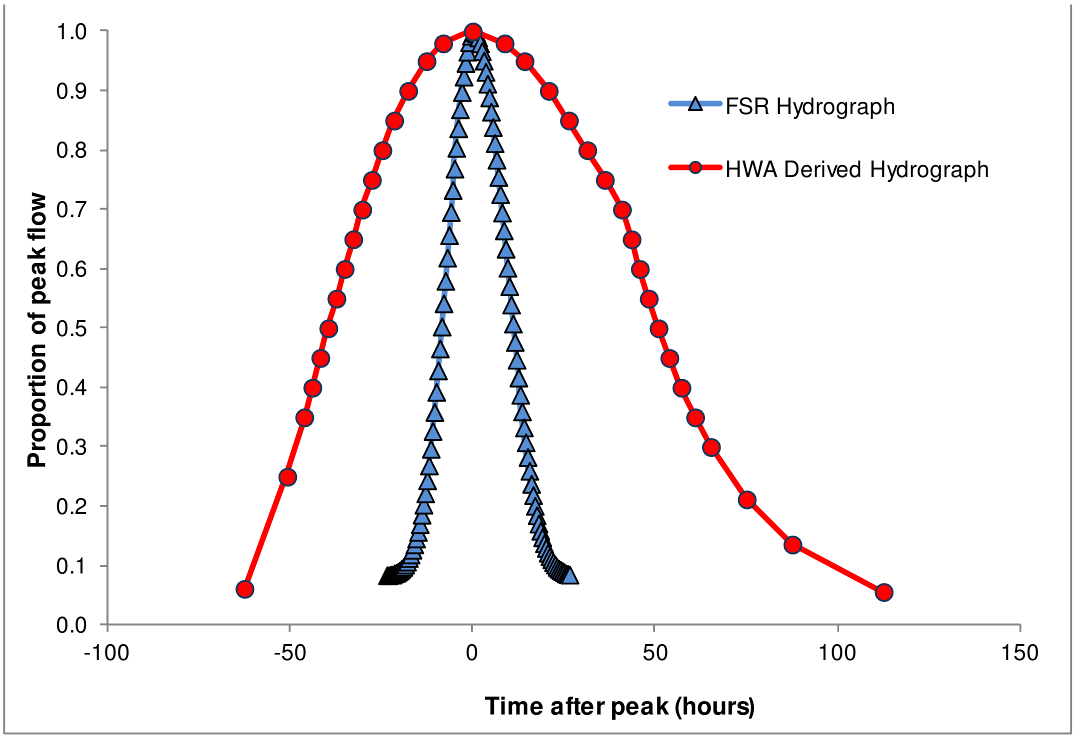
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
4.15	61.58	300.02	34.67	0.69

The 20 largest events on record were sampled at Rathgorgin, with no events removed due to erroneous data or missing periods of record. A number of the sample events were trimmed in order to discard complex areas of multi-peaked hydrographs. The parametric hydrograph produced from the HWA software is significantly wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 34.67 hours after the peak for the remaining receding limb.

Flood width analysis summary sheet

Station 29007	Dunkellin @ Craughwell (272km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	20/11/2009	65.74	11	27/11/2009	31.63
2	09/01/2005	42.70	12	05/02/2002	31.12
3	29/12/2007	41.38	13	01/02/1995	30.12
4	08/02/2011	40.39	14	23/09/1999	29.86
5	29/01/1995	39.04	15	14/12/1994	29.35
6	12/02/2002	34.07	16	10/12/1993	29.28
7	10/12/2007	33.20	17	25/08/2009	28.58
8	29/12/1994	33.20	18	22/01/1995	27.97
9	07/11/2000	32.33	19	29/10/1989	27.78
10	08/02/1990	32.23	20	17/01/2011	27.35

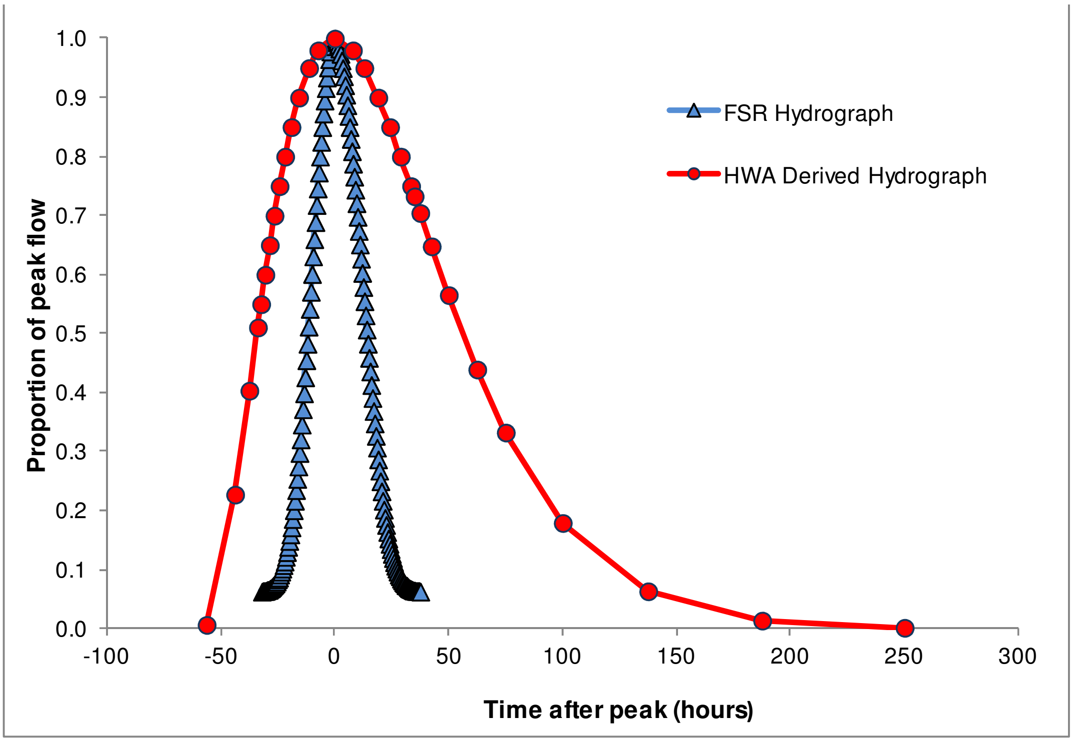
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
4.52	78.50	112.40	41.85	0.69

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is significantly wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 41.85 hours after the peak for the remaining receding limb.

Flood width analysis summary sheet

Station 30007	Clare @ Ballygaddy (470km²)



Flood events used in the analysis

Rank	Date	Flow (m³/s)	Rank	Date	Flow (m³/s)
1	21/11/2009	108.81	11	08/11/1977	69.55
2	30/11/1999	93.38	12	03/12/1992	67.97
3	30/10/1989	92.08	13	27/10/1995	67.34
4	07/02/1990	89.41	14	20/12/1982	66.66
5	05/12/2006	85.11	15	2/01/1991	66.53
6	03/11/1980	80.88	16	11/03/2002	66.08
7	09/01/1992	74.98	17	24/12/1990	66.04
8	07/08/1986	71.09	18	22/1/1995	65.98
9	19/03/1991	70.96	19	27/11/1979	65.92
10	27/05/1985	69.74	20	19/01/1988	64.13

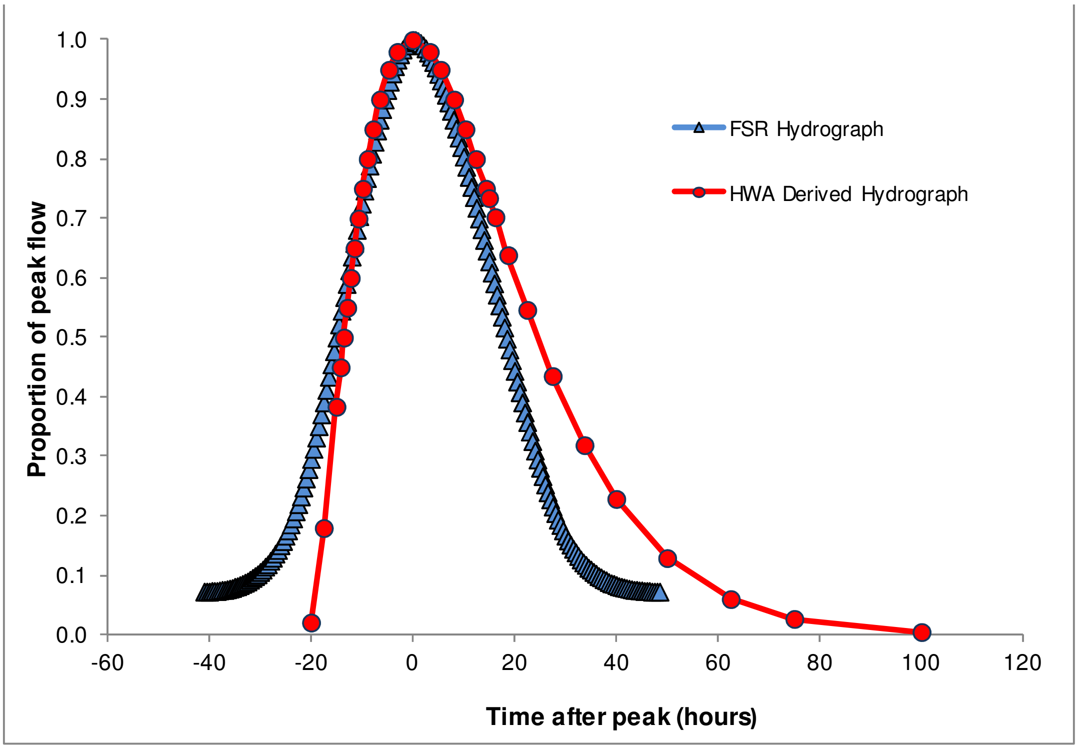
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
3.458	59.25	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The parametric hydrograph produced from the HWA software is significantly wider than that produced by the FSR Rainfall Runoff method. This was produced using only a Gamma curve (unlike some locations where the falling limb is derived using a Recession curve).

Flood width analysis summary sheet

Station 30012	Clare @ Claregalway (1073km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	22/11/2009	165.26	11	27/10/2008	117.37
2	07/12/2006	136.74	12	12/02/2002	115.46
3	08/02/2011	127.52	13	30/12/2007	112.02
4	18/01/2011	127.15	14	11/03/2002	109.60
5	12/10/2008	125.97	15	18/08/2008	107.26
6	10/01/2005	123.56	16	25/08/2009	104.54
7	07/04/2010	121.75	17	12/01/2007	104.00
8	06/02/2002	121.32	18	14/12/2000	103.87
9	10/12/2007	120.81	19	09/11/2010	103.73
10	6/02/2008	118.80	20	22/01/2008	103.06

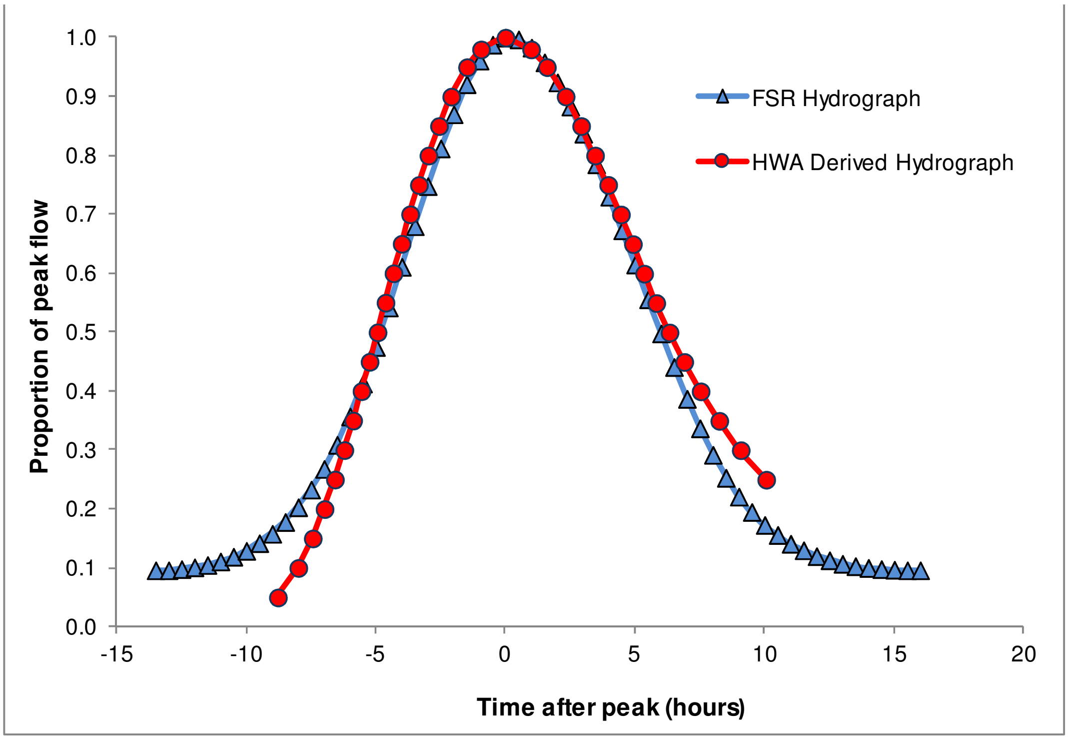
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
2.74	20.88	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The parametric HWA hydrograph is narrower than that produced by the FSR Rainfall Runoff method, with a slightly swifter time to rise, but a longer falling limb. This was produced using only a Gamma curve (unlike some locations where the falling limb is derived using a Recession curve).

Flood width analysis summary sheet

Station 30019	Owenriff @ Claremount (63km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	28/11/1999	204.83	11	14/12/1983	64.16
2	27/10/1989	92.54	12	19/09/1985	64.09
3	27/01/1995	84.78	13	21/10/1998	62.40
4	26/10/1995	76.96	14	12/10/1983	62.39
5	21/10/1988	73.65	15	13/12/1994	61.98
6	18/03/1991	73.37	16	05/12/1986	61.64
7	01/01/1991	67.21	17	07/11/1977	61.04
8	22/12/1991	66.48	18	10/04/1991	59.08
9	10/01/1998	64.86	19	28/11/1996	58.87
10	22/12/1999	64.38	20	31/01/1983	58.07

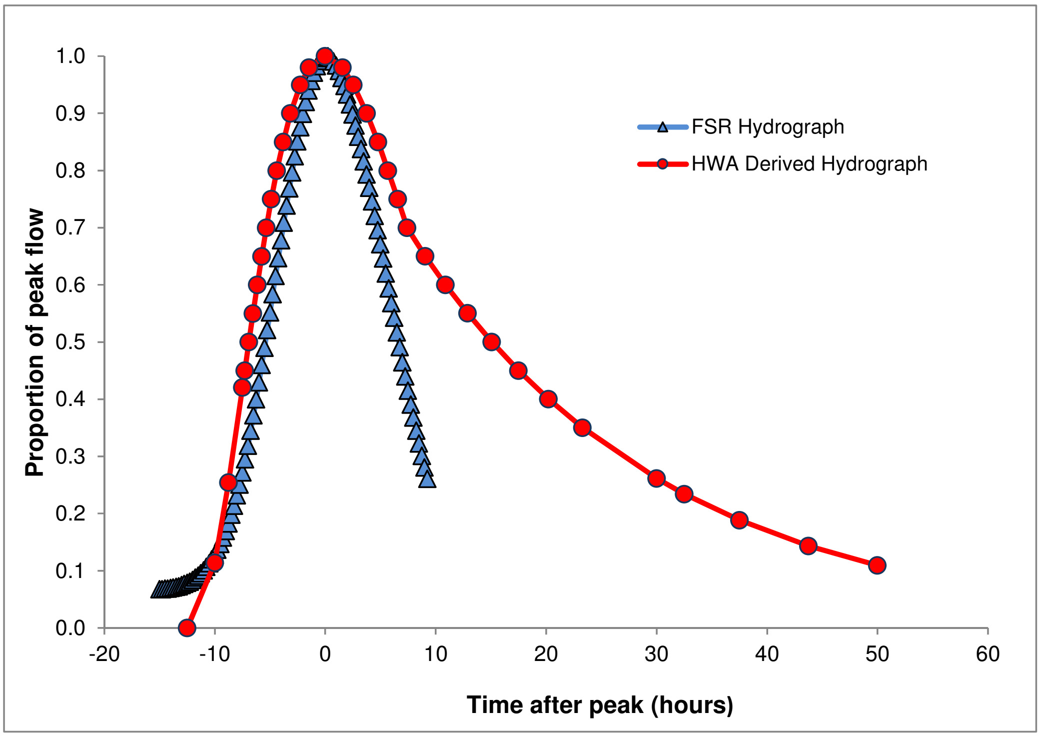
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
9.84	14.29	21.52	4.81	0.66

The 20 largest events on record were sampled at Claremount, with no events removed due to erroneous data or missing periods of record. A number of the sample events were trimmed in order to discard complex areas of multi-peaked hydrographs. The parametric hydrograph produced from the HWA software is very similar to that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 4.81 hours after the peak for the remaining receding limb.

Flood width analysis summary sheet

Station 30020	**Clare @ Ballyhaunis (21km²)**



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	19/11/2009	7.50	11	21/1/2008	4.04
2	06/09/2010	6.20	12	05/03/2007	3.97
3	03/12/2006	5.05	13	18/11/2009	3.94
4	16/11/2009	5.00	14	07/01/2005	3.92
5	01/02/2004	4.69	15	25/05/2005	3.92
6	08/12/2007	4.65	16	26/10/2006	3.87
7	03/12/2001	4.61	17	23/08/2009	3.84
8	10/03/2002	4.39	18	10/10/2008	3.79
9	07/02/2011	4.20	19	21/11/2009	3.66
10	03/02/2008	4.07	20	06/04/2010	3.52

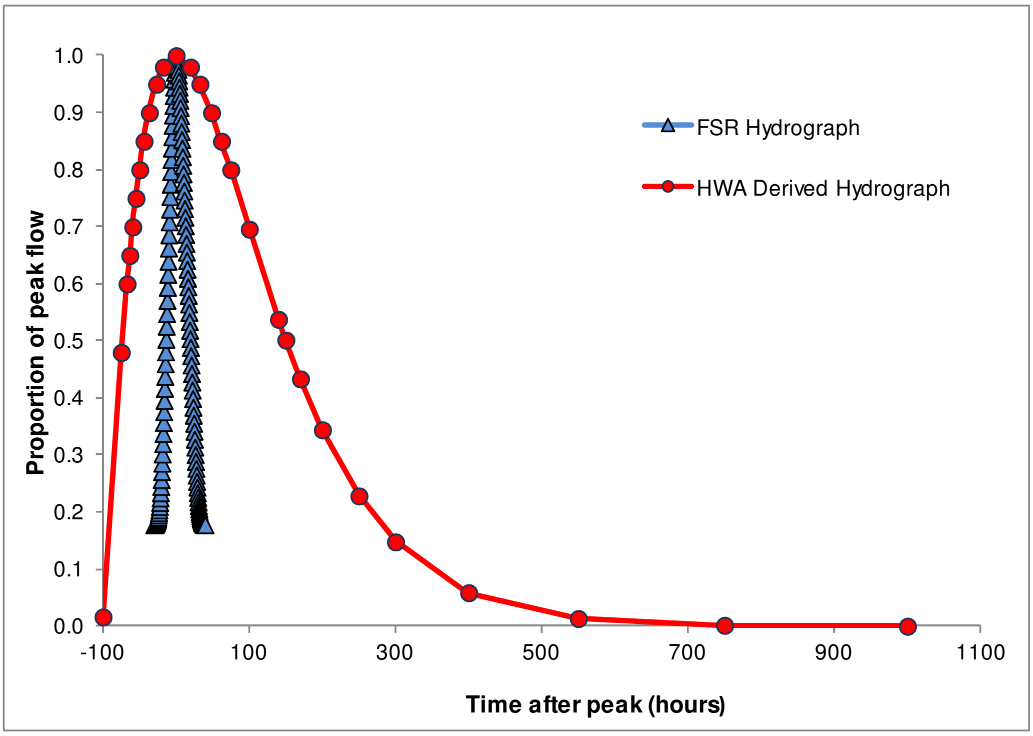
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
3.88	12.74	91.88	7.50	0.70

The 20 largest events on record were sampled at Ballyhaunis, with no events removed due to erroneous data or missing periods of record. Some events were trimmed in order to discard complex areas of multi-peaked hydrographs. The parametric hydrograph produced from the HWA software is narrower than that produced by the FSR Rainfall Runoff method, with a swifter time to rise, but a longer falling limb. This was produced using a Gamma curve for the rising and initial receding limbs, switching to a recession curve 7.50 hours after the peak for the remaining receding limb.

Flood width analysis summary sheet

Station 30061	Corrib @ Galway (Wolfe Tone Bridge) ($3136 km^2$)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	25/01/1975	441.05	11	12/04/1991	284.42
2	29/12/1974	337.33	12	17/01/1984	284.11
3	05/01/1991	332.12	13	07/02/1992	283.24
4	27/02/1990	321.91	14	09/02/1988	282.28
5	09/12/1954	299.33	15	01/02/1995	281.64
6	07/01/1975	297.81	16	09/03/1993	276.83
7	12/11/1977	289.82	17	06/01/1994	275.26
8	18/02/1980	286.87	18	24/01/1993	274.48
9	06/02/1980	286.11	19	01/01/1960	273.56
10	05/11/1989	285.75	20	20/12/1954	272.92

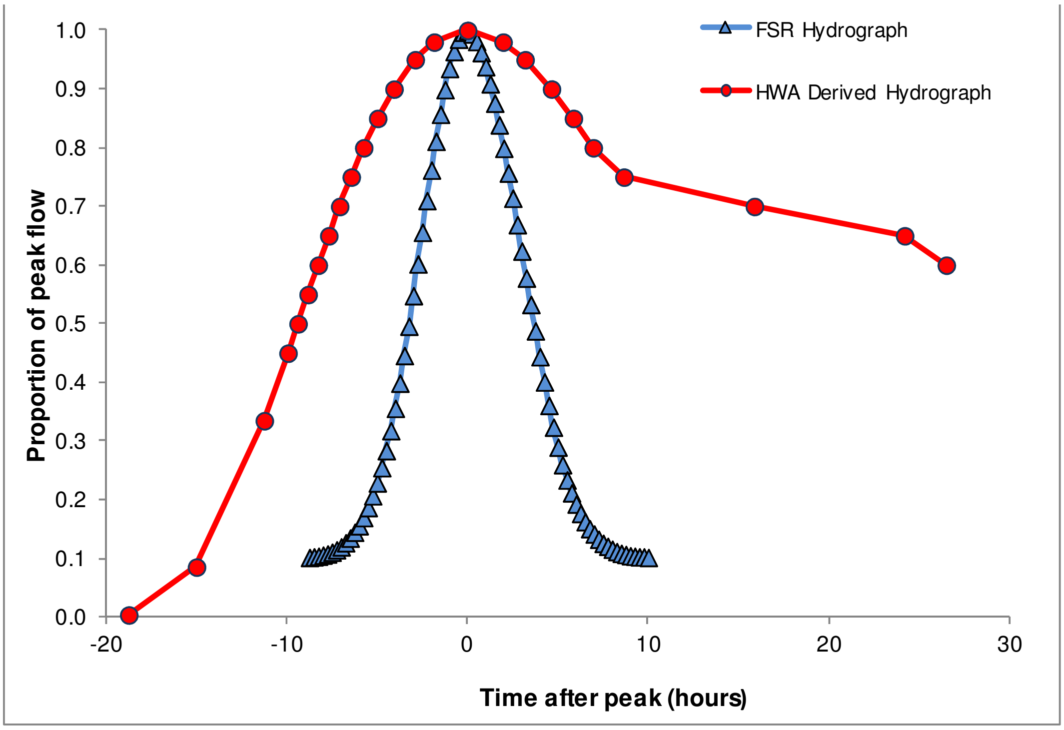
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
2.20	101	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The parametric HWA hydrograph is significantly wider than that produced by the FSR Rainfall Runoff method, with a much longer falling limb. The extreme difference in widths is unsurprising as the FSR method does not account for the presence of lakes in the catchment.

Flood width analysis summary sheet

Station 32012	Newport @ Newport Weir (146km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	03/12/2006	37.60	11	03/07/2009	31.76
2	07/02/2011	36.30	12	08/11/2010	30.58
3	08/12/2007	36.04	13	05/12/2001	30.12
4	04/11/2010	35.72	14	22/12/2004	29.71
5	13/08/2008	35.66	15	11/12/2006	29.60
6	10/10/2008	35.34	16	27/10/2000	29.37
7	15/01/2005	34.02	17	19/02/2002	29.08
8	21/01/2008	33.34	18	08/09/2010	28.62
9	27/10/2002	32.30	19	08/01/2007	28.35
10	20/01/2005	31.88	20	24/02/2002	28.24

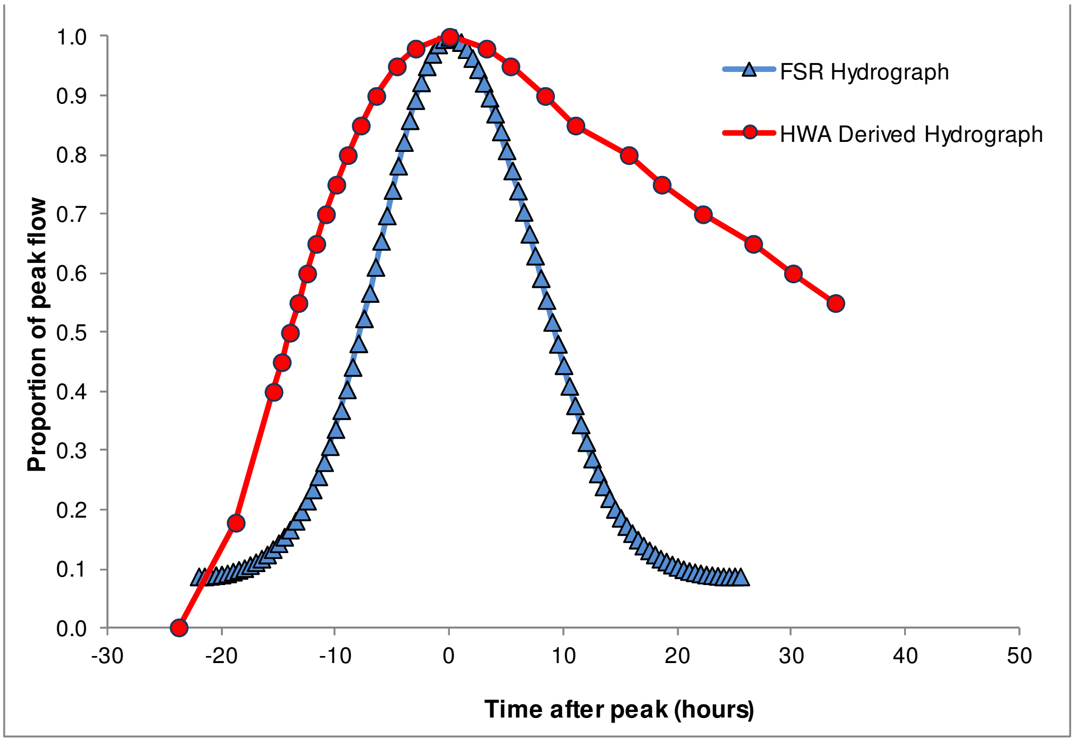
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
6.52	22.28	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to the non parametric hydrograph (as both the Gamma and Recession curves offered a poor fit).

Flood width analysis summary sheet

Station 32006	**Carrowbeg @ Coolloughra (36km²)**



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	19/11/2009	15.54	11	06/03/2007	9.35
2	03/12/2006	13.53	12	30/11/2006	9.26
3	24/11/2009	12.52	13	20/11/2006	9.07
4	16/08/2008	11.89	14	01/11/2009	8.59
5	14/12/2006	10.45	15	04/11/2009	8.50
6	22/11/2009	10.20	16	02/12/2007	8.20
7	09/12/2007	10.12	17	18/01/2009	7.82
8	25/10/2008	10.10	18	07/11/2009	7.76
9	14/08/2008	10.07	19	23/10/2008	7.66
10	03/02/2008	9.86	20	03/11/2005	7.65

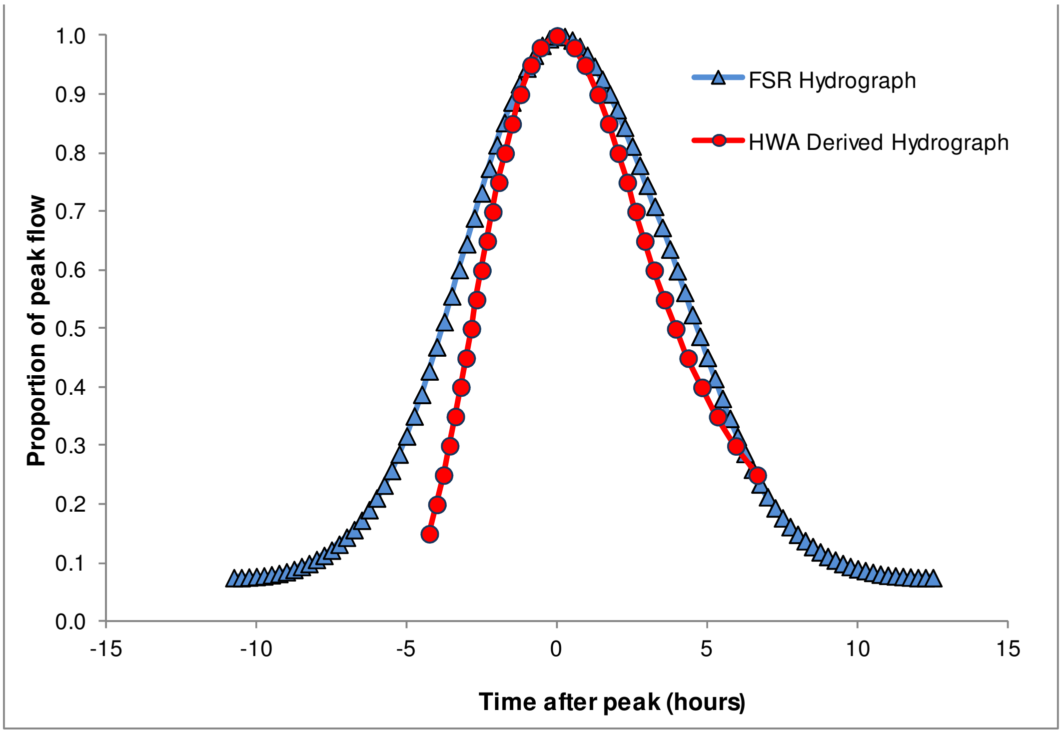
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
3.52	98.02	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to the non parametric hydrograph (as both the Gamma and Recession curves offered a poor fit).

Flood width analysis summary sheet

Station 32011	Bunowen @ Louisburg Weir (70km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	03/12/2006	122.61	11	16/09/2007	78.35
2	0 3/12/2001	110.68	12	01/04/2011	77.67
3	13/08/2008	102.39	13	10/03/2002	75.79
4	01/12/2006	102.00	14	21/04/2004	74.99
5	05/03/2007	94.15	15	10/10/2008	74.99
6	03/02/2008	90.42	16	27/09/2000	74.79
7	22/09/2006	87.02	17	19/11/2006	73.54
8	07/02/2011	85.66	18	21/08/2001	71.85
9	02/12/2000	83.95	19	05/04/2010	71.66
10	13/08/2008	102.39	20	02/02/2004	70.95

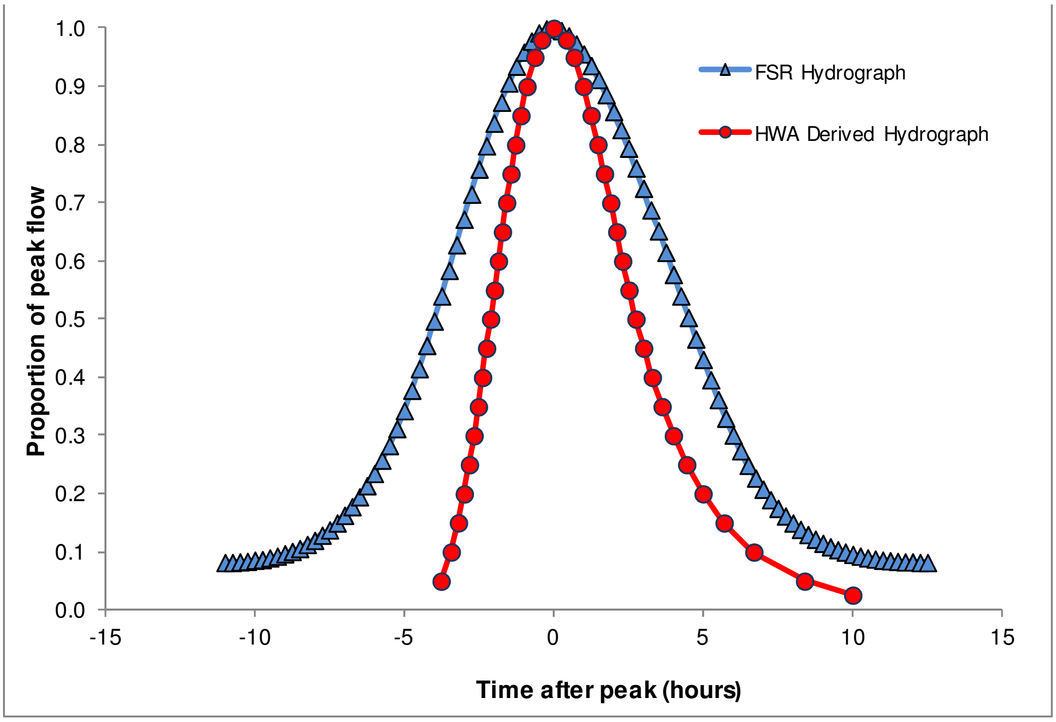
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
8.30	7.59	15.66	2.81	0.67

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is very similar to that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 2.81 hours after the peak for the remaining receding limb.

Flood width analysis summary sheet

Station 32004	**Owenglin @ Clifden (32km^2)**



Flood events used in the analysis

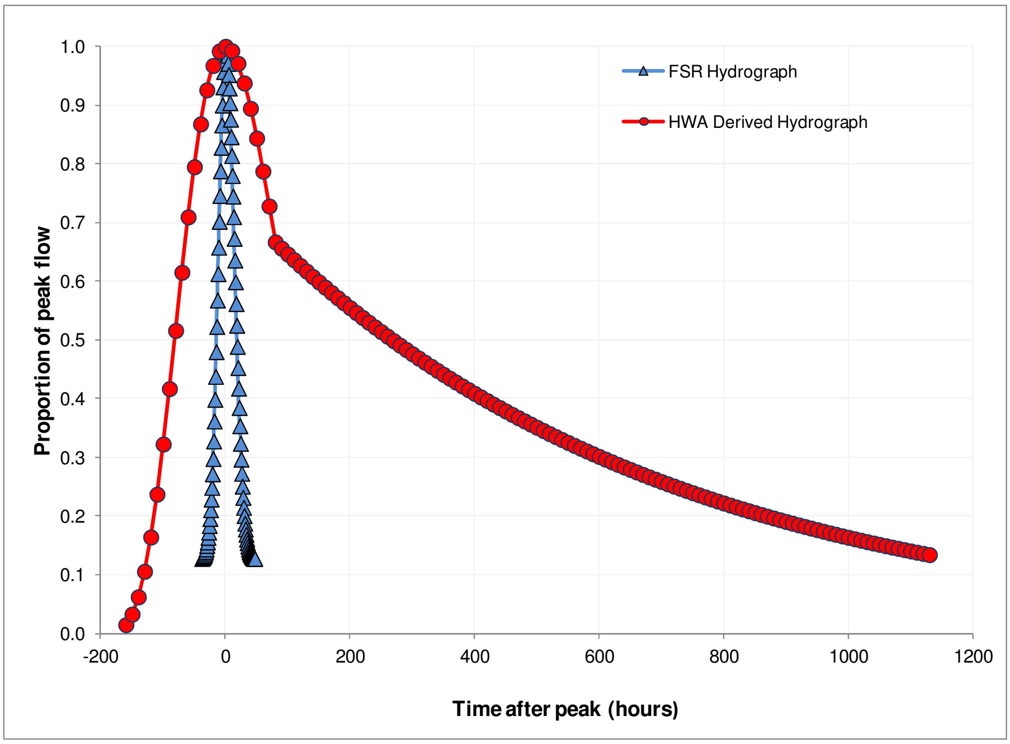
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	21/9/2006	56.00	11	10/10/2008	36.00
2	16/8/2008	49.20	12	26/2/2007	35.70
3	13/8/2008	47.10	13	10/12/2004	35.50
4	19/8/2009	44.90	14	7/9/2010	33.60
5	5/10/2006	41.60	15	21/5/2003	33.50
6	25/5/2005	40.50	16	11/11/2010	33.20
7	3/12/2006	38.60	17	22/6/2008	33.10
8	4/11/2010	38.00	18	30/11/2006	32.70
9	23/8/2004	37.90	19	20/6/2007	32.50
10	23/8/2009	36.20	20	18/1/2007	32.30

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
10.00	6.17	9.80	2.06	0.66

No events were removed due to erroneous data or missing periods of record. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is narrower than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 2.06 hours after the peak for the remaining receding limb.

Station 34001	Moy @ Rahans (1974km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	30/ 10/ 1989	288.34	6	10/ 1/ 1998	214.7
2	26/ 11/ 2009	230.71	7	3/ 11/ 1980	207.4
3	7/ 2/ 1990	228.37	8	2/ 2/ 1984	197.88
4	6/ 1/ 1991	223.18	9	24/ 1/ 1993	191.34
5	15/ 12/ 2006	215.96	10	10/ 12/ 2007	188.9

Parameters of the hydrograph

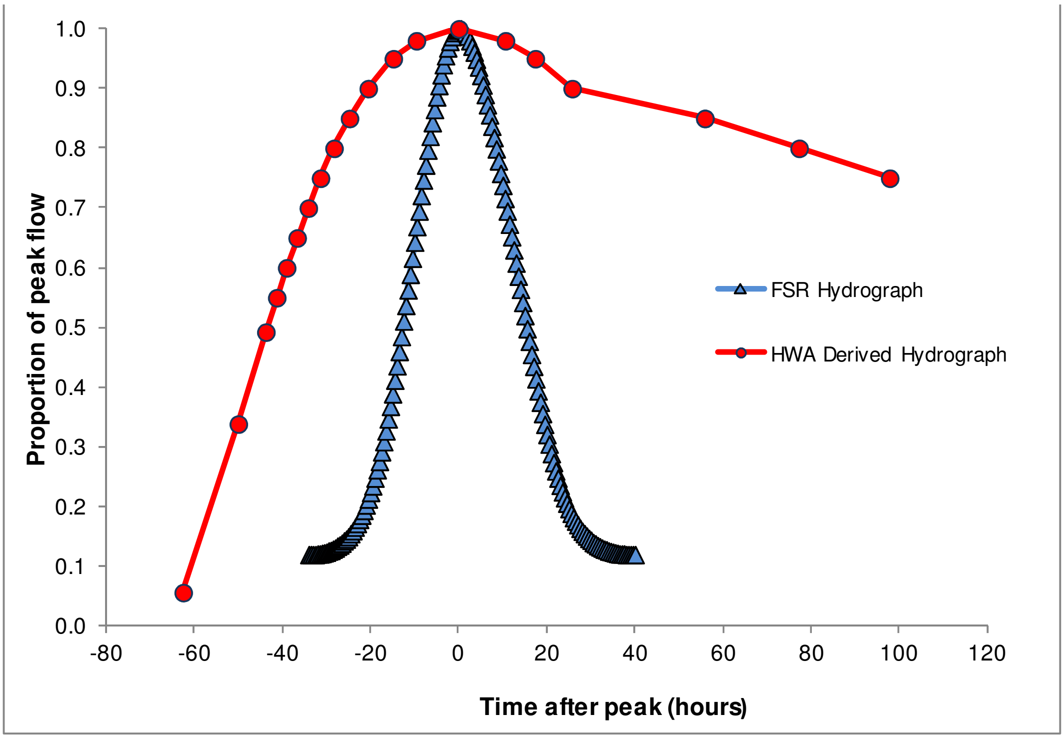
n	Tr (hours)	C	Xo	Yo
8.89	225	656	80.1	0.67

Analysis at this station was added after completion of the inception phase, when it became apparent that design hydrograph shapes for input to the HPW model at Ballina would be best represented using observed hydrograph shapes.

Hydrographs on the River Moy are extremely prolonged, with the river staying high typically for months during a winter flood. The time window for analysis using HWA was set to 600 hours before the peak and 1600 hours after the peak, i.e. a total time span of 3 months. Many events were excluded because there was no clearly defined hydrograph. Ten events were included in the final analysis.

HWA results for this station are included in the report on FSU WP 3.4. The derived median hydrograph is very different to that shown above. It appears that the analysis in WP 3.4 did not identify more than the top portion of most hydrographs, probably due to specifying a too-narrow time window. The results above are more convincing, giving a median hydrograph very much wider than that from the FSR rainfall-runoff method (applied using catchment descriptors), which does not account for the presence of large volumes of storage in the upstream catchment, in particular Loughs Conn and Cullin.

Station 34003	Moy @ Foxford (1805km^2)



Flood events used in the analysis

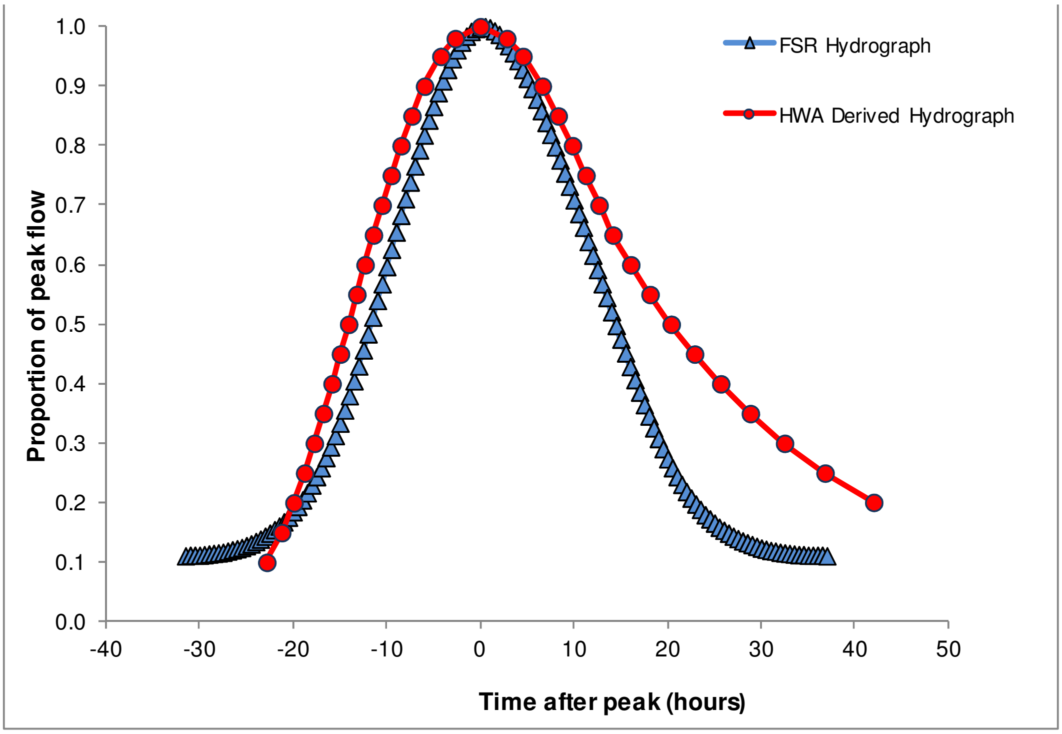
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	24/11/2009	259.23	11	17/11/2009	179.40
2	14/12/2006	243.00	12	11/01/2007	174.64
3	20/11/2009	231.33	13	07/02/2011	174.64
4	11/12/2006	223.43	14	16/01/2005	171.83
5	10/12/2007	195.01	15	07/12/2009	171.83
6	02/12/2009	189.89	16	18/01/2007	171.69
7	04/12/2006	189.59	17	20/02/2002	168.91
8	10/01/2005	184.08	18	13/12/2000	163.13
9	11/02/2002	182.41	19	04/12/2000	159.00
10	21/01/2005	179.40	20	21/01/2008	155.35

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
2.85	68.27	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to the non parametric hydrograph (as both the Gamma and Recession curves offered a poor fit).

Station 34004	Moy @ Ballylahan (935km^2)



Flood events used in the analysis

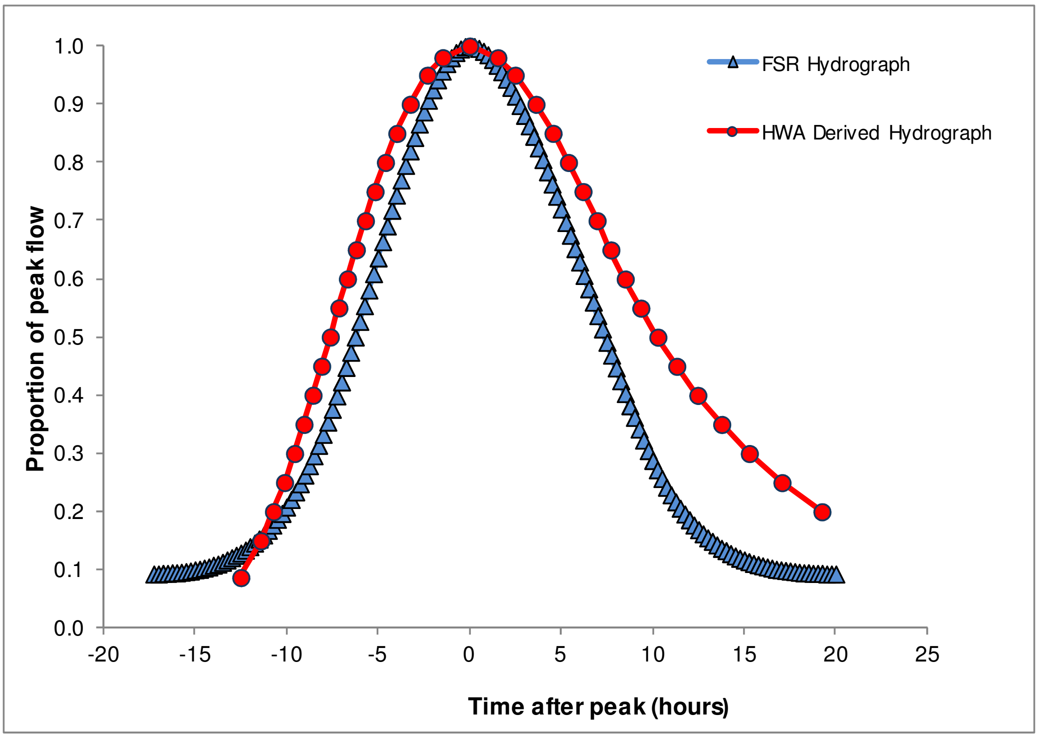
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	28/10/1989	374.50	11	27/05/1985	252.59
2	02/11/1980	331.08	12	15/01/1975	248.95
3	10/01/1998	308.04	13	21/10/1998	246.90
4	28/11/1999	299.89	14	14/12/1983	243.97
5	26/11/1979	291.78	15	21/12/1985	243.83
6	15/11/1978	283.29	16	19/12/1982	241.08
7	05/12/1986	278.59	17	08/01/2005	239.41
8	14/08/2008	263.9	18	26/10/1995	233.18
9	05/11/1999	258.67	19	21/09/1985	231.54
10	06/08/1986	253.87	20	08/01/1992	230.95

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
10.00	41.05	94.81	13.68	0.66

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is similar to that produced by the FSR Rainfall Runoff method, although the receding limb is a little longer. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 13.68 hours after the peak.

Station 34007	Deel @ Ballycarroon (152km²)



Flood events used in the analysis

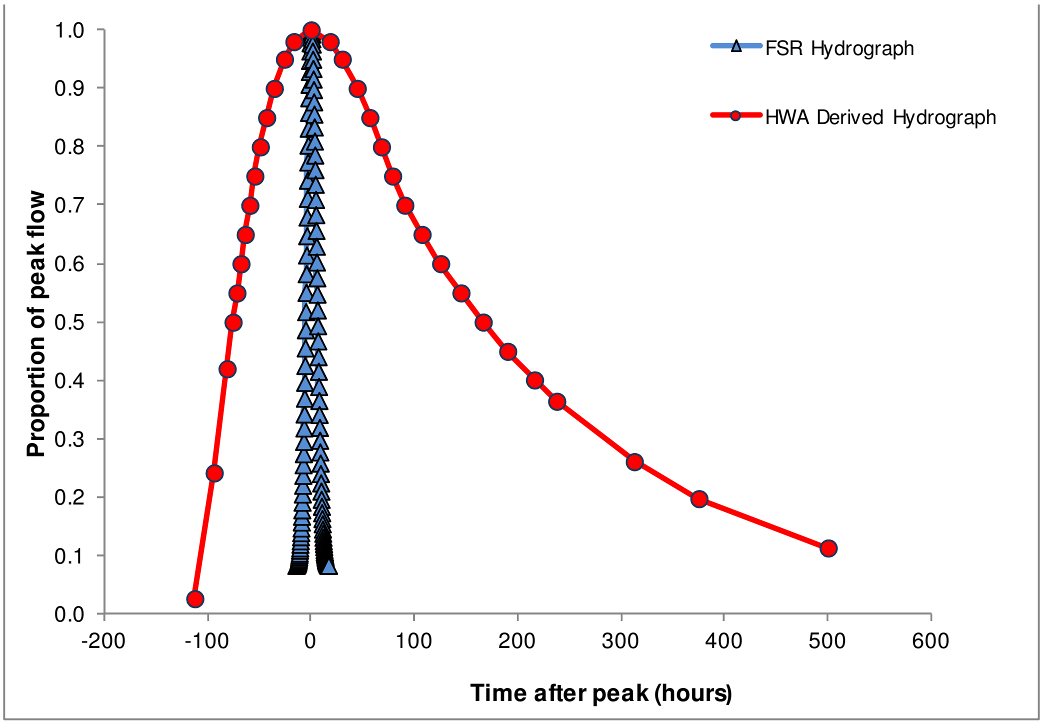
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	28/10/1989	159.84	11	02/11/1980	104.11
2	27/11/1979	144.07	12	21/10/1998	97.46
3	01/10/1985	143.29	13	19/12/1982	97.04
4	03/12/2006	133.93	14	03/12/2001	96.28
5	05/12/1986	132.91	15	14/01/1988	95.70
6	07/09/1980	122.90	16	01/11/1986	95.39
7	15/11/1978	118.32	17	27/10/2002	91.72
8	28/09/1978	116.42	18	06/08/1986	89.90
9	11/09/1992	108.61	19	16/11/1986	89.54
10	01/01/1998	105.93	20	18/10/1984	89.35

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
8.89	20.98	39.11	7.47	0.67

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is similar to that produced by the FSR Rainfall Runoff method, although the receding limb is a little longer. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 7.47 hours after the peak.

Station 34018	Castlebar @ Turlough (95km^2)



Flood events used in the analysis

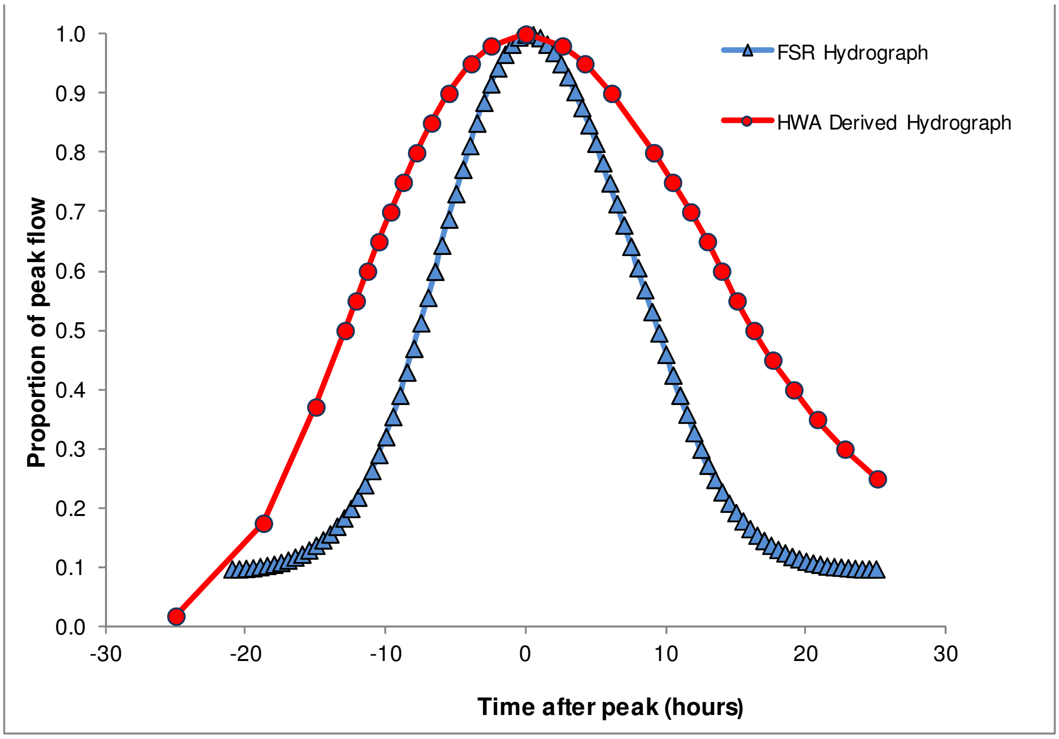
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	23/11/2009	19.37	11	05/02/1990	13.77
2	09/12/2007	18.01	12	05/12/2000	13.43
3	30/10/1989	16.37	13	08/02/2011	13.36
4	23/12/1999	15.14	14	29/10/2002	12.63
5	05/01/1991	14.85	15	11/12/1999	12.35
6	20/01/2005	14.50	16	28/01/1995	12.30
7	02/01/1999	14.47	17	10/02/2002	12.25
8	08/11/2010	14.29	18	24/01/2008	12.13
9	28/11/1999	14.14	19	24/11/1986	11.96
10	10/01/1998	14.10	20	01/12/1984	11.90

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
2.88	119.25	900.99	87.09	0.71

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is very much wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 87.09 hours after the peak.

Station 34029	Deel @ Knockadangan (227km²)



Flood events used in the analysis

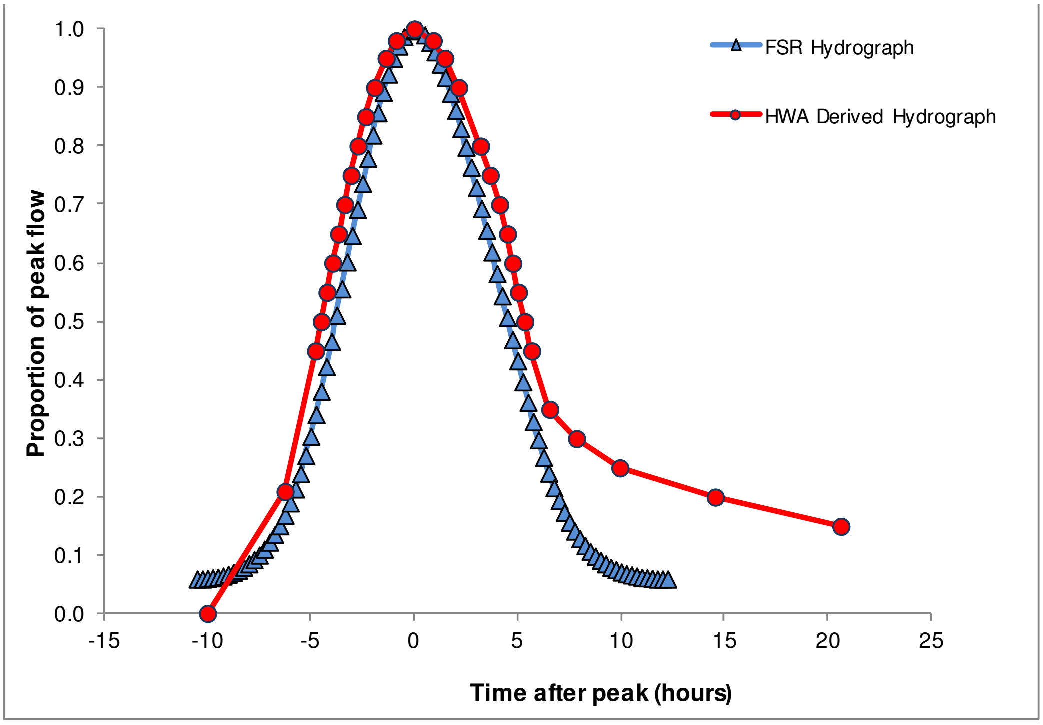
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	3/12/2006	151.48	11	07/02/2011	79.41
2	27/10/2002	111.11	12	20/11/2006	78.05
3	06/03/2007	106.83	13	21/10/2002	73.85
4	04/12/2001	102.39	14	14/08/2008	70.69
5	14/12/2006	97.03	15	05/04/2010	69.57
6	08/09/2010	91.75	16	31/01/2004	67.56
7	08/11/2010	90.45	17	16/08/2008	65.72
8	30/11/2006	83.94	18	09/01/2007	63.99
9	20/02/2002	82.96	19	04/11/2010	63.67
10	18/11/2009	79.65	20	10/10/2008	63.08

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
9.03	35.87	50.84	12.66	0.67

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The parametric HWA hydrograph is similar, but a little wider than that produced by the FSR Rainfall Runoff method, with a slower time to rise and a longer falling limb. This was produced using a Gamma curve for the rising and initial receding limbs, switching to a recession curve 12.67 hours after the peak.

Station 34031	Mullaghanoe @ Charlestown (23km^2)



Flood events used in the analysis

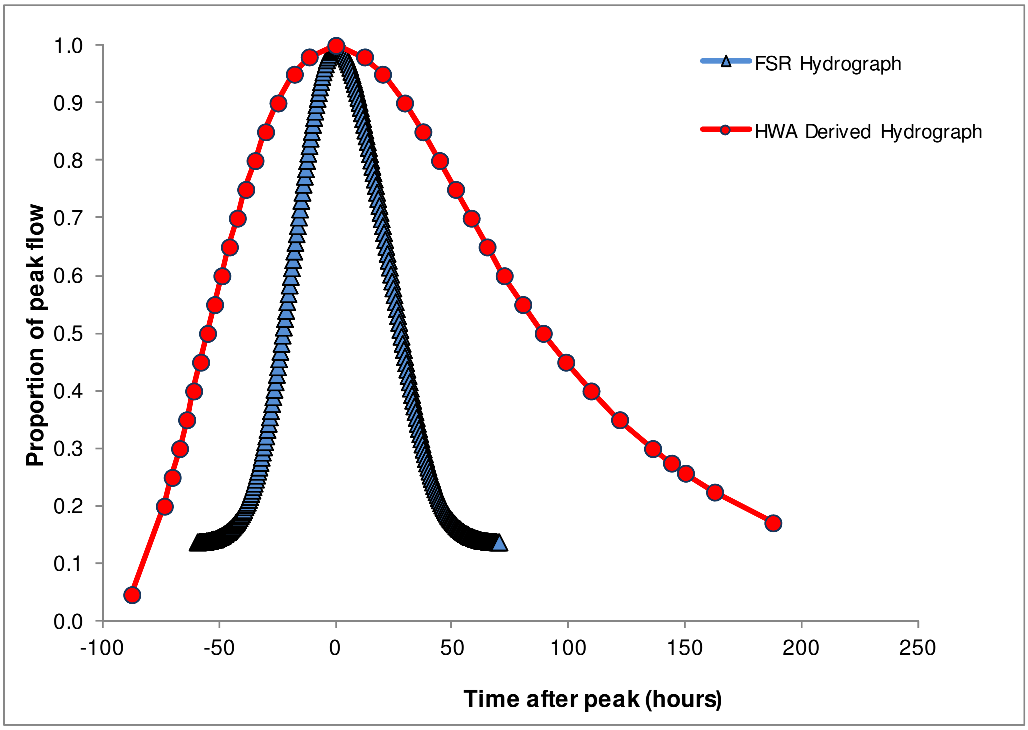
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	25/01/2009	10.80	11	8/11/2002	8.47
2	08/12/2007	10.30	12	09/02/2002	8.45
3	02/11/2002	9.74	13	25/05/2005	7.84
4	13/08/2008	9.68	14	19/02/2002	7.72
5	05/03/2007	9.53	15	21/09/2006	7.63
6	21/11/2009	9.33	16	12/12/2000	7.47
7	07/09/2010	8.71	17	27/10/2002	7.45
8	05/10/2001	8.64	18	08/11/2010	7.27
9	27/02/2000	8.59	19	10/11/2002	7.25
10	21/01/2008	8.54	20	10/10/2008	7.08

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
8.65	12.23	12.73	4.42	0.67

Many events at Charlestown were discounted due to periods of no data; this was often found during the higher events, therefore it is assumed this was due to logger failure. Extra, lower magnitude events have replaced these. The parametric HWA hydrograph is very similar to that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs, switching to a recession curve 4.42 hours after the peak. The latter receding limb is the non parametric HWA curve, given the poor fit of the recession curve after 6.5 hours.

Station 35001	Owenmore @ Ballynacarrow (300km^2)



Flood events used in the analysis

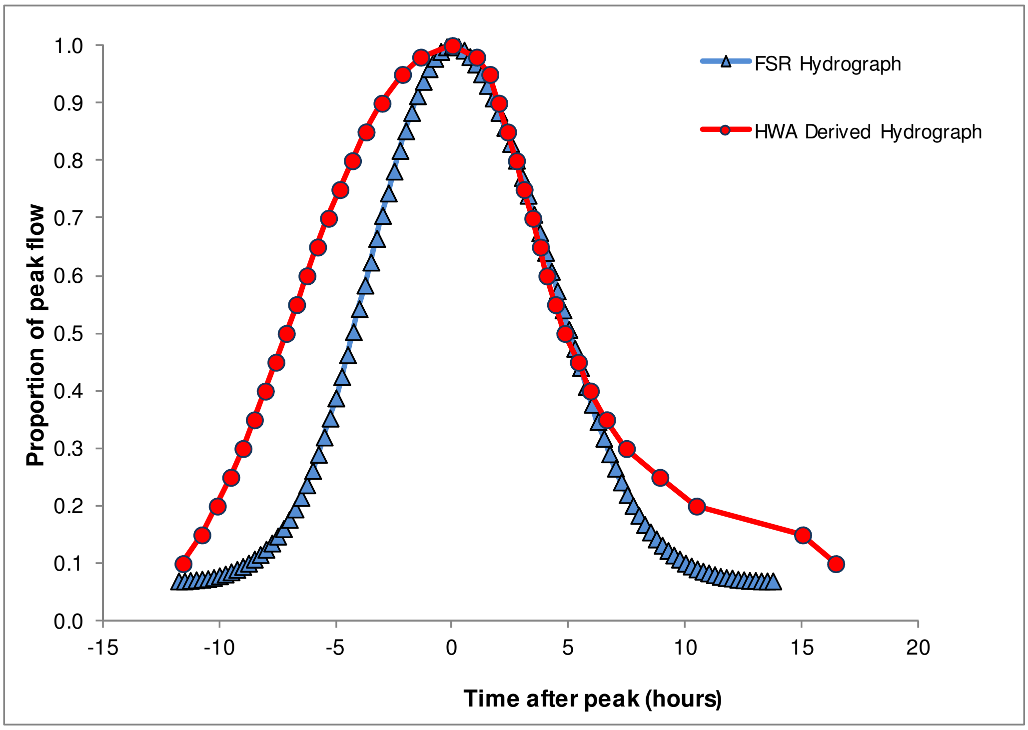
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	04/11/1968	48.78	11	20/09/1965	35.16
2	30/10/1989	44.83	12	28/11/1979	35.11
3	08/02/1990	39.23	13	11/10/1967	34.78
4	10/01/1992	38.88	14	11/03/1995	33.92
5	29/05/1985	38.58	15	03/01/1957	33.91
6	24/10/1967	38.04	16	18/10/1964	33.44
7	04/11/1980	36.60	17	23/11/1971	33.20
8	20/11/1965	36.46	18	30/09/1981	33.07
9	18/11/1978	36.32	19	09/10/1965	32.68
10	20/01/1965	35.59	20	17/11/1959	31.90

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
4.14	104.50	367.45	59.01	0.693

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is significantly wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 59.01 hours after the peak for the remaining receding limb.

Station 35002	Owenbeg @ Billa Bridge (89km^2)



Flood events used in the analysis

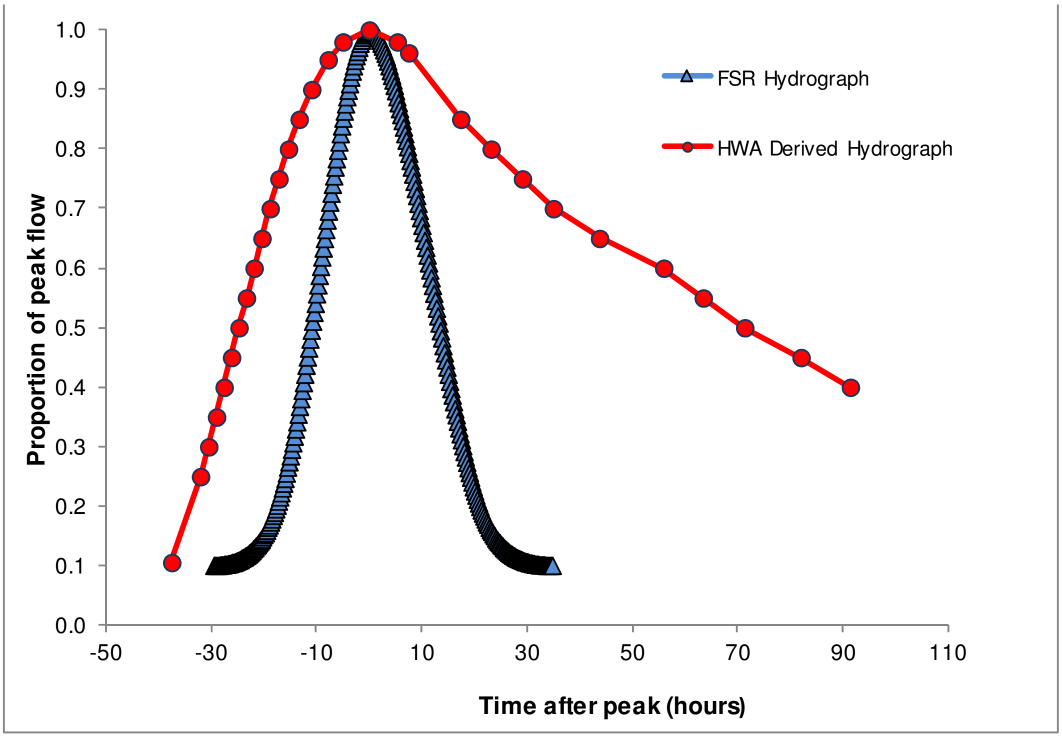
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	28/10/1989	69.30	11	20/08/1987	58.84
2	27/10/2002	66.85	12	04/11/1999	58.43
3	06/10/1990	66.85	13	06/08/1986	58.35
4	29/10/1989	62.46	14	16/11/2009	57.35
5	28/11/1999	61.64	15	03/11/2002	57.03
6	02/09/1988	61.24	16	24/10/1998	56.82
7	26/11/1979	60.55	17	12/10/1978	56.74
8	01/01/1991	59.44	18	11/02/1998	56.69
9	21/10/1998	59.14	19	21/09/1985	56.36
10	15/11/1978	59.09	20	28/11/1973	55.98

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
10.00	20.80	n/a	n/a	n/a

The 20 largest events on record were sampled with no events removed. A number of the sample events were trimmed in order to discard complex areas of multi-peaked hydrographs. The final HWA hydrograph has a similar width to that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising limb. The receding limb is the non parametric HWA curve, given the poor fit of the recession and gamma curves after the peak.

Station 35005	Ballysadare @ Ballysadare (640km^2)



Flood events used in the analysis

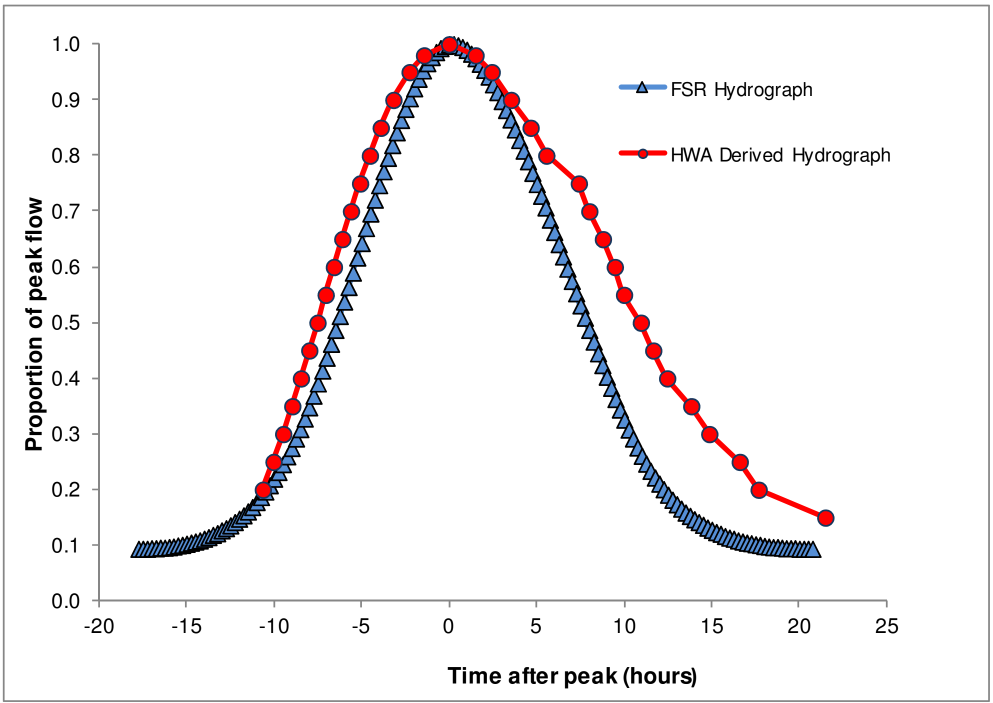
Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	20/11/2009	142.42	11	29/11/1999	98.36
2	29/10/1989	131.12	12	09/01/1992	98.18
3	02/11/1968	126.39	13	10/12/1999	94.24
4	27/10/2002	114.97	14	08/01/2005	92.88
5	26/11/1979	114.09	15	10/01/1965	92.45
6	09/01/1968	112.33	16	14/12/2006	91.05
7	19/10/1954	111.64	17	11/03/1995	88.88
8	09/12/2007	105.13	18	03/02/2004	86.55
9	10/01/1998	103.26	19	2/11/1980	85.72
10	01/03/1955	102.99	20	28/11/1954	85.31

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
5.21	52.63	n/a	n/a	n/a

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events and some events were trimmed to discard complex areas of multi-peaked hydrographs. The parametric HWA hydrograph is wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to the non parametric HWA curve, given the poor fit of the recession curve after 25 hours.

Station 35011	Bonet @ Dromahair (293km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	22/10/1987	187.79	11	27/10/2002	146.82
2	28/11/1999	176.38	12	08/11/2002	142.37
3	02/09/1988	167.82	13	02/03/2000	141.70
4	22/12/1991	161.62	14	18/11/1965	138.38
5	06/08/1986	159.51	15	21/10/1998	138.34
6	05/12/1986	157.44	16	10/03/1995	136.86
7	28/10/1989	152.23	17	27/02/2000	133.27
8	08/01/1992	150.83	18	26/10/1995	132.65
9	06/10/1990	148.50	19	03/12/1999	131.80
10	26/01/1993	147.02	20	22/11/1998	130.87

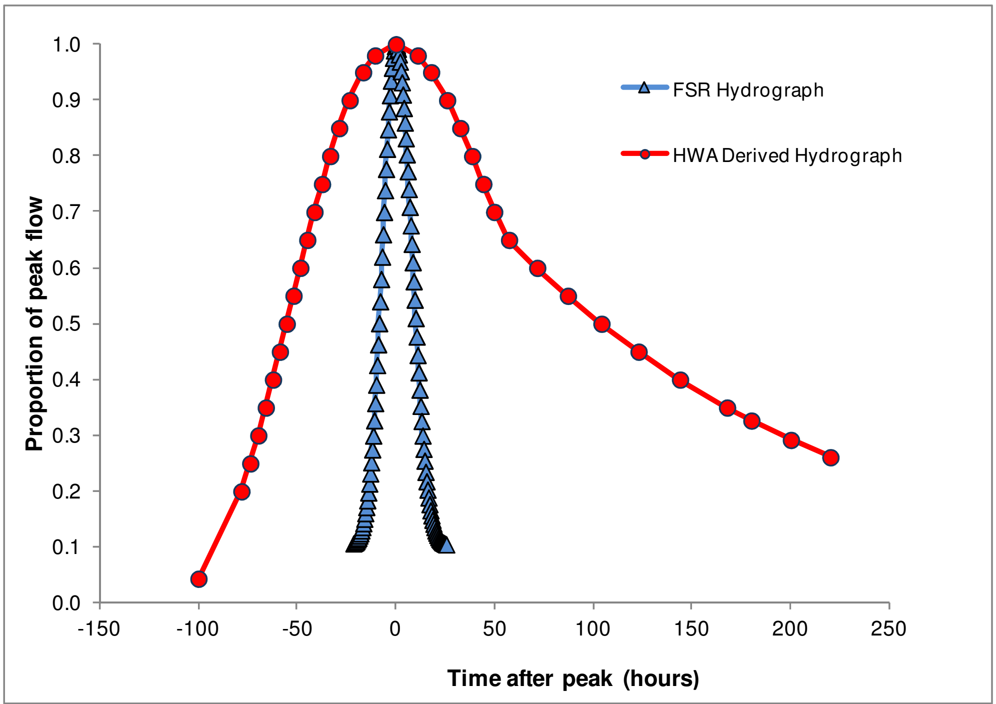
Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
9.98	21.92	n/a	n/a	n/a

One event was discounted due to irregularities in the data. This was replaced with another event and some events were trimmed to discard complex areas of multi-peaked hydrographs. The parametric HWA hydrograph is very similar to that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to the non parametric HWA curve, given the poor fit of the recession curve after 4.5 hours.

Flood width analysis summary sheet

Station 35012	Sligo River @ New Bridge (369km^2)



Flood events used in the analysis

Rank	Date	Flow (m^3/s)	Rank	Date	Flow (m^3/s)
1	19/11/2009	184.11	11	25/02/2002	145.35
2	19/10/2011	182.47	12	23/01/2008	139.95
3	07/11/2009	172.74	13	08/02/2011	138.11
4	09/12/2007	167.97	14	17/08/2008	129.46
5	10/11/2002	166.85	15	05/11/2010	126.31
6	21/01/2005	166.62	16	23/09/2004	125.34
7	09/01/2007	159.08	17	01/02/2009	123.01
8	28/10/2002	156.68	18	06/05/2004	122.43
9	09/01/2005	154.29	19	22/05/2003	119.75
10	02/02/2004	146.19	20	27/05/2002	118.99

Parameters of the hydrograph

n	Tr (hours)	C	Xo	Yo
10.00	161.23	178.34	53.74	0.66

Analysis at this station was added after completion of the inception phase, when it became apparent that design hydrograph shapes for input to the HPW model at Sligo would be best represented using observed hydrograph shapes.

A number of events were discounted due to irregularities in the data or the HWA software sampling a peak which was on the rising or falling limb of a larger event. These have been replaced with other events. Some events were trimmed to discard complex areas of multi-peaked hydrographs. The HWA parametric hydrograph is wider than that produced by the FSR Rainfall Runoff method. This was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 57 hours after the peak for the remaining receding limb.

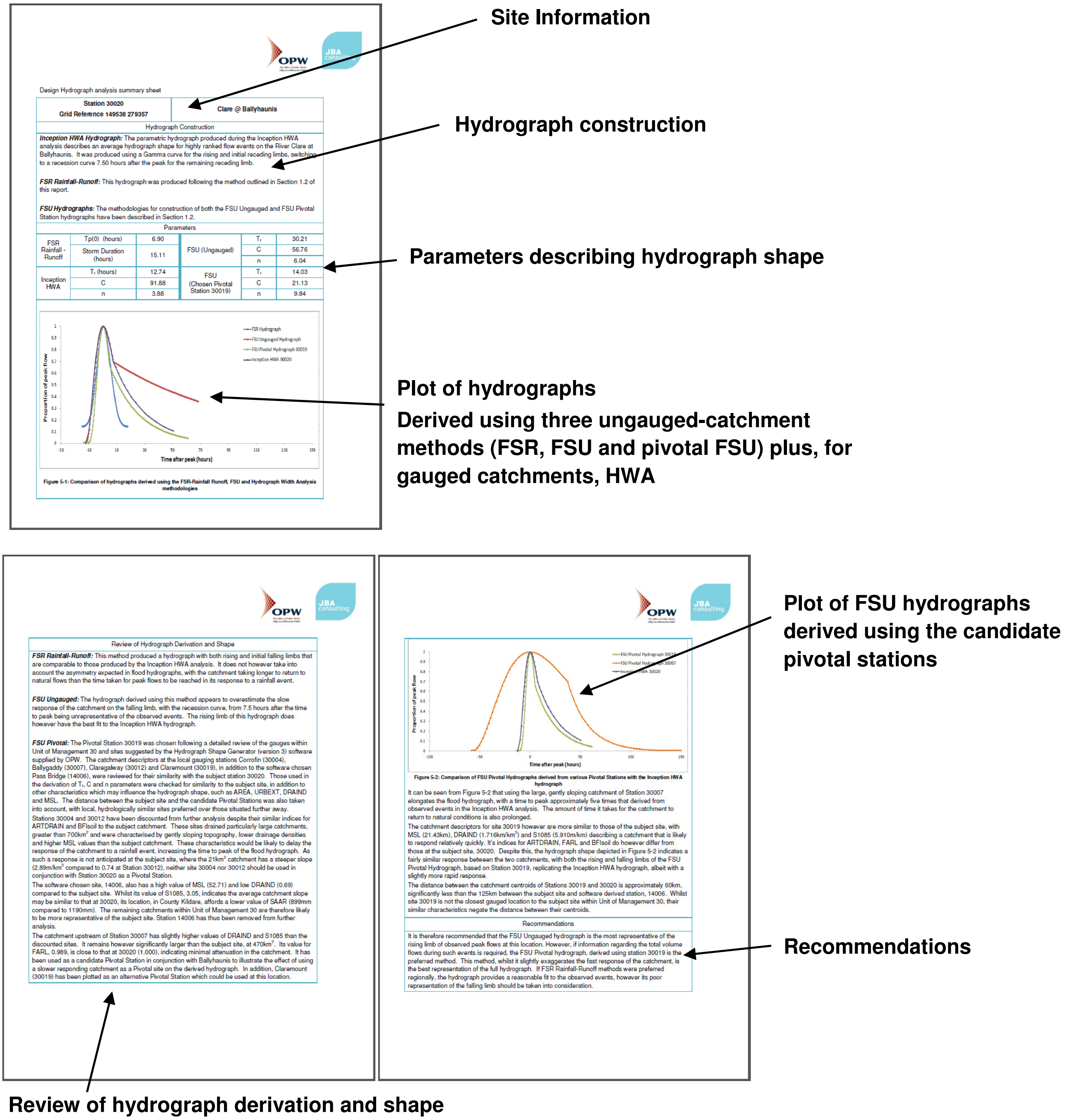
E Comparison of hydrograph shapes

Introduction to design flood hydrograph comparison summary sheets

This appendix provides a comparison of alternative design hydrograph shapes at a sample of five gauged and five ungauged catchments across the Western RBD.

For an explanation of the methods applied, please refer to Section 6.2 and 6.3 of this report. The ungauged variants of the FSR and FSU methods were applied at all ten sites. In addition, at the gauged sites, the FSU methods of averaging the widths of observed hydrographs (HWA) was applied.

Information provided in the summary sheets



Design Hydrograph analysis summary sheet

Station 30020 Grid Reference 149538 279357	Clare @ Ballyhaunis

Hydrograph Construction
Inception HWA Hydrograph: The parametric hydrograph produced during the Inception HWA analysis describes an average hydrograph shape for highly ranked flow events on the River Clare at Ballyhaunis. It was produced using a Gamma curve for the rising and initial receding limbs, switching to a recession curve 7.50 hours after the peak for the remaining receding limb.

Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	6.90	FSU (Ungauged)	T_r	30.21
	Storm Duration (hours)	15.11		C	56.76
				n	6.04
Inception HWA	T_r (hours)	12.74	FSU (Chosen Pivotal Station 30019)	T_r	14.03
	C	91.88		C	21.13
	n	3.88		n	9.84

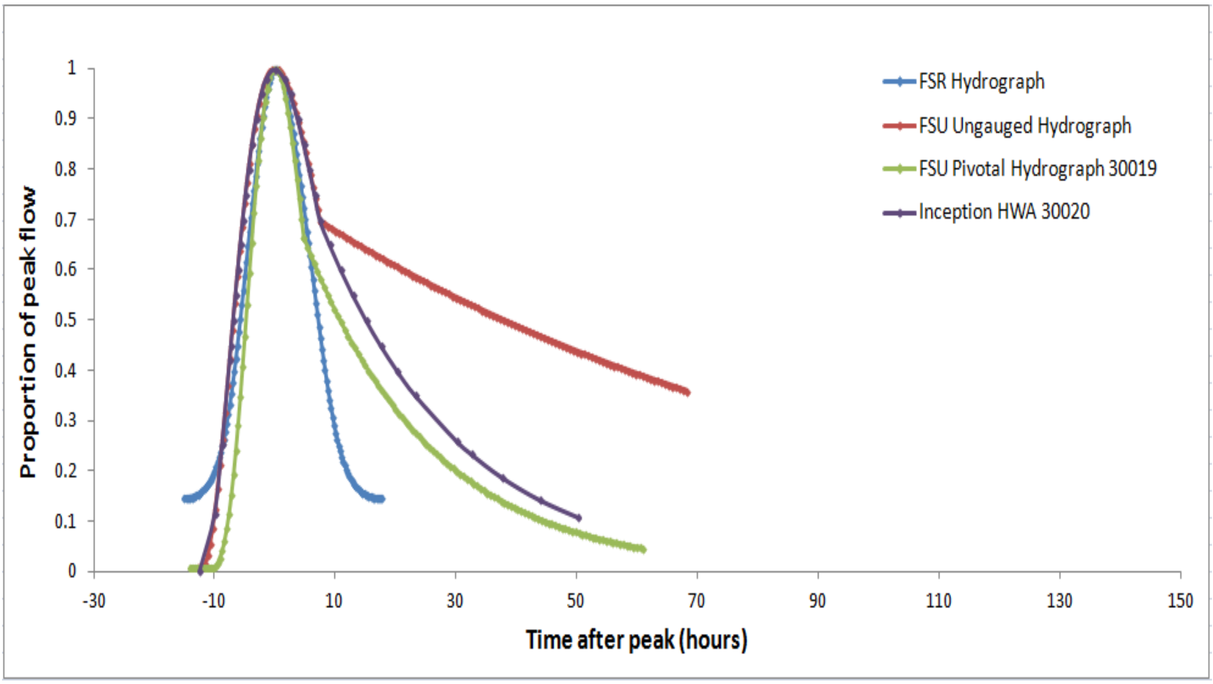


Figure E1-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

Review of Hydrograph Derivation and Shape

FSR Rainfall-Runoff: This method produced a hydrograph with both rising and initial falling limbs that are comparable to those produced by the Inception HWA analysis. It does not however take into account the asymmetry expected in flood hydrographs, with the catchment taking longer to return to natural flows than the time taken for peak flows to be reached in its response to a rainfall event.

FSU Ungauged: The hydrograph derived using this method appears to overestimate the slow response of the catchment on the falling limb, with the recession curve, from 7.5 hours after the time to peak being unrepresentative of the observed events. The rising limb of this hydrograph does however have the best fit to the Inception HWA hydrograph.

FSU Pivotal: The Pivotal Station 30019 was chosen following a detailed review of the gauges within Unit of Management 30 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Corrofin (30004), Ballygaddy (30007), Claregalway (30012) and Claremount (30019), in addition to the software chosen Pass Bridge (14006), were reviewed for their similarity with the subject station 30020. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Stations 30004 and 30012 have been discounted from further analysis despite their similar indices for ARTDRAIN and BFIsoil to the subject catchment. These sites drained particularly large catchments, greater than 700km^2 and were characterised by gently sloping topography, lower drainage densities and higher MSL values than the subject catchment. These characteristics would be likely to delay the response of the catchment to a rainfall event, increasing the time to peak of the flood hydrograph. As such a response is not anticipated at the subject site, where the 21km^2 catchment has a steeper slope (2.89m/km compared to 0.74 at Station 30012), neither site 30004 nor 30012 should be used in conjunction with Station 30020 as a Pivotal Station.

The software chosen site, 14006, also has a high value of MSL (52.71) and low DRAIND (0.69) compared to the subject site. Whilst its value of S1085, 3.05, indicates the average catchment slope may be similar to that at 30020, its location, in County Kildare, affords a lower value of SAAR (899mm compared to 1190mm). The remaining catchments within Unit of Management 30 are therefore likely to be more representative of the subject site. Station 14006 has thus been removed from further analysis.

The catchment upstream of Station 30007 has slightly higher values of DRAIND and S1085 than the discounted sites. It remains however significantly larger than the subject site, at 470km^2. Its value for FARL, 0.989, is close to that at 30020 (1.000), indicating minimal attenuation in the catchment. It has been used as a candidate Pivotal Station in conjunction with Ballyhaunis to illustrate the effect of using a slower responding catchment as a Pivotal site on the derived hydrograph. In addition, Claremount (30019) has been plotted as an alternative Pivotal Station which could be used at this location.

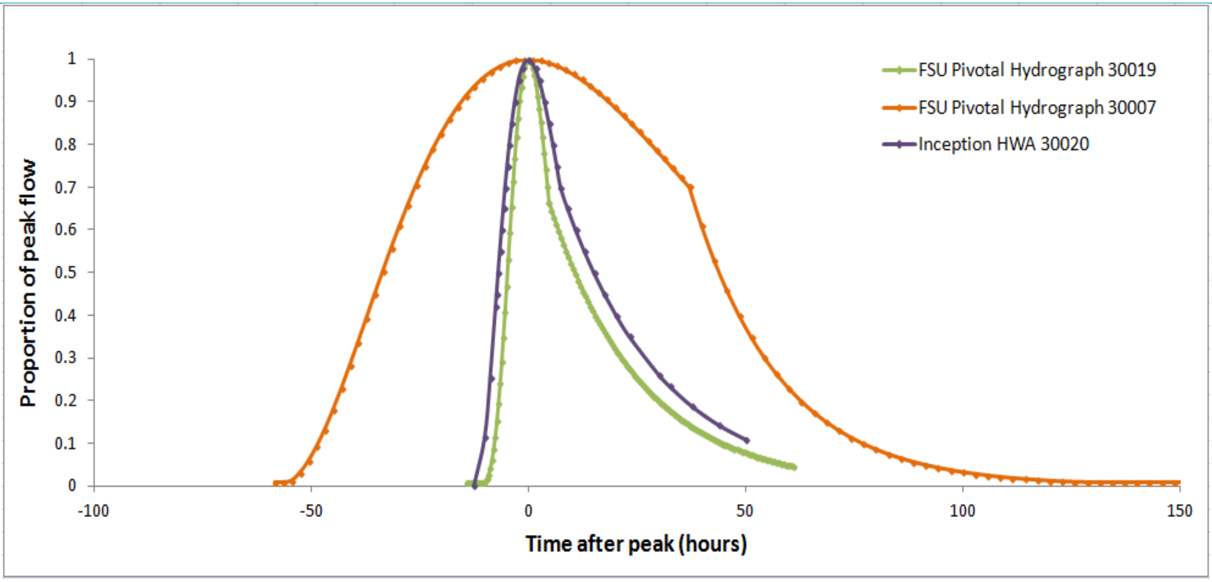


Figure E1-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

It can be seen from Figure E1-2 that using the large, gently sloping catchment of Station 30007 elongates the flood hydrograph, with a time to peak approximately five times that derived from observed events in the Inception HWA analysis. The amount of time it takes for the catchment to return to natural conditions is also prolonged.

The catchment descriptors for site 30019 however are more similar to those of the subject site, with MSL (21.43km), DRAIND (1.716km/km^2) and S1085 (5.910m/km) describing a catchment that is likely to respond relatively quickly. It's indices for ARTDRAIN, FARL and BFIsoil do however differ from those at the subject site, 30020. Despite this, the hydrograph shape depicted in Figure E1-2 indicates a fairly similar response between the two catchments, with both the rising and falling limbs of the FSU Pivotal Hydrograph, based on Station 30019, replicating the Inception HWA hydrograph, albeit with a slightly more rapid response.

The distance between the catchment centroids of Stations 30019 and 30020 is approximately 60km, significantly less than the 125km between the subject site and software derived station, 14006. Whilst site 30019 is not the closest gauged location to the subject site within Unit of Management 30, their similar characteristics negate the distance between their centroids.

Recommendations

It is therefore recommended that the FSU Ungauged hydrograph is the most representative of the rising limb of observed peak flows at this location. However, if information regarding the total volume flows during such events is required, the FSU Pivotal hydrograph, derived using station 30019 is the preferred method. This method, whilst it slightly exaggerates the fast response of the catchment, is the best representation of the full hydrograph. If FSR Rainfall-Runoff methods were preferred regionally, the hydrograph provides a reasonable fit to the observed events, however its poor representation of the falling limb should be taken into consideration.

Design Hydrograph analysis summary sheet

<table>
<tr><td colspan="3">Station 32011
Grid Reference 80906 280601</td><td colspan="3">Bunowen @ Louisburg Weir</td></tr>
<tr><td colspan="6">Hydrograph Construction</td></tr>
<tr><td colspan="6">Inception HWA Hydrograph: The parametric hydrograph produced during the Inception HWA analysis describes an average hydrograph shape for highly ranked flow events on the River Bunowen at Louisburg Weir. It was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 2.81 hours after the peak flow for the remaining receding limb.</td></tr>
<tr><td colspan="6">Parameters</td></tr>
<tr><td rowspan="3">FSR Rainfall - Runoff</td><td>Tp(0) (hours)</td><td>4.51</td><td rowspan="3">FSU (Ungauged)</td><td>T_r</td><td>13.93</td></tr>
<tr><td rowspan="2">Storm Duration (hours)</td><td rowspan="2">11.79</td><td>C</td><td>7.91</td></tr>
<tr><td>n</td><td>10.21</td></tr>
<tr><td rowspan="3">Inception HWA</td><td>T_r (hours)</td><td>7.59</td><td rowspan="3">FSU (Chosen Pivotal Station 32004)</td><td>T_r</td><td>6.17</td></tr>
<tr><td>C</td><td>15.66</td><td>C</td><td>9.80</td></tr>
<tr><td>n</td><td>8.30</td><td>n</td><td>10.00</td></tr>
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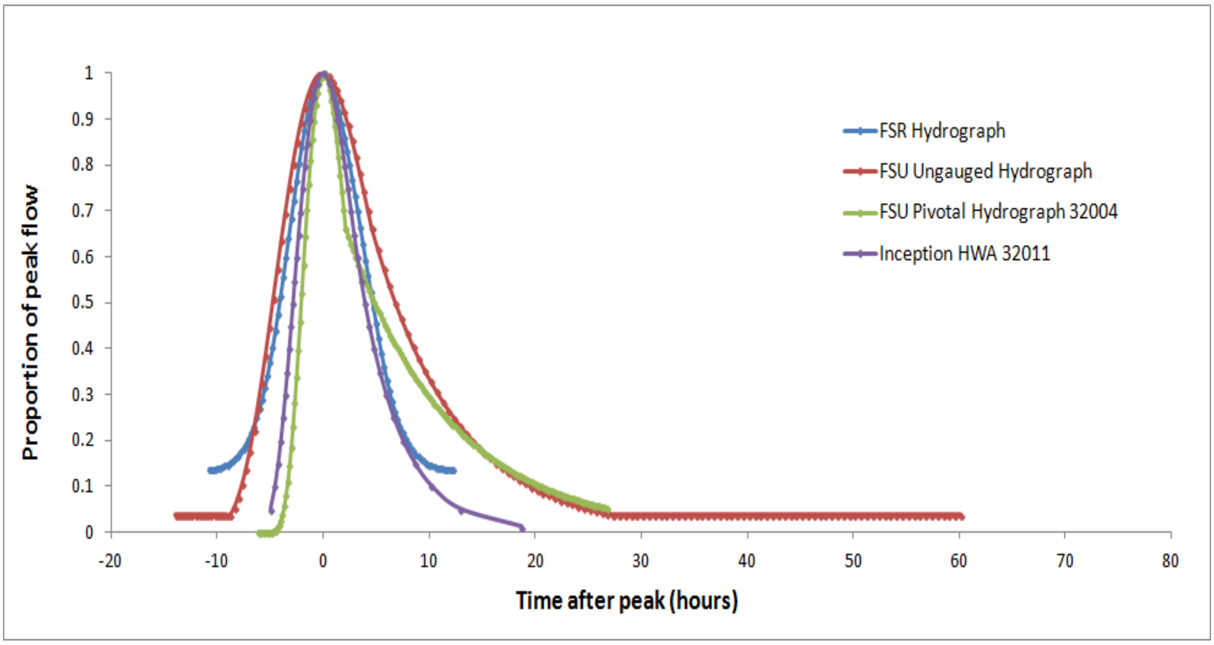

Figure E2-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

FSR Rainfall-Runoff: The FSR Rainfall-Runoff method produced a hydrograph with a falling limb that is very similar to that produced by the Inception HWA analysis. The rising limb of the FSR Rainfall-Runoff hydrograph however achieves a poorer fit to the steep limb of the Inception HWA hydrograph.

FSU Ungauged: This mirrors the FSR Rainfall-Runoff hydrograph, having a similar fit on the rising limb and upper falling limb. The recession curve, from 4.6 hours after the time to peak, is unrepresentative of this quickly responding catchment.

FSU Pivotal: The Pivotal Station 32004 was chosen following a detailed review of the gauges within Unit of Management 32 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Clifden (32004), Coolloughra (32006) and Newport Weir (32012), in addition to the software chosen Ballymullen (23012), were reviewed for their similarity with the subject station 32011. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Station 32012 has been discounted from further analysis due to a particularly low value of FARL, 0.843, compared to that of the subject station, 0.986, as a result of Beltra Lough in the upper catchment. This lake may attenuate peak flows and increase the lag time, causing a hydrograph at Newport Weir that is dissimilar from that expected at Louisburgh Weir where attenuation is less severe. The remaining stations have been used as candidate Pivotal Stations in conjunction with Louisburgh Weir, and their hydrographs plotted for examination:

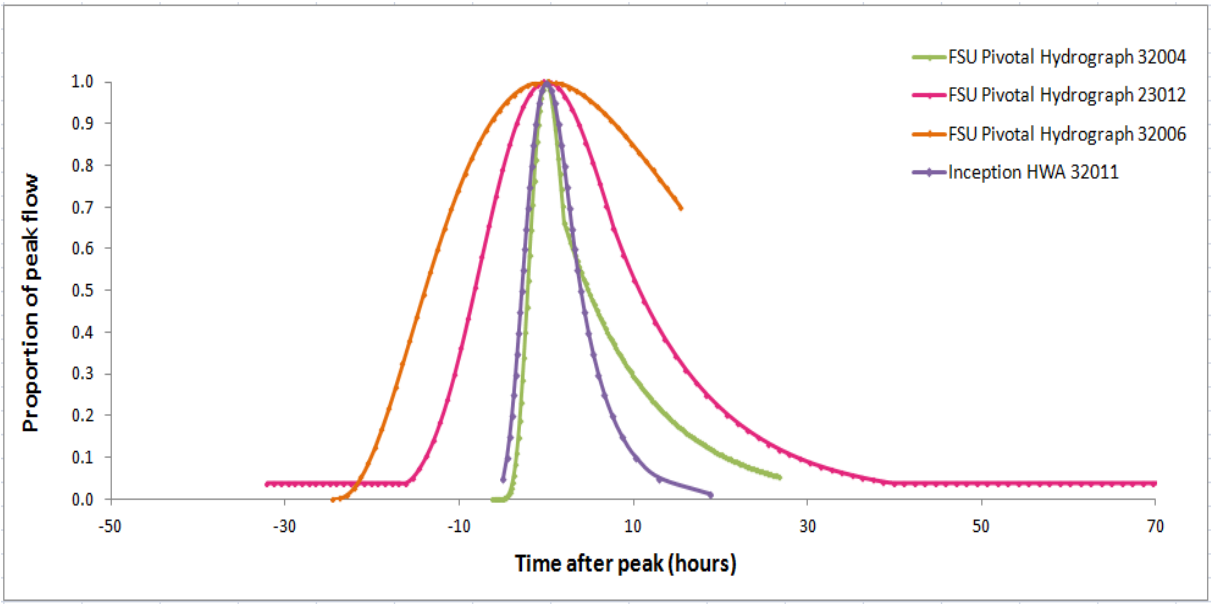

Figure E2-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

The catchment descriptors for these three sites are all fairly similar to the subject site, with station 32006 being the least representative, as BFIsoil and SAAR were larger than at 32011. At station 23012, only URBEXT was particularly high in the Pivotal Station catchment compared to the subject

site (2.43 and 0.15 respectively), whilst station 32004 is most similar, with only DRAIND and ALLUV slightly higher and lower than the values at the subject site respectively. The AREA of the catchments at the subject site and Station 32004 are 70.1km^2 and 32.4km^2 respectively, and the distance between their centroids is approximately 26km. This is a relatively small distance and confirms that in this case, the most hydrologically similar catchment is situated relatively near to the subject site. Whilst the catchments are broadly hydrologically similar, the hydrographs produced from using them as candidate Pivotal Stations suggest that the descriptors BFIsoil, SAAR and URBEXT have a greater influence on the hydrograph shape than DRAIND and ALLUV. This is represented in Figure E2-2, where Pivotal Hydrographs utilising data from Station 23012 and 32006 indicate catchments with slower response times than expected at the subject site. The FSU Pivotal hydrograph, incorporating data from Station 32011, sufficiently describes a faster responding catchment, replicating the rising limb of the Inception HWA hydrograph. The falling limb of the FSU Pivotal 32004 hydrograph is also a good fit to the typical shape derived from observed events for the first 5 hours after the peak flow. Beyond this, it takes slightly longer for the FSU derived hydrograph to return to baseflow conditions, however the fit is not particularly dissimilar from the observed events.
Recommendations
It is therefore recommended that the FSU Pivotal hydrograph, derived using station 32004, is the most representative of the observed hydrographs at this location. Whilst it overestimates the time it takes for the catchment to return to natural flows after the peak event, its representation of the rising limb is significantly better than the hydrographs derived using other methods. If FSR Rainfall-Runoff methods were preferred regionally, the hydrograph provides a reasonable fit to the observed events however the slightly longer lag time should be taken into consideration.

Design Hydrograph analysis summary sheet

Station 34007 Grid Reference 112087 316052	Deel @ Ballycarroon

Hydrograph Construction

Inception HWA Hydrograph: The parametric hydrograph produced during the Inception HWA analysis describes an average hydrograph shape for highly ranked flow events on the River Deel at Ballycarroon. It was produced using a Gamma curve for the rising and initial receding limbs of the hydrograph, switching to a recession curve 7.47 hours after the peak flow for the remaining receding limb.

Parameters

FSR Rainfall - Runoff	Tp(0) (hours)	7.16	FSU (Ungauged)	T_r	21.30
	Storm Duration (hours)	18.54		C	9.26
				n	9.56
Inception HWA	T_r	20.98	FSU (Chosen Pivotal Station 27001)	T_r	17.72
	C	39.11		C	11.24
	n	8.89		n	5.00

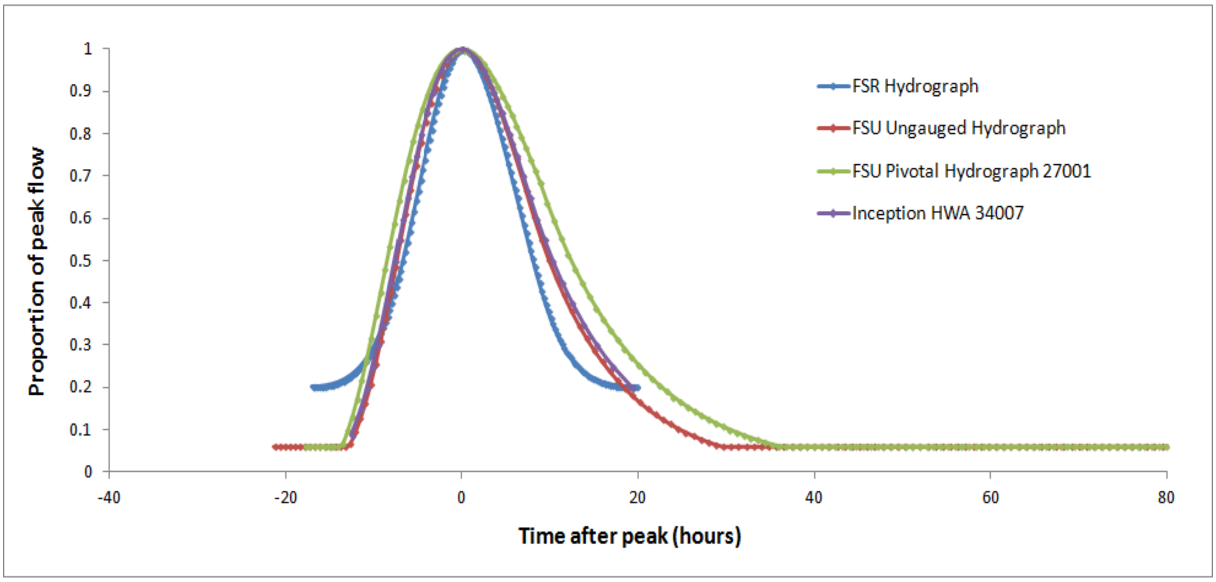

Figure E3-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

Review of Hydrograph Derivation and Shape

FSR Rainfall-Runoff: This method produced a hydrograph with a rising limb that is similar, but slightly steeper, than that produced by the Inception HWA analysis. The falling limb of the FSR Rainfall-Runoff hydrograph also describes a more responsive catchment than that of the Inception HWA hydrograph.

FSU Ungauged: The hydrograph gives a particularly good fit to both the rising and falling limbs of the observed events, representing the responsive nature of the catchment and its return to natural flows.

FSU Pivotal: The Pivotal Station 27001 was chosen following a detailed review of the gauges within Unit of Management 34 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Rahans (34001), Turlough (34018) and Lannagh (34073), in addition to the automatically selected station Inch Bridge (27001) were reviewed for their similarity with the subject station 34007. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away. Station 34001 has been discounted from further analysis due to its low value of FARL, 0.85, compared to that of the subject station 0.978, as a result of Loughs Conn and Cullin, through which a substantial proportion of the catchment drains. Station 34073, in the lower reaches of the River Moy, is also affected by various upstream waterbodies, decreasing FARL to 0.825. These features may attenuate peak flows and increase the lag time, causing hydrograph shapes at Rahans and Lannagh that deviate from that expected at Ballycarroon, where less attenuation of flows occurs. In addition, the URBEXT value for Rahans is much higher, at 12.08 compared to 0.00 at the subject site, and the BFIsoil value for Lannagh is 0.763, whereas at Ballycarroon it is 0.349. These characteristics are likely to result in differing volumes of runoff in these catchments compared to the site of interest and therefore they have not been included in further analysis of candidate Pivotal Stations.

Station 34018 has been investigated in the Inception HWA stage, with the derivation of T_r, C and n parameters. However, whilst the site's location makes it preferable as a Pivotal Station, a number of catchment descriptors are dissimilar. In particular, ARTDRAIN, URBEXT and BFIsoil are higher at Turlough than Ballycarroon:

Catchment Descriptor	Ballycarroon (34007)	Turlough (34018)
ARTDRAIN (%)	0.00	13.70
URBEXT (%)	0.00	5.53
BFIsoil	0.349	0.750

These characteristics imply that using this site as a Pivotal Station may make the hydrograph respond quicker to rainfall as a result of greater runoff volumes and faster routing of flows to the main watercourse. These features are not expected at Ballycarroon and therefore Station 34018 has also been discounted as a candidate Pivotal Station.

The remaining station, 27001, chosen by the Hydrograph Shape Generator software as being most similar to the subject site, has been reviewed manually. The values of ALLUV, ARTDRAIN, S1085, URBEXT, FARL and BFIsoil are consistent with those at Ballycarroon. The AREA of the catchments at the subject site and Station 27001 are 151.7km^2 and 46.7km^2 respectively, and the distance between their centroids is approximately 140km. Whilst this Pivotal Site is therefore located some distance from the subject catchment, its characteristics make it the most suitable site for this analysis.

The FSU Pivotal hydrograph, incorporating data from Station 27001, whilst representing the hydrograph shape of the Inception HWA well, is slightly slower responding than the FSU Ungauged hydrograph.

Recommendations
It is therefore recommended that the FSU Ungauged hydrograph is used, as it is the most representative of flood events at this location. This hydrograph estimates the response time of the catchment and the volume of water well, capturing the overall characteristics of a typical event at Station 34007. If the FSU Ungauged method was not the preferred regional method, the FSR Rainfall-Runoff and FSU Pivotal hydrographs, utilising data from Station 27001, could be used at this location as they give a relatively good fit to the observed data. The former would however infer the catchment is more responsive than has been observed, whilst the latter indicates a slower responding catchment and the conveyance of a greater volume of flood water.

Design Hydrograph analysis summary sheet

Station 34018 Grid Reference 120613 293565	Castlebar @ Turlough

Hydrograph Construction

Inception HWA Hydrograph: The parametric hydrograph produced during the Inception HWA analysis describes an average hydrograph shape for highly ranked flow events on the Castlebar River at Turlough. The process involved discounting a number of events with suspicious data and the removal of events with multi-peaked hydrographs. The resulting hydrograph was produced using a Gamma curve for the rising limb and initial receding limb, switching to the non parametric Hydrograph Width Analysis curve at 25.7 hours after the peak, given the poor fit of the recession and Gamma curves. Caution should be exerted when comparing hydrographs produced using alternative methods with this Inception HWA hydrograph.

Parameters

FSR Rainfall - Runoff	Tp(0) (hours)	6.26	FSU (Ungauged)	T_r	45.41
	Storm Duration (hours)	15.98		C	529.60
				n	2.02
Inception HWA	T_r	119.25	FSU (Chosen Pivotal Station 34003)	T_r	68.27
	C	900.99		C	n/a
	n	2.88		n	2.85

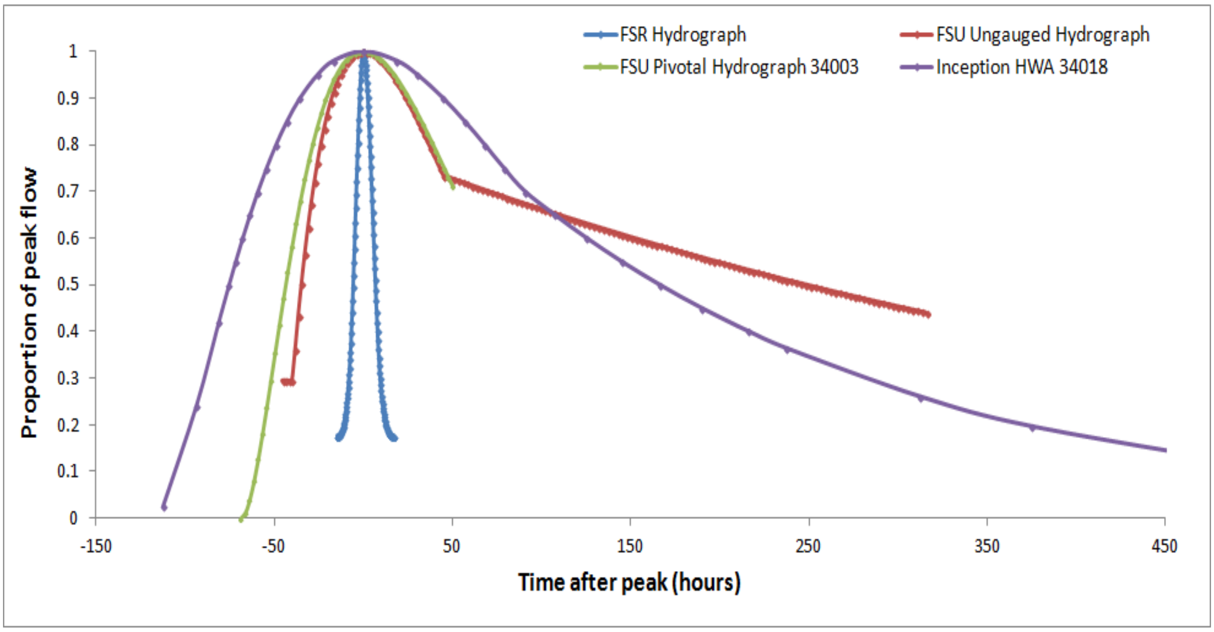

Figure E4-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

Review of Hydrograph Derivation and Shape

FSR Rainfall-Runoff: This method produced a hydrograph that does not represent peak flow events at Turlough. It estimated a time to peak of 6.5 hours, very much shorter than that implied by observed hydrographs.

FSU Ungauged: This hydrograph also has a poor fit to the recorded events, estimating flows to be routed through the catchment more quickly than observed. In addition, the recession curve, from 45 hours after the time to peak, is very shallow as a result of the low FARL value at this location (0.732).

FSU Pivotal: The Pivotal Station 34003 was chosen following a detailed review of the gauges within Unit of Management 34 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Rahans (34001), Foxford (34003), Ballylahan (34004), Charlestown (34031) and Lannagh (34073) were reviewed for their similarity with the subject station 34018. The Hydrograph Shape Generator software automatically selected the parameters at 34018 given the catchment characteristics matched those describing the gauged subject site. Given this analysis requires treatment of the site as an ungauged location, this station was removed from the list of possible Pivotal Stations and alternative sites were examined for their suitability.

The catchment descriptors used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the potential Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Stations 34004, 34031 and 34073 have not been analysed in further detail as a number of their catchment descriptors are dissimilar to those for the subject site, potentially producing unrealistic hydrograph shapes. For sites 34001 and 34031, considerable differences were noted in BFIsoil, URBEXT, FARL and ARTDRAIN to those at 34018:

Catchment Descriptor	**Turlough (34018)**	**Ballylahan (34004)**	**Charlestown (34031)**
BFIsoil	0.750	0.485	0.330
URBEXT (%)	5.53	0.81	0.62
FARL	0.732	0.959	1.000
ARTDRAIN (%)	13.70	0.00	0.00

Station 34073, whilst having a lower value of FARL (0.85), mirroring the greater attenuation expected at the subject site, has a poor match for ARTDRAIN, URBEXT, ALLUV and BFIsoil. For this reason it has also been excluded from further analysis. The remaining stations, 34001 and 34003 have more comparable catchment descriptors to station 34018 and therefore have been used to derive candidate Pivotal Hydrographs which are plotted below:

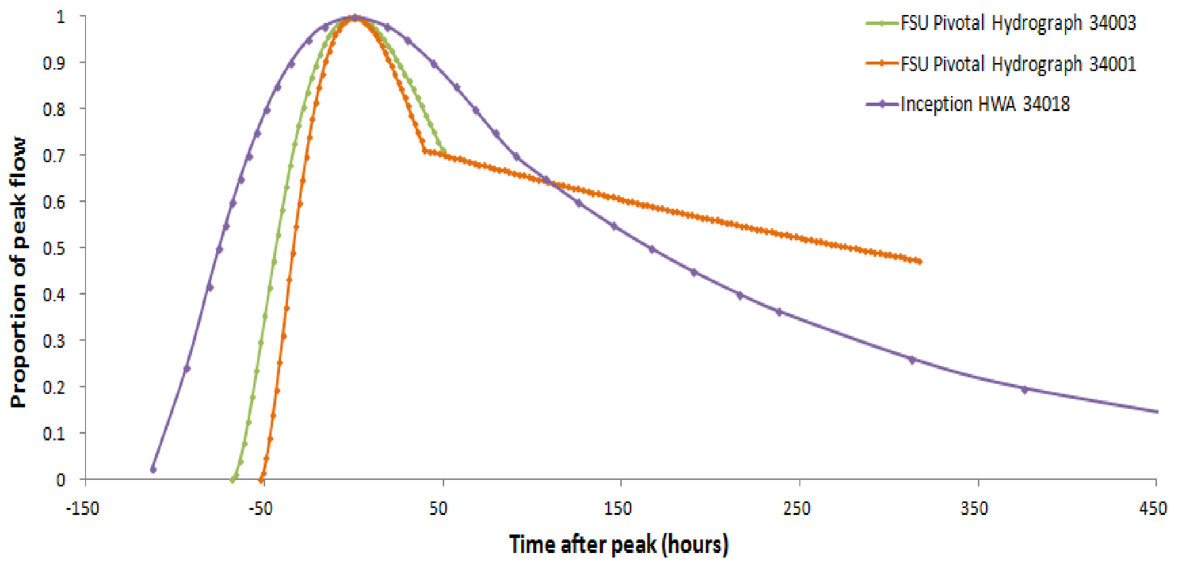

Figure E4-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

Whilst the values of ARTDRAIN, FARL, BFIsoil and DRAIND are more appropriate at these sites, values of URBEXT (≈ 0.8) remain much lower than at 34018. Typically, more urban areas induce a shorter time to peak and a steeper hydrograph due to the greater volume of runoff and faster routing of water to the main watercourse. The observed events at Turlough do not reflect this process though given the URBEXT value of 5.5 indicates the catchment is still predominantly rural. Given the subject catchment is also much smaller and steeper than these potential Pivotal Sites, it is expected that significant attenuation by Castlebar Lough causes the longer lag time observed at the subject location. However, as the parameters used in the FSU derivation utilise FARL, this analysis may indicate that the methodology is unable to accurately represent the degree of attenuation in catchments containing large waterbodies.

The FSU Pivotal hydrograph, incorporating data from Station 34003, whilst underestimating the lag time, remains the best fit to the observed data. It may be disconcerting that using the FSU Pivotal method at this location, utilising both site specific information and data from local gauges, is unable to reproduce either the rising limb or falling limb of the Inception HWA hydrograph. However, the uncertainty in the derivation of the Inception HWA hydrograph outlined in Hydrograph Construction above, implies that confidence in the shape of this hydrograph is limited.

Recommendations

As noted in the inception report, the Turlough at Castlebar appears to experience flood hydrographs that are much more prolonged than expected for a catchment of its size. A more detailed investigation into the hydraulics of the watercourse (including backwater effects) is being carried out as part of the hydraulic modelling study, and a decision on the design flood hydrograph will be made after that. Neither the FSU Ungauged Hydrograph nor the FSR Rainfall-Runoff methodologies appear to be suitable. The FSU pivotal hydrograph provides little improvement on the ungauged hydrograph.

Design Hydrograph analysis summary sheet

Station 35002 **Grid Reference 163917 325724**	**Owenbeg @ Billa Bridge**

Hydrograph Construction

Inception HWA Hydrograph: The parametric hydrograph produced during the Inception HWA analysis describes an average hydrograph shape for highly ranked flow events on the Owenbeg River at Billa Bridge. It was produced using a Gamma curve for the rising limb, with the receding limb derived using the non parametric Hydrograph Width Analysis curve, given the poor fit of the recession and Gamma curves after the peak.

Parameters

FSR Rainfall - Runoff	Tp(0) (hours)	5.13	FSU (Ungauged)	T_r	22.06
	Storm Duration (hours)	12.20		C	35.79
				n	6.64
Inception HWA	T_r	20.80	FSU (Chosen Pivotal site 35011)	T_r	21.92
	C	n/a		C	n/a
	n	10.00		n	9.98

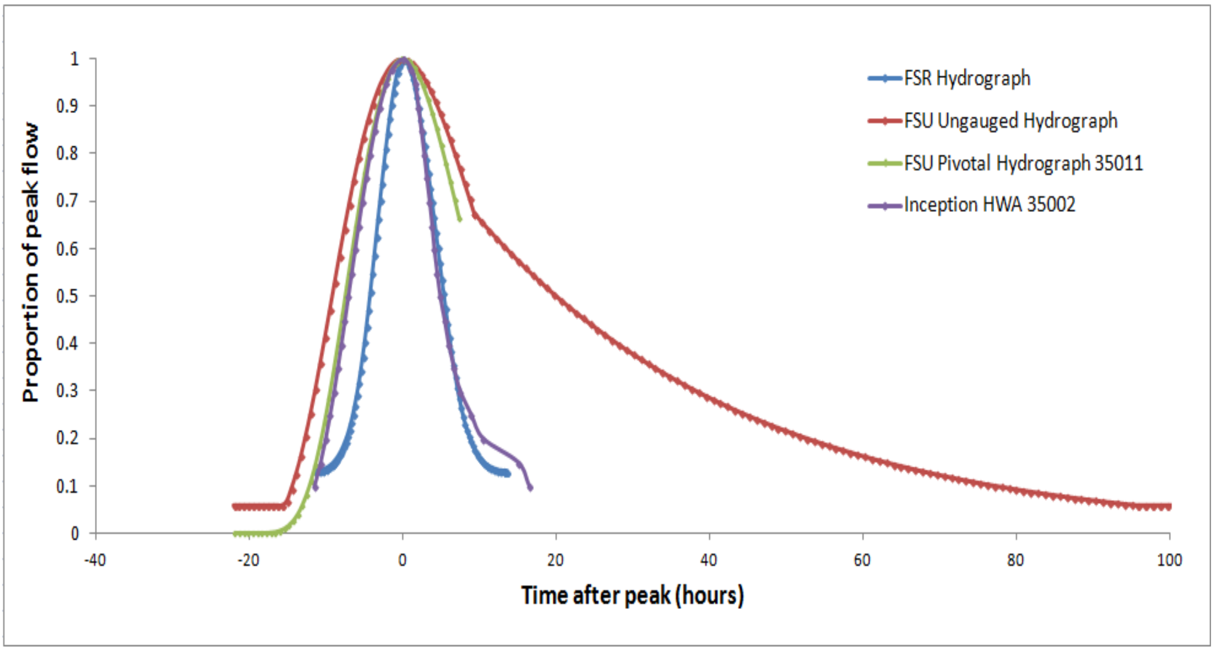

Figure E5-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

FSR Rainfall-Runoff: This method produced a hydrograph with a falling limb that is very similar to that produced by the Inception HWA analysis. The rising limb of the FSR Rainfall-Runoff hydrograph however achieves a poorer fit to the steep limb of the Inception HWA hydrograph.

The FSU derived hydrographs do not replicate this similarity in the falling limb, with flows taking a longer time to be routed through the catchment. They do however illustrate a better fit to the rising limb than the FSR Rainfall-Runoff derived hydrograph.

FSU Ungauged:. The FSU Ungauged hydrograph describes a catchment which is slightly less responsive on the rising limb than the observed events of the Inception HWA hydrograph. In addition, the recession curve, from 9.3 hours after the time to peak, is unrepresentative of this quickly responding catchment.

FSU Pivotal: The Pivotal Station 35011 was chosen following a detailed review of the gauges within Unit of Management 35 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Ballynacarrow (35001), Ballygrania (35003), Ballysadare (35005), and Dromahair (35011) were reviewed for their similarity with the subject station 35002.

The catchment descriptors used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the potential Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Stations 35003 and 35005 have been discounted from further analysis due to low values of FARL (0.814 and 0.898 respectively) compared to that of the subject station, 0.986. These are due to the presence of numerous waterbodies in their upper catchments, such as Lough Arrow 20km upstream of Station 35003. As a result, peak flows are likely to experience some attenuation, slowing the response of the catchment to rainfall events and increasing the lag time of the hydrographs. This process is unlikely to occur at the subject station, 35002 and therefore these sites are deemed unrepresentative as Pivotal Stations. The remaining stations have been used as candidate Pivotal Stations for Billa Bridge and the resulting hydrographs have been plotted below:

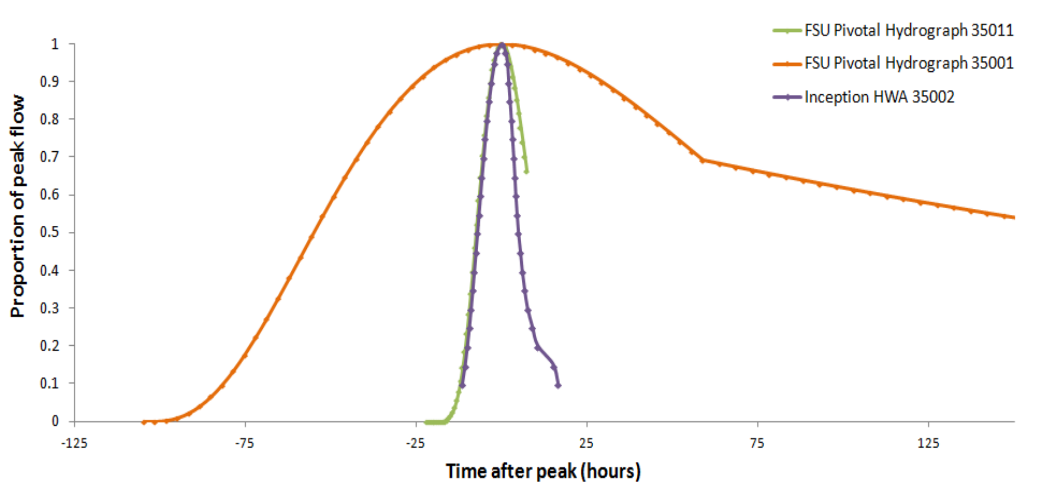

Figure E5-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

Some of the catchment descriptors for sites 35001 and 35011 are similar to the subject site, with FARL values of 0.923 and 0.978 much closer to that at 35002, 0.986. Whilst both of the sites represent catchments that are slightly more urbanised than the subject catchment, the catchment descriptors MSL, DRAIND, SAAR, ALLUV and BFIsoil are similar between these three sites. However, the indices representing catchment area and slope are less comparable to the site of interest:

Catchment Descriptor	**Billa Bridge (35002)**	**Ballynacarrow (35001)**	**Dromahair (35011)**
AREA (km^2)	88.8	299.5	293.2
S1085 (m/km)	13.3	0.1	4.1

The larger area and shallow slope of the catchment area upstream of station 35001 is likely to contribute to the slow response to rainfall events, causing the wider hydrograph depicted in Figure E5-2. At station 35011, the steeper slope, combined with a higher value of ARTDRAIN, 4.78%, may route flows relatively quickly through the catchment, offsetting the large catchment area. The FSU Pivotal hydrograph, incorporating data from Station 35011, sufficiently replicates the fast response on both the rising limb and first 7 hours after the peak flow. The steep nature of this hydrograph is aided by the high gradient and arterial drainage of the Pivotal station, whilst the hydrograph derived from Station 35001, illustrates the effect of using an unrepresentative large, shallow gradient catchment as a Pivotal Station.

Recommendations

It is therefore recommended that the FSU Pivotal hydrograph, derived using station 35011, is the most representative of the Inception HWA observed flows at this location. Whilst it may slightly overestimate the time it takes for the catchment to return to natural flows after the peak event, it has the best fit to both the rising and initial falling limbs of the Inception HWA hydrograph in comparison to the hydrographs derived using alternative methods. If this hydrograph were to be incorporated into the hydraulic models, a more detailed investigation into the derivation of a recession limb would be required. The FSR Rainfall-Runoff derived hydrograph, whilst describing a more responsive catchment than that observed, has an acceptable fit and could be utilised if the FSR method was preferred regionally. The FSU Ungauged Hydrograph should not be used at this site as it exaggerates the length of time it takes for this catchment to respond to a rainfall event.

Design Hydrograph analysis summary sheet

<table>
<tr><td colspan="3">Ungauged Site
Grid Reference 152472 229990</td><td colspan="3">Clarinbridge @ Athenry</td></tr>
<tr><td colspan="6">Parameters</td></tr>
<tr><td rowspan="6">FSR Rainfall - Runoff</td><td rowspan="3">Tp(0) (hours)</td><td rowspan="3">7.87</td><td rowspan="3">FSU (Ungauged)</td><td>T_r</td><td>50.03</td></tr>
<tr><td>C</td><td>42.48</td></tr>
<tr><td>n</td><td>6.54</td></tr>
<tr><td rowspan="3">Storm Duration (hours)</td><td rowspan="3">16.6</td><td rowspan="3">FSU (Pivotal site 26022)</td><td>T_r</td><td>53.64</td></tr>
<tr><td>C</td><td>66.94</td></tr>
<tr><td>n</td><td>5.00</td></tr>
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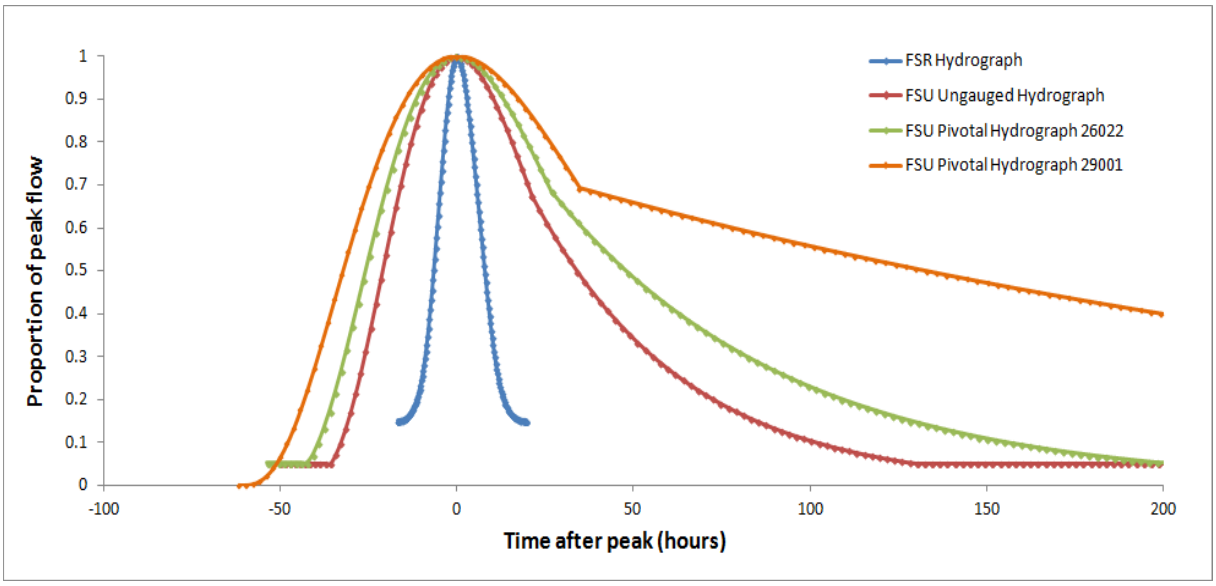


Figure E6-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

Review of Hydrograph Derivation and Shape

This ungauged site was not included in the Inception Hydrograph Width Analysis (HWA) and therefore there is no comparison between the derived hydrographs and the expected shape as with the gauged locations. This analysis focuses on the differences in hydrograph shape between the various methods tested.

FSR Rainfall-Runoff: This method produced a hydrograph with steep rising and falling limbs compared to the FSU methods, with a time to peak of approximately 17 hours. The near-symmetrical limbs do not account for the longer time taken for the channel to return to natural flows than its initial rapid response to rainfall.

FSU Ungauged: The hydrograph derived using this method estimates a slower catchment response than the FSR Rainfall-Runoff method, with the peak flow occurring approximately 37 hours into the event.

FSU Pivotal: The Pivotal Station 26022 was chosen following a detailed review of the gauges in Units of Management 29 and 26 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Rathgorgin

(29001) and Craughwell (29007) in addition to Kilmore (26022) and the software chosen Sunville (25005) were reviewed for their similarity with the ungauged site at Athenry. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Station 29007 has been excluded from further analysis given the large differences between key catchment descriptors at Craughwell and the subject site. Of particular note are the parameters MSL, ARTDRAIN, FARL, URBEXT and ALLUV, which are unrepresentative of the catchment upstream of Athenry. FARL, for example, at 0.969, implies a degree of attenuation which is not reflected in the value of 1.000 at Athenry, whilst the catchment at Craughwell is partially urbanised (URBEXT of 1.29%) compared to the rural catchment at the subject location. The station 29007 does provide similar descriptors for DRAIND, SAAR, S1085 and BFIsoil however these do not outweigh the number of parameters that make the site unsuitable for use as a Pivotal Station.

The software chosen site, 25005, is less urbanised than Craughwell (URBEXT is 0.65 at Sunville) and has a more representative value for FARL (0.999). It however still performs poorly with respect to MSL, ARTDRAIN and ALLUV, the latter two of which influence the T_r parameter. This gauged location is also significantly larger than the subject site (193km^2 compared to 32km^2) and therefore it is likely that alternative stations offer more suitable catchments for use in Pivotal adjustments. This station has therefore been removed from further analysis.

The catchments upstream of stations 29001 and 26022 have descriptors that are more consistent with those at Athenry compared to stations 29007 and 25005. The FSU Pivotal hydrographs have been plotted for each of these sites in Figure E6-1. The values of DRAIND, URBEXT and ALLUV are more similar to those of the subject site than the catchment descriptors from the other gauging stations (1.039, 0.66 and 2.29 respectively), whilst S1085, FARL, BFIsoil and SAAR are comparable between station 29001 and Athenry. However, the parameters for MSL and ARTDRAIN are not a good fit to those at the subject site. The ARTDRAIN value of 0.01 compared to 1.03 at Athenry suggests that the T_r parameter will vary between the two sites, influencing the shape of both the rising and falling limbs. It is likely that this parameter contributes to the slower response time of the hydrograph in Figure E6-1 and therefore it is suggested that station 29001 is not used as a Pivotal site for Athenry.

Station 26022 offers a better fit to the catchment descriptors at Athenry for the majority of parameters, including URBEXT, ALLUV, MSL and FARL. ARTDRAIN remains low at 0.04 but improves upon the value of 0.01 at station 29001. Whilst the values of BFIsoil, SAAR, S1085 and DRAIND are less similar to those at the subject site than station 29001, they remain within a suitable range for use of Station 26022 as a Pivotal site. These values result in a hydrograph which represents a more responsive catchment than station 29001, as shown in Figure E6-1.

Recommendations

The various versions of the FSU hydrograph are all considerably wider than the FSR hydrograph. Without any observed data it is not possible to give a definitive recommendation on which is the most realistic design hydrograph shape. Further comparisons are described in the main text of the report.

Ungauged Site **Grid Reference 166343 315264**			**Carrigans Upper @ Ballymote**		
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	3.08	FSU (Ungauged)	T_r	56.31
				C	49.78
				n	6.26
	Storm Duration (hours)	6.67	FSU (Pivotal site 26022)	T_r	41.65
				C	51.98
				n	5.00

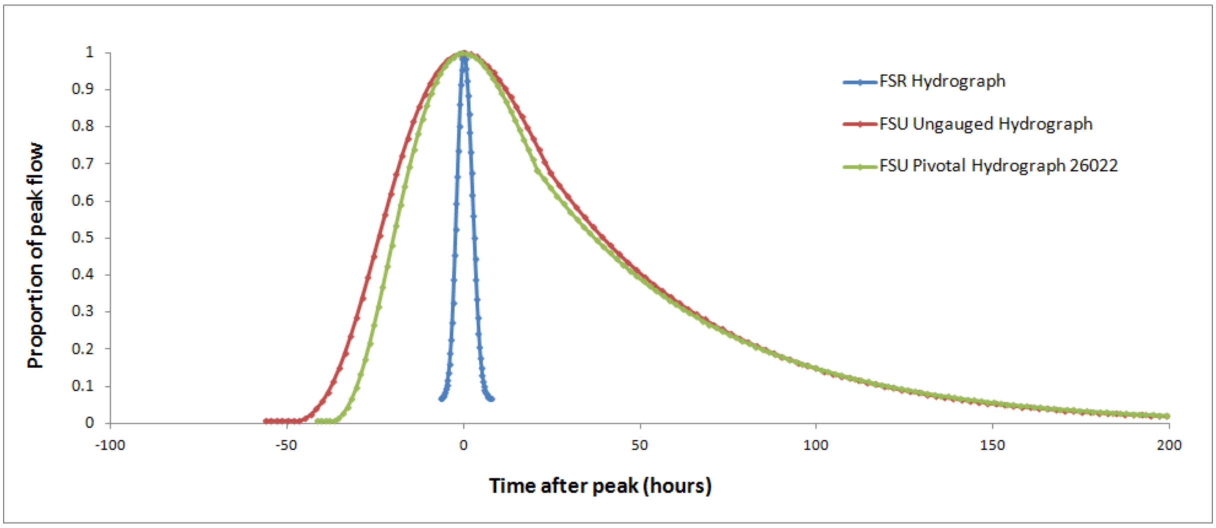


Figure E7-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

Review of Hydrograph Derivation and Shape

This ungauged site was not included in the Inception Hydrograph Width Analysis (HWA) and therefore there is no comparison between the derived hydrographs and the expected shape as with the gauged locations. This analysis focuses on the differences in hydrograph shape between the various methods tested.

FSR Rainfall-Runoff: This method produced a hydrograph with particularly steep rising and falling limbs in comparison with the FSU hydrographs. Its symmetrical shape does not take into account the change in catchment response throughout the event and different rates at which flow pathways transport water to the channel, which would result in a steep rising limb and shallow falling limb as seen in the FSU hydrographs. However, given the catchment size of 2.5km^2, the hydrograph's representation of a short-lived flood event reflects the small drainage area.

FSU Ungauged: The hydrograph derived using the FSU ungauged methodology describes a slowly responding catchment in comparison to the FSR Rainfall-Runoff method, with the peak flow occurring at approximately 50 hours into the event. It is highly unlikely that a catchment of this size would support a flood for this duration, therefore this method is believed to be unsuitable at Ballymote.

FSU Pivotal: The FSU Pivotal hydrograph also represents a slowly responding catchment, which is unlikely given the size and urban extent of the catchment. The process of choosing the Pivotal Station 26022 is detailed below.

The Pivotal Station 26022 was chosen following a detailed review of the gauges within Units of Management 35 and 26 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the gauging stations Ballynacarrow (35001), Kilmore (26022) and the software chosen Sunville (25005) were reviewed for their similarity with the ungauged site at Ballymote. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Station 35001 has been discounted from further analysis due to the large differences between key catchment characteristics at Ballynacarrow and Ballymote. Of particular note are the parameters AREA (299km^2 at Station 35001 compared to 2.5km^2 at the subject site), MSL (24.7km at Ballynacarrow compared to 2.2km at Ballymote) and URBEXT (0.33 at Station 35001 compared to 14.57 at Ballymote). In addition, S1085, which influences the T_r parameter, various from 0.1 at the candidate pivotal station to 2.6 at the subject site. More suitable values are present for DRAIND, FARL, SAAR, ALLUV and BFIsoil, however these do not outweigh the number of descriptors that make Ballynacarrow unsuitable for use as a Pivotal Station.

The software chosen station, 25005, represents a catchment with a similar degree of attenuation to Ballymote (FARL is 0.999 and 1.000 respectively). It also has comparable parameters for BFIsoil, SAAR, DRAIND and S1085, which influence the T_r, C and n parameters of the hydrograph shape. However, the disparity between the AREA, ALLUV, MSL, URBEXT and ARTDRAIN parameters at the two sites is also likely to be reflected in the hydrograph shape.

Catchment Descriptor	Ballymote	Sunville (25005)
AREA (km^2)	2.5	192.6
ALLUV (%)	0.00	7.99
MSL (km)	2.2	25.0
URBEXT (%)	14.57	0.65
ARTDRAIN (%)	0.00	8.97

This site has therefore not been plotted in Figure E7-1, as it is not considered suitable for use as a Pivotal station.

The catchment upstream of Station 26022 is described by parameters that improve upon those at stations 35001 and 25005. As for the software derived station, the catchment descriptors BFIsoil, SAAR and S1085 are similar to those at Ballymote, whilst AREA and MSL remain significantly different (61.9km^2 and 13.9km^2 respectively). However, Station 26022 improves upon the parameters at Station 25005 for ALLUV (1.27) and ARTDRAIN (0.04), influencing the T_r parameter. Despite this, the hydrograph shape depicted in Figure E7-1 indicates the FSU Pivotal method is not taking account of the small catchment area at Ballymote, resulting in an unrealistic duration for the hydrograph. Use of the FSU Pivotal method, with Station 26022 as the most representative pivotal station, should therefore not be used to estimate the hydrograph at Ballymote.

Recommendations

The various versions of the FSU hydrograph are extremely wide in comparison with the FSR hydrograph and they are considered unrepresentative of the expected flood duration on this very small

catchment. The FSR hydrograph is more realistic. Further tests of the hydrographs can be found in the main text of the report.

Ungauged Site **Grid Reference 144139 246625**			**Grange @ Corrofin**		
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	12.15	FSU (Ungauged)	T_r	28.24
				C	45.80
				n	6.36
	Storm Duration (hours)	25.24	FSU (Pivotal site)	T_r	39.70
				C	8.31
				n	9.10

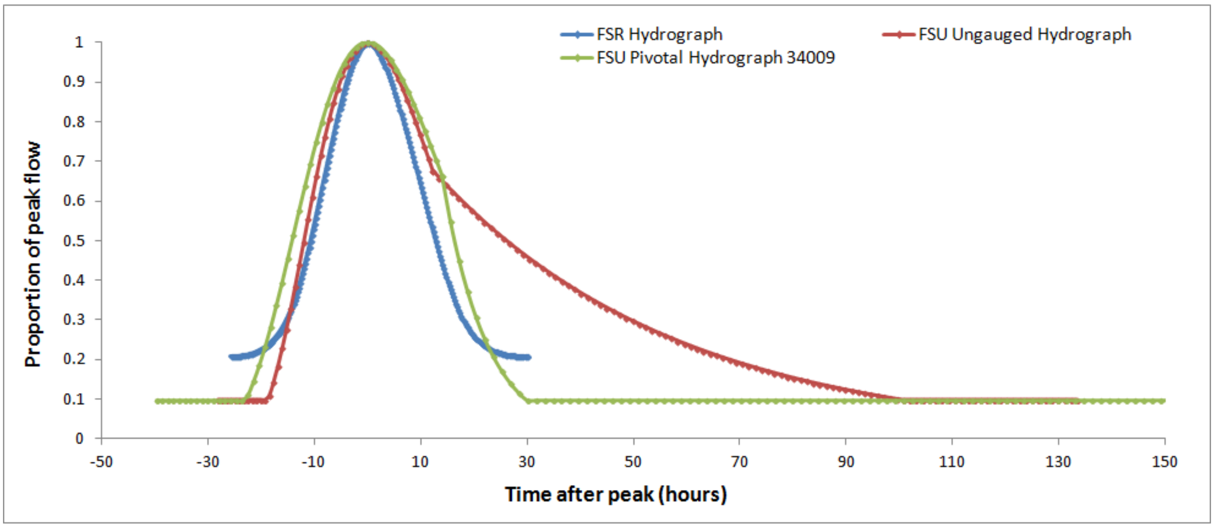


Figure E8-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

Review of Hydrograph Derivation and Shape

This ungauged site was not included in the Inception Hydrograph Width Analysis (HWA) and therefore there is no comparison between the derived hydrographs and the expected shape as with the gauged locations. This analysis focuses on the differences in hydrograph shape between the various methods tested.

FSR Rainfall-Runoff: This method produced a hydrograph with rising and falling limbs of a similar gradient to the FSU methods, with a time to peak of approximately 12 hours. Whilst comparable in shape, it does not account for the asymmetry expected in flood hydrographs which results from the catchment taking longer to return to natural flows than the time taken for peak flows to be reached in its response to rainfall.

FSU Ungauged: The hydrograph derived using this method estimates a comparable rising limb and initial falling limb to those from the FSR Rainfall-Runoff method. However, 12 hours after the peak flow, the falling limb decreases at a shallower gradient, implying a large proportion of the flow is from throughflow. This may be unrealistic given the catchment is not particularly permeable (BFIsoil is 0.571) and there is a high degree of arterial drainage works routing flows to the Grange River (ARTDRAIN is 18.1%).

FSU Pivotal: The Pivotal Station 34009 was chosen following a thorough review of the gauges in Units of Management 30 and 34 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Ballygaddy (30007) and Clare (30012) in addition to Curraghbonaun (34009) and the software chosen Boleany (11001) were reviewed for their similarity with the ungauged site at Corrofin. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

The software chosen site, 11001, has been excluded from further analysis given the disparity between two key catchment descriptors at Boleany compared to Corrofin. The differences in ARTDRAIN (6.3% compared to 18.05% at the subject site) and ALLUV (4.60% compared to 1.02% at the subject site) are much greater than at the local sites 30007 and 30012. Whilst some of the remaining descriptors, including AREA and S1085, are more comparable to those at Corrofin, ARTDRAIN and ALLUV are likely to alter the hydrograph shape through the T_r parameter. The remaining stations have more comparable values for these parameters and are therefore likely to act as more suitable Pivotal stations.

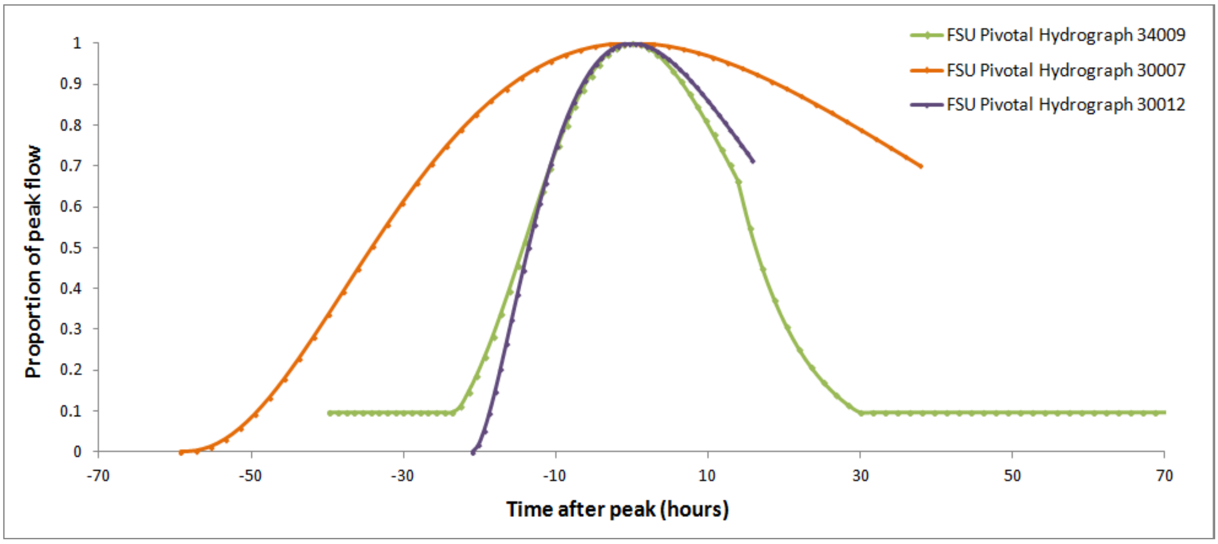


Figure E8-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations

The FSU Pivotal hydrographs have been plotted for these three stations in Figure E8-2. It is clear that using station 30007 as a Pivotal site results in a hydrograph with a longer response time – it takes 55 hours for the hydrograph to reach peak flows compared to 20-25 hours when stations 30012 or 34009 are utilised. This extended response may be explained by the 470km² catchment at Ballygaddy, combined with the slightly more permeable soils and attenuation. These characteristics appear to outweigh the steeper slope and greater urban extent in catchment 30007 compared to the catchment upstream of site 30012, which is also large (1073km²) yet produces a relatively narrow hydrograph. Given the disparity between the hydrograph based on station 30007, the alternative FSU hydrographs and the FSR Rainfall-Runoff method, it is suggested that this site is not used as a Pivotal station for Corrofin.

The majority of catchment descriptors for Station 30012 are comparable to those at Corrofin, with ARTDRAIN, FARL, BFIsoil and ALLUV providing similar parameters to the subject site. The hydrograph shape reflects this, with both the rising limb and initial falling limb having similar gradients to the FSU Ungauged and FSR Rainfall-Runoff hydrographs. However, the disparity between AREA (1073km² compared to 125.3km² at Corrofin) suggests the flow pathways are likely to be substantially

different between these two catchments despite the similarity in hydrograph shape. If no other sites could be utilised as a Pivotal station Clare could be used with caution, however given station 34009 remains a viable option, station 30012 is not likely to be used as the Pivotal station for Corrofin. Station 34009, Curraghbonaun, offers a better fit to the catchment descriptors at the subject site. Of particular note are the similarities in AREA, MSL, DRAIND, FARL, ALLUV and URBEXT, whilst BFIsoil, and S1095 still offer suitable values. ARTDRAIN, at 5.73%, is less comparable to the subject site than at stations 30007 and 30012. However, a degree of drainage is accounted for, and, given the remaining descriptors that contribute to the T_r parameter are consistent with those at Corrofin, it is likely that the predicted hydrograph shape is representative of the subject site. Whilst the centroid of this catchment is approximately 52km from that of the subject site, the above review of more local gauging stations suggests that station 34009, despite not being the closest to the subject catchment, is the most hydrologically similar.
Recommendations
It is therefore recommended that the FSU Pivotal hydrograph, derived using station 34009, is the most representative of the flows at this location. The rising and falling limbs appear to replicate the expected response to a rainfall event given the natural catchment topography and additional arterial drainage. If the FSU Ungauged method were preferred regionally, the hydrograph provides a reasonable representation of the catchment flows for the rising and initial falling limb, however the volume of flow is likely to be misrepresented given the delayed return to natural conditions. If the FSR Rainfall-Runoff method were utilised regionally, it could be used at Corrofin as it has a similar hydrograph shape to the FSU methods. At this ungauged site however, observed data is not available to support this conclusion. Further tests of the hydrographs can be found in the main text of the report.

Design Hydrograph analysis summary sheet

Ungauged Site **Grid Reference 162244 216389**			**St Clerans South @ Lough Rea**		
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	3.81	FSU (Ungauged)	T_r	61.72
				C	3087.17
				n	1.20
	Storm Duration (hours)	8.14	FSU (Pivotal site 34018)	T_r	32.05
				C	169.23
				n	1.27

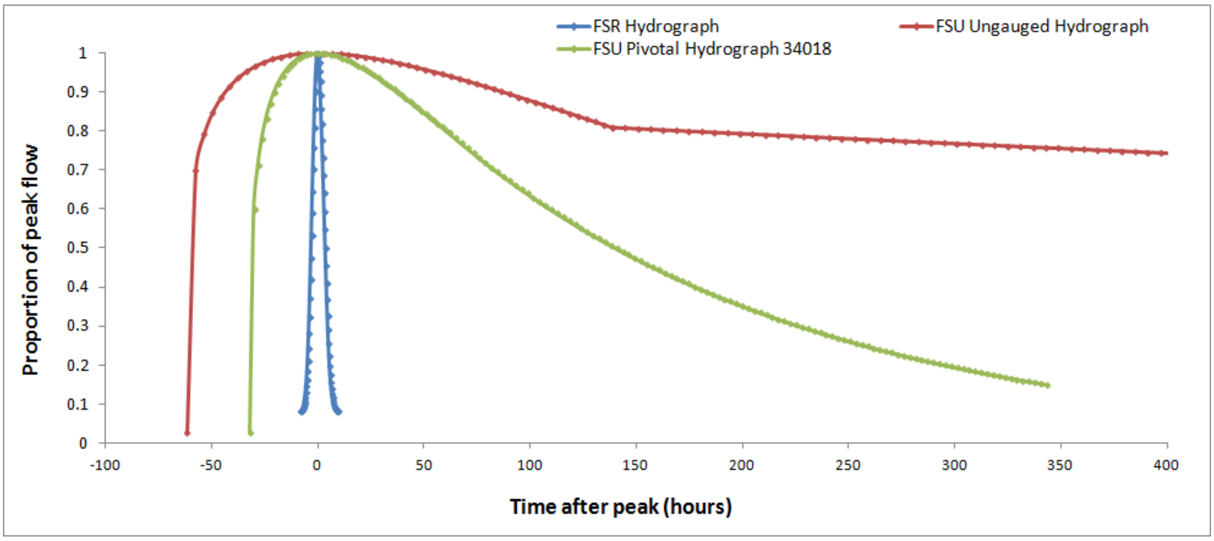


Figure E9-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

Review of Hydrograph Derivation and Shape

This site was not included in the Inception Hydrograph Width Analysis (HWA) and therefore there is no comparison between the derived hydrographs and the expected shape as with the locations for which flow data is available. This analysis focuses on the differences in hydrograph shape between the various methods tested.

FSR Rainfall-Runoff: This method produced a hydrograph with very steep rising and falling limbs compared to the FSU methods, with a time to peak of approximately 4 hours. Whilst the catchment is small (12.0 km^2), a large proportion of the catchment consists of Lough Rea, reducing the value of FARL to 0.499. This degree of attenuation is unlikely to be reflected in the quickly responding hydrograph produced by the FSR Rainfall-Runoff method. In addition, the symmetrical nature of the hydrograph does not account for the greater time taken for the channel to return to natural flows compared to the initial response to rainfall.

FSU Ungauged: The hydrograph estimated using this method describes a much slower catchment response than the FSR Rainfall-Runoff method, with the peak flow occurring approximately 60 hours into the event. Given the large degree of attenuation afforded by Lough Rea, this delayed response is a likely characteristic of the catchment during a flood event. However, the falling limb of this hydrograph is suspect given the large amount of time anticipated for the flows to return to natural levels. The low value for FARL (0.499) makes this exponential curve particularly shallow, however

given the catchment size, it is unlikely that such flows could be maintained at this level. This hydrograph is therefore unlikely to represent the complex hydrology at Lough Rea.

FSU Pivotal: The Pivotal Station 22071 was chosen following a thorough review of the gauges in Units of Management 29 and 34 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Rathgorgin (29001), Turlough (34018) and the software chosen Lough Leane (22071) were reviewed for their similarity with the ungauged site at Lough Rea. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away. However, given the unusual nature of the catchment, with a large degree of attenuation within a small, relatively steep, upland area, it is anticipated that a compromise will need to be made in finding the most hydrologically representative catchment for use as a Pivotal station.

Station 29001 has been excluded from further analysis given the large differences between key catchment descriptors at Rathgorgin and Lough Rea. Suitable descriptors include ARTDRAIN, SAAR and BFIsoil, however there are significant differences between all the remaining parameters, with FARL and S1085 in particular not representing the topography and attenuation in the Lough Rea catchment. A sample of these parameters is summarised below:

Catchment Descriptor	Lough Rea	Rathgorgin (29001)
ARTDRAIN (%)	0.00	0.01
SAAR	1134	1090
BFIsoil	0.727	0.581
FARL	0.499	0.998
S1085	6.85	2.22
URBEXT (%)	5.78	0.66

The catchment at Rathgorgin is therefore likely to be a poor representation of that at Lough Rea, such that the data should not be used to create a FSU Pivotal hydrograph at this site.

The software chosen site, 22071, improves upon the parameters for ARTDRAIN, S1085, FARL and BFIsoil at station 29001. The values at station 22071 are 0.00%, 7.76m/km, 0.730 and 0.638 respectively, better representing the rate at which water is routed through the catchment. However, the catchment area, rainfall and urban extent are not well represented by station 22071. This station has therefore been discounted in favour of station 34018 which has a more comparable set of descriptors for deriving the hydrograph shape parameters.

The FSU hydrograph, utilising station 34018 as a Pivotal station, has been plotted in Figure E9-1. Station 34018, whilst still relatively large at 95.4km^2, is smaller than the other options for a Pivotal station and has more comparable rainfall statistics to Lough Rea. The catchment upstream of Turlough is also described by an URBEXT value of 5.53 (compared to 5.78 at Lough Rea) and a BFIsoil value of 0.750 (0.727 at the subject site). However, the shallower gradient and increased arterial drainage in the catchment for station 34018 may cause the flows to respond differently between the candidate Pivotal station and the subject site. The method appears to produce a realistic hydrograph shape, with a steep rising limb due to the small catchment area followed by a delayed response due to attenuation of flows by upstream waterbodies. Whilst the falling limb is more realistic

than the FSU Ungauged hydrograph, it is still unlikely that a small catchment, such as Lough Rea, is able to produce floods of up to 350 hours duration, as illustrated in Figure E9-1.
Recommendations
The planned approach for flood estimation at Loughrea is the FSR rainfall-runoff method, with flood hydrographs routed through the lough using the hydraulic model. The FSR hydrograph shown above does not include flood routing, hence the short flood duration. The FSU hydrographs are very much more prolonged and produce a flood duration which is probably unrealistic given the small size of the catchment. Further tests of the hydrographs can be found in the main text of the report.

Design Hydrograph analysis summary sheet

Ungauged Site Grid Reference 138637 299815			Swinford @ Swinford		
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	5.42	FSU (Ungauged)	T_r	22.39
				C	30.70
				n	6.08
	Storm Duration (hours)	12.17	FSU (Pivotal site 27001)	T_r	17.72
				C	11.24
				n	5.00

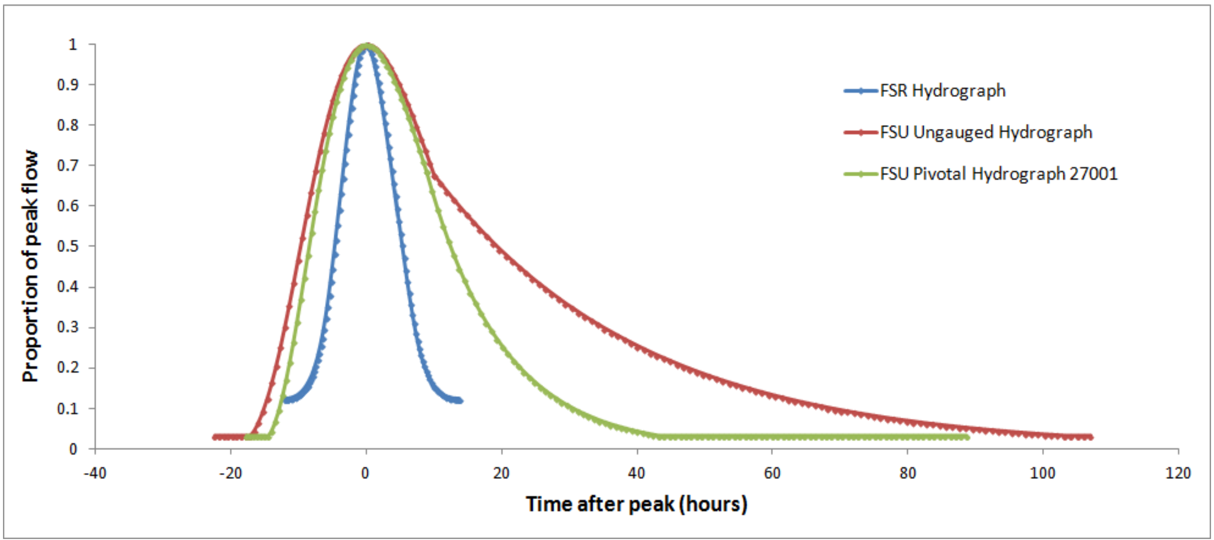


Figure E10-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

Review of Hydrograph Derivation and Shape

This ungauged site was not included in the Inception Hydrograph Width Analysis (HWA) and therefore there is no comparison between the derived hydrographs and the expected shape as with the gauged locations. This analysis focuses on the differences in hydrograph shape between the various methods tested.

FSR Rainfall-Runoff: This method produced a hydrograph with both rising and initial falling limbs that are steeper than the FSU hydrographs. It shows little sign of the asymmetry expected in flood hydrographs, with the catchment taking longer to return to natural flows than the time taken for peak flows to be reached in its response to a rainfall event.

FSU Ungauged: The hydrograph derived using this method appears to estimate a slower response of the catchment than the FSR Rainfall-Runoff method, with the peak flow being met approximately 17 hours into the event. This may be explained by attenuation in the catchment, with a FARL value of 0.933 increasing the response time of both the rising and falling limbs.

FSU Pivotal: The Pivotal Station 27001 was chosen following a detailed review of the gauges within Units of Management 34 and 30 and sites suggested by the Hydrograph Shape Generator (version 3) software supplied by OPW. The catchment descriptors at the local gauging stations Charlestown (34031), Curraghbonaun (34009), Ballyhaunis (30020), Turlough (34018) and Foxford (34003), in addition to Inch Bridge (27001) and the software chosen Aughnagross (16005) were reviewed for their

similarity with the ungauged site at Swinford. Those used in the derivation of T_r, C and n parameters were checked for similarity to the subject site, in addition to other characteristics which may influence the hydrograph shape, such as AREA, URBEXT, DRAIND and MSL. The distance between the subject site and the candidate Pivotal Stations was also taken into account, with local, hydrologically similar sites preferred over those situated further away.

Station 34018 has been discounted from further analysis given the large disparities between key catchment descriptors at this site and the subject location. Of particular note are the high values of MSL (23.738km compared to 9.897 at Swinford) and BFIsoil (0.750 compared to 0.462 at the subject site) which indicate station 34018 represents a larger, shallow gradient catchment with more permeable soils than the subject catchment. These characteristics would be likely to delay the response of the catchment to a rainfall event, increasing the time to peak of the flood hydrograph, which is not anticipated at the subject site. The remaining catchment descriptors are also unrepresentative of the catchment upstream of Swinford, ruling this site out for use as a Pivotal station.

Station 34003 also represents a catchment that is dissimilar to that upstream of Swinford. Whilst the indices for DRAIND and URBEXT are similar to those of the subject site, the values of 69.18km for MSL, 0.747 for BFIsoil and 0.961 for S1085 indicate this catchment is more similar to Station 34018 than the subject site. The catchment descriptor FARL also indicates a large degree of attenuation (FARL is 0.817 compared to 0.933) which is likely to overestimate that observed at Swinford. This site has therefore been discounted from further analysis.

The software chosen site, 16005, also has relatively high values of MSL and BFIsoil compared to the subject site, however they are more realistic than Stations 34018 and 34003. However, this station does not account for the attenuation expected at Swinford, with FARL given as 1.000. The remaining descriptors, including DRAIND, S1085 and URBEXT are similar to those at the subject site, however the distance of 170km between the sites suggests the remaining stations may be more suitable, Station 16005 has thus been removed from further analysis.

The catchments upstream of Stations 34031, 34009, 27001 and 30020 have catchment descriptors that are more consistent with those of the subject site. Station 34031 has indices for MSL and BFIsoil of 9.102km and 0.329 respectively; however the catchment has no reservoir attenuation and is steeper than the subject catchment. Rainfall is therefore anticipated to be routed quickly through the catchment, resulting in a shorter time to peak in the event hydrograph. This can be seen in Figure E10-2 where using this site as a Pivotal Station gives the steepest hydrograph. Station 30020 also has representative descriptors for MSL and URBEXT, however the high percentage of ARTDRAIN (19.37% compared to 0% at the subject site), shallow slope of 2.891m/km and permeable soils (BFIsoil 0.610) result in a hydrograph with a relatively steep rising limb but a slow return back to baseflow conditions. Using station 34009 as a Pivotal Station results in a hydrograph with a delayed response to rainfall, as seen in Figure E10-2. This is likely to be due to the shallow gradient of the 117km^2 catchment (S1085 is 3.33m/km), which, despite arterial drainage, routes flows relatively slowly to the gauging station. The subject catchment is much smaller (13.2km^2) and steeper (S1085 is 6.83m/km) and therefore using Station 34009 to create a pivotal hydrograph is not recommended.

Station 27001, whilst outside the UoM, appears to give the best fit of catchment descriptors at Swinford. The catchment has a similar slope and drainage density to the subject site (S1085 is 4.448m/km compared to 6.83 at Swinford) and has no arterial drainage or urban development. BFIsoil is also similar at 0.330 whilst a value of 0.987 for FARL indicates some a degree of attenuation similar to that of the catchment upstream of Swinford. Use of this site as a Pivotal Station results in a hydrograph shape that is steeper than that of 34009, but accounts for more attenuation and a more delayed catchment response than 34031 and 30020.

The distance between the catchment centroids of Station 27001 and the 13km^2 catchment at Swinford is approximately 3.8km, much smaller than the distance between the subject site and stations within UoM 34. Whilst site 27001 is therefore not located in the same UoM as the subject site, its centroid is

nearby and its catchment is similar to that at Swinford. This gauged location is therefore deemed the most suitable site as a Pivotal Station for Swinford.

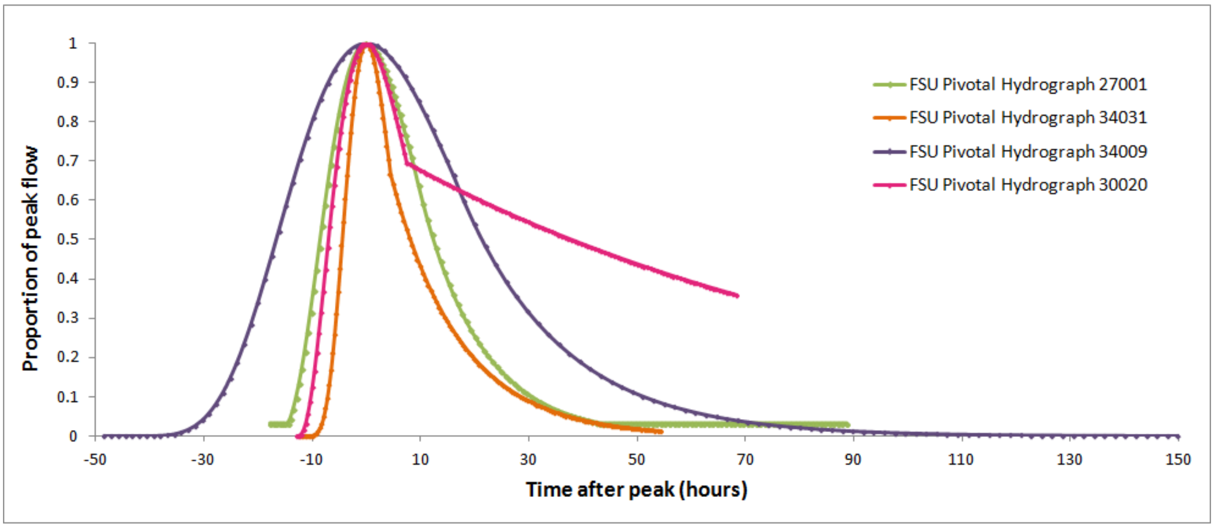


Figure E10-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations

Recommendations

The various versions of the FSU hydrograph are rather wider than the FSR hydrograph. Without any observed data it is not possible to give a definitive recommendation on which is the most realistic design hydrograph shape. Further tests are described in the main text of the report.

F Design flows for each HEP

Design flows given in the table below have been developed from the recommended design flows at gauging stations but these have been further modified in some areas through regional smoothing of the QMED adjustment factor. In addition, for all HEPs the flood growth curve was extended for AEPs lower than 1% using ratios from FSR rainfall-runoff method growth curves. Please refer to Appendix B Flood peak analysis for the preliminary recommended design flows at gauging stations prior to these additional modifications. A summary of the flood estimation process is given in Table 7-1 of the main report.

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
ABB_001	30_3424_2	141398	241757			P	GL	1.13	1.54	33.83	41.05	46.00	51.11	58.44	64.57	72.85	99.71
CLR_001	30_211_1	143170	263983	Weighted 30007 and 30020	0.77	P	GL	1.12	1.50	24.01	29.23	32.74	36.32	41.36	45.51	50.92	68.28
CLR_002	30_1916_1	143111	263892	Weighted 30007 and 30020	0.85	P	GL	1.12	1.50	43.02	52.38	59.08	66.21	76.80	85.95	96.09	128.62
CLR_003	30_3411_2	140649	262988	Weighted 30007 and 30020	0.86	P	GL	1.12	1.50	44.79	54.54	61.52	68.94	79.96	89.49	100.03	133.86
CLR_004	30_2794_1	140559	262957	Weighted 30007 and 30020	0.87	P	GL	1.12	1.50	49.56	60.58	68.46	76.84	89.27	100.00	111.79	149.60
CLR_005	30_2824_1	139968	259539	Weighted 30007 and 30020	0.89	P	GL	1.12	1.50	58.36	71.34	80.62	90.48	105.12	117.76	131.66	176.28
CLR_006	30_1924_5	141181	256130	Weighted 30007 and 30020	0.90	P	GL	1.12	1.49	58.04	71.28	80.74	90.81	105.73	118.63	132.57	177.26
CLR_007	30_1924_1	142056	253776	30007	0.91	P	GL	1.12	1.49	58.94	72.27	81.79	91.93	106.95	119.92	133.99	179.13
CLR_008	30_1924_13	141217	253666	Weighted 30004 and 30007	0.91	P	GL	1.12	1.49	58.96	72.30	81.83	91.97	107.00	119.98	134.05	179.18

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
CLR_009	30_2643_2	141540	253265	Weighted 30004 and 30007	0.91	P	GL	1.12	1.49	60.38	74.14	83.99	94.46	109.98	123.40	137.90	184.42
CLR_010	30_2832_1	141644	252863	Weighted 30004 and 30007	0.90	P	GL	1.12	1.49	65.08	79.91	90.53	101.81	118.55	133.01	148.64	198.78
CLR_011	30_2832_5	140518	252654	Weighted 30004 and 30007	0.90	P	GL	1.12	1.49	64.94	79.75	90.33	101.60	118.30	132.73	148.31	198.30
CLR_012	30_3012_10	140687	250095	Weighted 30004 and 30007	0.90	P	GL	1.12	1.50	67.59	81.92	92.03	102.68	118.31	131.67	147.19	197.00
CLR_013	30_1130_5	141850	248526	Weighted 30004 and 30007	0.90	P	GL	1.12	1.50	67.99	82.41	92.58	103.29	119.01	132.45	148.07	198.18
CLR_014	30_2902_1	141885	248356	Weighted 30004 and 30007	0.90	P	GL	1.12	1.50	69.92	85.22	96.02	107.40	124.10	138.38	154.73	207.19
CLR_015	30_2843_7	142890	244915	Weighted 30004 and 30007	0.90	P	GL	1.12	1.50	69.98	83.81	93.26	102.98	116.90	128.52	143.70	192.42
CLR_016	30_3003_1	142918	244827	Weighted 30004 and 30007	0.88	SS	G	1.12	1.50	84.01	108.80	123.84	138.29	156.94	170.88	191.08	255.93
CLR_017	30_3003_5	142664	243022	30004	0.88	SS	G	1.12	1.50	84.33	109.20	124.30	138.80	157.52	171.52	191.79	256.87
CLR_018	30_3003_9	141344	241769	Weighted 30004 and 30012	0.88	SS	G	1.12	1.50	84.61	109.57	124.71	139.27	158.05	172.09	192.43	257.73

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
CLR_019	30_557_1	141326	241712	Weighted 30004 and 30012	0.86	SS	G	1.12	1.50	107.09	138.68	157.85	176.27	200.05	217.82	243.87	327.63
CLR_020	30_557_11	141997	237558	Weighted 30004 and 30012	0.85	SS	G	1.12	1.50	106.25	137.60	156.62	174.89	198.48	216.12	241.88	324.69
CLR_021	30_557_21	141525	233537	Weighted 30004 and 30012	0.85	SS	G	1.12	1.50	110.82	143.51	163.34	182.41	207.01	225.40	252.39	339.20
CLR_022	30_248_2	139758	232888	Weighted 30004 and 30012	0.84	SS	G	1.12	1.51	118.50	153.46	174.67	195.05	221.36	241.03	270.05	363.43
CLR_023	30_248_9	137800	233362	Weighted 30004 and 30012	0.84	SS	G	1.12	1.51	119.79	155.12	176.57	197.17	223.76	243.65	272.97	367.35
CLR_024	30_249_2	137302	233237	30012	0.84	SS	G	1.12	1.50	120.06	155.48	176.97	197.62	224.27	244.20	273.35	367.03
CNF_001	30_2327_7	147730	272068			P	GL	1.12	1.53	7.15	8.76	9.82	10.89	12.37	13.57	15.26	20.70
CRB_001	30_2615_1	125417	230660	30061	1.56	P	GL	1.12	1.49	237.25	283.93	314.54	345.12	387.49	421.76	471.12	629.43
CRB_002	30_246_3	127222	228732	30061	1.56	P	GL	1.12	1.49	246.89	295.47	327.32	359.15	403.23	438.90	490.23	654.84
CRB_003	30_729_2	128239	227945	30061	1.56	P	GL	1.12	1.49	242.54	290.27	321.55	352.82	396.13	431.17	481.45	642.59
CRB_004	30_3419_5	129629	224898	30061	1.56	P	GL	1.12	1.49	245.52	293.83	325.50	357.15	400.99	436.46	487.40	650.70
CRT_001	30_3317_2	140646	262953			P	GL	1.12	1.52	7.81	9.74	11.06	12.40	14.31	15.90	17.85	24.17
CUR_001	30_2503_6	150484	278481	30020	0.71	P	GL	1.13	1.55	0.88	1.13	1.31	1.51	1.81	2.07	2.34	3.21
CUR_002	30_2503_7	150099	278680	30020	0.71	P	GL	1.13	1.55	0.95	1.22	1.41	1.61	1.92	2.19	2.47	3.39
CUR_003	30_2503_8	150061	279110	30020	0.71	P	GL	1.13	1.55	0.97	1.25	1.46	1.68	2.00	2.29	2.58	3.54
CUR_004	30_2503_9	149914	279439	30020	0.71	P	GL	1.13	1.55	0.98	1.27	1.48	1.70	2.03	2.32	2.62	3.60
CUR_005	30_30_2	149880	279630	30020	0.71	P	GL	1.13	1.55	1.07	1.36	1.57	1.79	2.13	2.41	2.72	3.73
DER_001	30_33_3	148648	277881			P	GL	1.13	1.56	2.52	3.18	3.67	4.19	4.97	5.65	6.39	8.80
DLG_000	30_2471_1	150068	280030	30020	0.71	P	GL	1.13	1.53	3.53	4.43	5.04	5.66	6.53	7.26	8.17	11.11

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
DLG_001	30_2471_3	149958	279786	30020	0.71	P	GL	1.13	1.53	3.55	4.48	5.10	5.74	6.65	7.40	8.33	11.32
DLG_002	30_2340_1	149855	279657	30020	0.71	P	GL	1.13	1.53	4.36	5.49	6.24	7.01	8.09	8.98	10.11	13.75
DLG_003	30_2340_2	149614	279434	30020	0.71	P	GL	1.12	1.53	4.23	5.28	5.98	6.68	7.67	8.48	9.54	12.96
DLG_004	30_2340_4	149201	278670	Weighted 30007 and 30020	0.71	P	GL	1.12	1.53	4.93	6.17	7.00	7.86	9.07	10.07	11.33	15.40
DLG_005	30_1566_3	148670	277894	Weighted 30007 and 30020	0.71	P	GL	1.13	1.53	5.38	6.73	7.64	8.57	9.90	10.99	12.37	16.83
DLG_006	30_2218_1	148661	277835	Weighted 30007 and 30020	0.71	P	GL	1.13	1.53	6.50	8.13	9.23	10.36	11.96	13.28	14.95	20.32
DLG_007	30_2428_1	149150	276295	Weighted 30007 and 30020	0.71	P	GL	1.12	1.53	7.01	8.67	9.80	10.96	12.60	13.97	15.70	21.31
DLG_008	30_2371_2	148117	274977	Weighted 30007 and 30020	0.72	P	GL	1.12	1.52	7.66	9.33	10.43	11.55	13.09	14.36	16.14	21.89
DLG_009	30_2367_4	148307	273835	Weighted 30007 and 30020	0.72	P	GL	1.12	1.52	8.03	9.75	10.89	12.04	13.64	14.94	16.79	22.73
DLG_010	30_644_1	148307	273802	Weighted 30007 and 30020	0.72	P	GL	1.12	1.52	10.75	13.12	14.71	16.31	18.57	20.41	22.93	31.02
DLG_011	30_644_3	148064	273143	Weighted 30007 and 30020	0.73	P	GL	1.12	1.52	10.78	13.17	14.76	16.36	18.63	20.48	22.99	31.09

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
DLG_012	30_680_1	148094	273101	Weighted 30007 and 30020	0.73	P	GL	1.12	1.52	11.64	14.06	15.66	17.27	19.50	21.33	23.94	32.37
DLG_013	30_519_2	147697	272083	Weighted 30007 and 30020	0.73	P	GL	1.12	1.52	12.03	14.55	16.18	17.80	20.01	21.79	24.45	33.03
DLG_014	30_214_1	147716	272037	Weighted 30007 and 30020	0.74	P	GL	1.12	1.52	16.34	19.95	22.33	24.71	28.03	30.73	34.48	46.56
DLG_015	30_214_3	147441	271537	Weighted 30007 and 30020	0.74	P	GL	1.12	1.51	16.34	19.95	22.32	24.71	28.03	30.73	34.47	46.53
DLG_016	30_215_1	147421	271492	Weighted 30007 and 30020	0.75	P	GL	1.12	1.51	18.78	22.74	25.28	27.79	31.21	33.93	38.06	51.37
DLG_017	30_3045_2	146345	269427	Weighted 30007 and 30020	0.76	P	GL	1.12	1.51	20.25	24.64	27.55	30.48	34.56	37.88	42.46	57.21
DLG_018	30_1115_1	144635	267679	Weighted 30007 and 30020	0.76	P	GL	1.12	1.51	21.91	26.51	29.57	32.67	37.02	40.58	45.46	61.16
DLG_019	30_1515_1	142300	267645	Weighted 30007 and 30020	0.77	P	GL	1.12	1.50	24.10	28.90	31.98	35.03	39.19	42.50	47.59	63.93
DRP_001	30_3404_8	145493	252950			P	GL	1.12	1.52	3.20	4.04	4.63	5.24	6.13	6.89	7.73	10.46
DRP_002	30_3404_1	144006	252172			P	GL	1.12	1.52	3.75	4.74	5.43	6.15	7.20	8.08	9.08	12.31
DVL_002	30_2503_9	150331	279356	30020	0.71	P	GL	1.13	1.55	0.04	0.05	0.06	0.07	0.08	0.09	0.11	0.14
DVL_003	30_2340_2	150174	279440	30020	0.71	P	GL	1.13	1.55	0.05	0.07	0.08	0.10	0.12	0.13	0.15	0.21
DVL_004	30_2503_9	149935	279481	30020	0.71	P	GL	1.13	1.55	0.07	0.10	0.12	0.14	0.16	0.19	0.21	0.29

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
FMN_001	30_3131_5	147400	271526			P	GL	1.12	1.52	3.49	4.35	4.93	5.53	6.38	7.09	7.96	10.78
GRN_001	30_2925_5	144139	246625			P	GL	1.12	1.51	23.25	27.69	30.58	33.44	37.36	40.51	45.43	61.28
GRN_002	30_2925_1	142938	244951			P	GL	1.12	1.51	22.06	26.77	29.92	33.11	37.60	41.29	46.30	62.46
KLB_001	30_2410_7	141184	253620			P	GL	1.15	1.63	1.21	1.59	1.87	2.17	2.63	3.03	3.48	4.95
LUG_001	30_2394_6	148091	273149			P	GL	1.13	1.53	1.29	1.65	1.90	2.18	2.59	2.96	3.33	4.54
NAN_001	30_2339_3	145203	251728			P	GL	1.12	1.50	0.25	0.33	0.39	0.45	0.54	0.63	0.70	0.94
NAN_002	30_2339_4	144912	251673			P	GL	1.13	1.53	0.41	0.54	0.64	0.74	0.90	1.04	1.17	1.59
NAN_003	30_391_1	144872	251652			P	GL	1.13	1.53	0.67	0.88	1.03	1.19	1.42	1.62	1.82	2.48
NAN_004	30_391_2	144456	251880			P	GL	1.13	1.54	0.88	1.15	1.34	1.55	1.85	2.11	2.38	3.25
NAN_005	30_391_3	143992	252124			P	GL	1.13	1.54	0.95	1.25	1.46	1.69	2.02	2.30	2.59	3.54
NAN_006	30_1128_1	143920	252167			P	GL	1.12	1.52	4.70	5.91	6.74	7.61	8.85	9.90	11.13	15.10
NAN_007	30_1128_7	141737	252965			P	GL	1.13	1.53	5.75	7.23	8.26	9.32	10.85	12.14	13.66	18.61
ONR_001	30_291_2	110627	242273	30101	1.25	P	GL	1.12	1.51	23.61	28.08	31.11	34.20	38.60	42.24	47.34	63.73
ONR_002	30_3142_2	110693	242516	30101	1.25	P	GL	1.12	1.51	25.01	29.74	32.95	36.22	40.88	44.74	50.13	67.47
ONR_003	30_3142_5	111983	242832	30101	1.25	P	GL	1.12	1.51	25.36	30.16	33.40	36.73	41.45	45.36	50.82	68.39
ONR_004	30_3396_1	112049	242868	30101	1.25	P	GL	1.12	1.51	26.56	31.59	34.99	38.47	43.41	47.51	53.24	71.64
ONR_005	30_3396_5	112947	243678	30101	1.25	P	GL	1.12	1.51	27.19	32.34	35.82	39.39	44.45	48.64	54.50	73.35
RNK_001	30_2244_5	142189	250749			P	GL	1.15	1.67	0.92	1.21	1.42	1.65	2.01	2.32	2.68	3.86
SNK_001	30_1911_1	143176	263923	30020	0.71	P	GL	1.12	1.50	17.50	21.23	23.85	26.59	30.59	33.99	38.00	50.89
SUL_001	30_2487_1	141636	251671			P	GL	1.19	1.84	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.05
SUL_002	30_2487_1	141847	251389			P	GL	1.19	1.84	0.18	0.24	0.28	0.32	0.40	0.46	0.55	0.85
SUL_003	30_2487_2	142160	250762			P	GL	1.19	1.81	0.55	0.73	0.86	1.00	1.22	1.41	1.68	2.56
SUL_004	30_2645_1	142204	250659			P	GL	1.16	1.70	1.48	1.94	2.28	2.66	3.23	3.75	4.35	6.37
SUL_005	30_2645_2	142083	250262			P	GL	1.16	1.69	1.49	1.95	2.29	2.67	3.24	3.76	4.36	6.35
SUL_006	30_2645_3	141897	249824			P	GL	1.16	1.67	1.41	1.85	2.17	2.53	3.06	3.54	4.09	5.93
SUL_007	30_2645_4	141950	249332			P	GL	1.15	1.66	1.40	1.84	2.17	2.52	3.06	3.54	4.08	5.87
SUL_008	30_2645_6	141891	248480			P	GL	1.15	1.66	1.65	2.17	2.54	2.95	3.58	4.13	4.76	6.87
TNP_001	30_2339_3	145285	251436			P	GL	1.12	1.50	0.20	0.26	0.31	0.36	0.44	0.50	0.56	0.76
TNP_002	30_2339_4	144915	251603			P	GL	1.13	1.53	0.42	0.56	0.66	0.77	0.93	1.07	1.21	1.64
TNS_004	30_3141_3	111979	242854			P	GL	1.13	1.54	0.79	0.98	1.12	1.26	1.48	1.66	1.88	2.56

HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m^3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
TNW_001	30_3141_1	110816	242976			P	GL	1.13	1.54	0.47	0.59	0.67	0.75	0.88	0.98	1.11	1.51
TNW_002	30_3141_1	111256	242876			P	GL	1.13	1.54	0.58	0.72	0.81	0.91	1.06	1.19	1.34	1.82
TNW_003	30_3141_2	111658	242896			P	GL	1.13	1.54	0.70	0.87	0.99	1.12	1.31	1.48	1.66	2.27
TUL_001	30_40_2	148333	273827			P	GL	1.13	1.54	5.12	6.40	7.27	8.16	9.41	10.46	11.78	16.07

HEP label	Future peak flow - MRFS (m^3/s) by AEP(%)								Future peak flow - HEFS (m^3/s) by AEP(%)							
	50	20	10	5	2	1	0.5	0.1	50	20	10	5	2	1	0.5	0.1
ABB_001	40.59	49.25	55.20	61.33	70.12	77.49	87.42	119.65	43.98	53.36	59.79	66.44	75.97	83.95	94.71	129.62
CLR_001	28.86	35.15	39.37	43.66	49.72	54.72	61.22	82.08	31.30	38.11	42.69	47.34	53.92	59.33	66.38	89.01
CLR_002	51.70	62.95	71.00	79.57	92.29	103.29	115.47	154.57	56.05	68.25	76.98	86.27	100.06	111.98	125.19	167.57
CLR_003	53.83	65.54	73.93	82.85	96.09	107.54	120.21	160.87	58.35	71.05	80.14	89.82	104.18	116.59	130.32	174.40
CLR_004	59.55	72.79	82.26	92.33	107.26	120.16	134.32	179.76	64.56	78.91	89.17	100.09	116.28	130.26	145.61	194.87
CLR_005	70.13	85.73	96.88	108.74	126.32	141.51	158.22	211.84	76.03	92.94	105.02	117.88	136.94	153.41	171.53	229.65
CLR_006	69.75	85.65	97.02	109.12	127.05	142.55	159.29	213.00	75.61	92.85	105.18	118.29	137.73	154.53	172.69	230.91
CLR_007	70.83	86.85	98.30	110.47	128.53	144.12	161.03	215.27	76.79	94.15	106.57	119.77	139.34	156.24	174.58	233.38
CLR_008	70.86	86.89	98.34	110.53	128.59	144.19	161.10	215.34	76.82	94.20	106.62	119.82	139.41	156.32	174.65	233.45
CLR_009	72.56	89.10	100.93	113.52	132.18	148.30	165.72	221.63	78.66	96.60	109.42	123.07	143.30	160.77	179.67	240.27
CLR_010	78.29	96.14	108.90	122.48	142.61	160.01	178.81	239.13	84.92	104.28	118.12	132.85	154.69	173.56	193.95	259.38
CLR_011	78.12	95.94	108.67	122.22	142.31	159.67	178.42	238.55	84.74	104.06	117.88	132.57	154.36	173.19	193.53	258.75
CLR_012	81.30	98.55	110.71	123.52	142.32	158.39	177.06	236.97	88.19	106.89	120.08	133.98	154.36	171.80	192.05	257.03
CLR_013	81.79	99.13	111.36	124.25	143.16	159.33	178.11	238.39	88.71	107.52	120.79	134.77	155.28	172.81	193.18	258.57
CLR_014	84.14	102.55	115.54	129.23	149.33	166.52	186.19	249.32	91.27	111.25	125.34	140.20	162.00	180.65	201.98	270.46
CLR_015	84.20	100.83	112.20	123.90	140.65	154.62	172.89	231.51	91.33	109.38	121.71	134.40	152.57	167.73	187.54	251.13
CLR_016	101.03	130.84	148.92	166.30	188.73	205.50	229.79	307.78	109.57	141.89	161.51	180.35	204.68	222.86	249.21	333.78
CLR_017	101.41	131.32	149.48	166.92	189.43	206.26	230.64	308.91	109.98	142.42	162.11	181.02	205.44	223.69	250.13	335.01
CLR_018	101.75	131.76	149.97	167.47	190.06	206.95	231.41	309.93	110.34	142.89	162.64	181.62	206.12	224.43	250.96	336.12
CLR_019	128.72	166.69	189.73	211.87	240.44	261.81	293.12	393.79	139.56	180.73	205.71	229.71	260.69	283.86	317.80	426.95
CLR_020	127.71	165.38	188.24	210.20	238.56	259.75	290.72	390.25	138.46	179.30	204.09	227.90	258.64	281.62	315.20	423.11
CLR_021	133.20	172.49	196.34	219.25	248.82	270.93	303.37	407.71	144.42	187.02	212.87	237.71	269.78	293.75	328.92	442.05
CLR_022	142.42	184.43	209.93	234.42	266.04	289.68	324.55	436.78	154.41	199.96	227.60	254.15	288.43	314.06	351.87	473.54
CLR_023	143.96	186.43	212.20	236.97	268.93	292.82	328.07	441.50	156.08	202.13	230.06	256.91	291.56	317.47	355.68	478.66
CLR_024	144.29	186.86	212.69	237.51	269.54	293.49	328.52	441.11	156.44	202.59	230.59	257.50	292.23	318.19	356.17	478.24
CNF_001	8.58	10.51	11.79	13.06	14.84	16.29	18.31	24.84	9.30	11.39	12.77	14.15	16.08	17.64	19.83	26.91
CRB_001	285.02	341.10	377.87	414.61	465.51	506.68	565.98	756.16	308.94	369.73	409.59	449.41	504.58	549.21	613.49	819.64
CRB_002	296.60	354.96	393.22	431.46	484.42	527.27	588.94	786.69	321.50	384.76	426.23	467.68	525.09	571.53	638.38	852.72
CRB_003	291.39	348.72	386.31	423.87	475.91	518.00	578.40	772.00	315.85	378.00	418.74	459.46	515.86	561.49	626.96	836.81
CRB_004	295.19	353.27	391.35	429.40	482.11	524.75	586.00	782.33	320.09	383.07	424.36	465.63	522.79	569.02	635.44	848.33
CRT_001	9.37	11.69	13.27	14.88	17.17	19.07	21.43	29.01	10.16	12.67	14.37	16.12	18.60	20.66	23.21	31.42
CUR_001	1.06	1.36	1.58	1.81	2.17	2.48	2.80	3.85	1.15	1.47	1.71	1.96	2.35	2.69	3.04	4.17
CUR_002	1.14	1.46	1.69	1.94	2.31	2.62	2.97	4.07	1.23	1.58	1.83	2.10	2.50	2.84	3.21	4.41

HEP label	Future peak flow - MRFS (m^3/s) by AEP(%)								Future peak flow - HEFS (m^3/s) by AEP(%)							
	50	20	10	5	2	1	0.5	0.1	50	20	10	5	2	1	0.5	0.1
CUR_003	1.16	1.51	1.75	2.01	2.40	2.74	3.10	4.25	1.26	1.63	1.90	2.18	2.60	2.97	3.36	4.60
CUR_004	1.19	1.54	1.79	2.06	2.46	2.81	3.17	4.36	1.30	1.68	1.95	2.24	2.68	3.06	3.45	4.74
CUR_005	1.29	1.65	1.90	2.17	2.57	2.92	3.30	4.52	1.41	1.79	2.07	2.36	2.80	3.18	3.59	4.92
DER_001	3.03	3.82	4.40	5.03	5.97	6.79	7.68	10.56	3.28	4.14	4.77	5.45	6.47	7.36	8.32	11.45
DLG_000	4.24	5.32	6.04	6.79	7.84	8.71	9.80	13.33	4.59	5.76	6.55	7.35	8.49	9.43	10.62	14.44
DLG_001	4.26	5.37	6.13	6.89	7.98	8.88	9.99	13.58	4.62	5.82	6.64	7.47	8.65	9.62	10.82	14.71
DLG_002	5.24	6.59	7.50	8.42	9.73	10.80	12.15	16.53	5.69	7.15	8.14	9.14	10.55	11.71	13.18	17.92
DLG_003	5.09	6.35	7.19	8.04	9.23	10.21	11.48	15.60	5.53	6.89	7.80	8.72	10.01	11.07	12.45	16.92
DLG_004	5.98	7.48	8.50	9.53	11.01	12.22	13.75	18.69	6.52	8.15	9.26	10.39	11.99	13.32	14.98	20.36
DLG_005	6.53	8.17	9.27	10.41	12.01	13.34	15.02	20.43	7.11	8.90	10.11	11.34	13.09	14.54	16.36	22.25
DLG_006	7.87	9.85	11.19	12.55	14.49	16.09	18.11	24.61	8.57	10.72	12.17	13.66	15.77	17.51	19.71	26.79
DLG_007	8.49	10.50	11.86	13.26	15.25	16.90	19.00	25.79	9.23	11.42	12.90	14.43	16.59	18.39	20.67	28.05
DLG_008	9.26	11.28	12.62	13.96	15.84	17.36	19.51	26.47	10.07	12.27	13.72	15.18	17.22	18.88	21.22	28.79
DLG_009	9.70	11.78	13.16	14.55	16.48	18.06	20.29	27.47	10.54	12.81	14.31	15.82	17.92	19.63	22.05	29.86
DLG_010	12.97	15.83	17.74	19.68	22.39	24.62	27.65	37.42	14.08	17.19	19.27	21.37	24.32	26.74	30.03	40.64
DLG_011	13.01	15.88	17.79	19.74	22.46	24.70	27.73	37.50	14.12	17.24	19.33	21.43	24.39	26.82	30.11	40.72
DLG_012	14.04	16.95	18.88	20.81	23.51	25.71	28.86	39.02	15.24	18.41	20.50	22.60	25.53	27.91	31.34	42.37
DLG_013	14.50	17.54	19.50	21.45	24.12	26.26	29.47	39.81	15.74	19.04	21.17	23.29	26.19	28.51	31.99	43.22
DLG_014	19.67	24.01	26.87	29.75	33.74	36.99	41.50	56.04	21.34	26.05	29.16	32.27	36.61	40.13	45.03	60.81
DLG_015	19.67	24.01	26.87	29.74	33.74	36.98	41.49	56.00	21.34	26.05	29.15	32.27	36.60	40.13	45.01	60.76
DLG_016	22.60	27.36	30.42	33.43	37.55	40.83	45.80	61.81	24.52	29.68	33.00	36.27	40.73	44.29	49.68	67.05
DLG_017	24.35	29.64	33.14	36.66	41.57	45.57	51.08	68.81	26.42	32.15	35.94	39.76	45.09	49.42	55.40	74.63
DLG_018	26.34	31.88	35.56	39.29	44.52	48.80	54.67	73.54	28.57	34.57	38.57	42.61	48.28	52.92	59.29	79.76
DLG_019	28.98	34.75	38.46	42.12	47.12	51.11	57.22	76.86	31.43	37.68	41.70	45.67	51.09	55.42	62.05	83.35
DRP_001	3.84	4.86	5.56	6.30	7.37	8.28	9.30	12.58	4.17	5.27	6.03	6.83	7.99	8.98	10.08	13.64
DRP_002	4.52	5.72	6.55	7.41	8.67	9.74	10.94	14.83	4.91	6.21	7.11	8.05	9.42	10.57	11.88	16.10
DVL_002	0.04	0.06	0.07	0.08	0.10	0.11	0.13	0.17	0.05	0.06	0.08	0.09	0.11	0.12	0.14	0.19
DVL_003	0.06	0.09	0.10	0.12	0.14	0.16	0.18	0.25	0.07	0.09	0.11	0.13	0.15	0.18	0.20	0.27
DVL_004	0.09	0.12	0.14	0.16	0.20	0.23	0.26	0.36	0.10	0.13	0.15	0.18	0.22	0.25	0.28	0.39
FMN_001	4.19	5.22	5.92	6.63	7.66	8.51	9.56	12.94	4.54	5.65	6.41	7.19	8.30	9.22	10.35	14.01
GRN_001	27.89	33.23	36.69	40.12	44.83	48.61	54.52	73.54	30.22	36.00	39.75	43.47	48.57	52.66	59.06	79.67
GRN_002	26.47	32.13	35.91	39.74	45.12	49.54	55.56	74.95	28.67	34.80	38.90	43.05	48.88	53.67	60.19	81.20
KLB_001	1.46	1.91	2.24	2.60	3.15	3.64	4.17	5.94	1.58	2.07	2.43	2.82	3.42	3.94	4.52	6.43

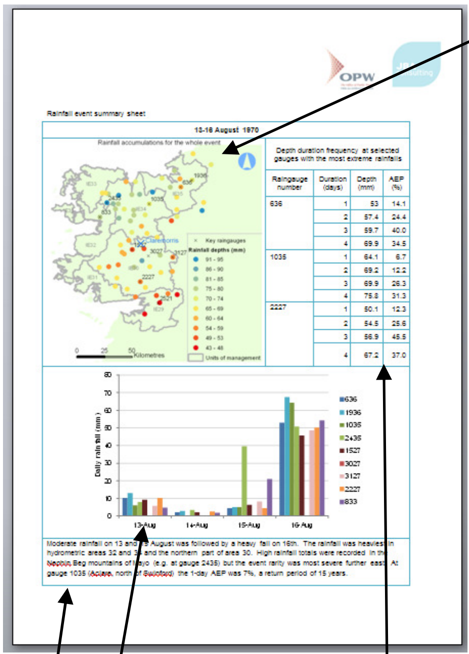
HEP label	Future peak flow - MRFS (m^3/s) by AEP(%)								Future peak flow - HEFS (m^3/s) by AEP(%)							
	50	20	10	5	2	1	0.5	0.1	50	20	10	5	2	1	0.5	0.1
LUG_001	1.55	1.97	2.28	2.61	3.11	3.55	4.00	5.44	1.68	2.14	2.47	2.83	3.37	3.84	4.33	5.90
NAN_001	0.29	0.39	0.46	0.54	0.65	0.75	0.84	1.13	0.32	0.42	0.50	0.58	0.71	0.82	0.91	1.23
NAN_002	0.49	0.65	0.76	0.89	1.08	1.25	1.40	1.91	0.53	0.70	0.83	0.96	1.17	1.35	1.52	2.07
NAN_003	0.81	1.06	1.24	1.43	1.71	1.95	2.19	2.98	0.88	1.15	1.34	1.55	1.85	2.11	2.38	3.23
NAN_004	1.06	1.39	1.63	1.88	2.25	2.56	2.89	3.94	1.16	1.52	1.77	2.05	2.45	2.79	3.15	4.30
NAN_005	1.16	1.52	1.78	2.05	2.45	2.80	3.15	4.30	1.27	1.66	1.94	2.24	2.68	3.05	3.44	4.69
NAN_006	5.67	7.13	8.13	9.18	10.69	11.95	13.44	18.22	6.16	7.74	8.84	9.97	11.61	12.99	14.60	19.79
NAN_007	7.01	8.82	10.06	11.35	13.22	14.79	16.65	22.67	7.65	9.62	10.98	12.39	14.43	16.14	18.17	24.75
ONR_001	28.34	33.70	37.33	41.04	46.32	50.69	56.81	76.47	30.70	36.51	40.44	44.46	50.18	54.91	61.54	82.85
ONR_002	30.01	35.69	39.54	43.47	49.05	53.68	60.16	80.96	32.51	38.66	42.83	47.09	53.14	58.16	65.17	87.71
ONR_003	30.46	36.23	40.13	44.13	49.80	54.50	61.06	82.17	33.02	39.27	43.50	47.83	53.98	59.07	66.19	89.07
ONR_004	31.92	37.96	42.05	46.23	52.17	57.09	63.98	86.10	34.60	41.15	45.58	50.12	56.56	61.90	69.36	93.34
ONR_005	32.72	38.92	43.11	47.40	53.49	58.53	65.59	88.27	35.50	42.22	46.77	51.42	58.03	63.50	71.16	95.76
RNK_001	1.13	1.48	1.74	2.03	2.46	2.85	3.29	4.75	1.24	1.63	1.91	2.22	2.70	3.12	3.60	5.20
SNK_001	21.02	25.51	28.65	31.94	36.75	40.83	45.66	61.14	22.79	27.65	31.05	34.62	39.83	44.26	49.49	66.27
SUL_001	0.01	0.02	0.02	0.02	0.03	0.03	0.04	0.06	0.01	0.02	0.02	0.02	0.03	0.03	0.04	0.06
SUL_002	0.23	0.30	0.35	0.41	0.51	0.59	0.70	1.08	0.26	0.34	0.40	0.46	0.56	0.65	0.78	1.20
SUL_003	0.70	0.92	1.09	1.27	1.55	1.80	2.13	3.26	0.78	1.03	1.21	1.41	1.72	2.00	2.37	3.63
SUL_004	1.84	2.41	2.84	3.30	4.02	4.66	5.41	7.92	2.03	2.66	3.13	3.64	4.43	5.14	5.96	8.73
SUL_005	1.84	2.41	2.84	3.30	4.02	4.66	5.40	7.87	2.03	2.65	3.12	3.64	4.43	5.13	5.95	8.66
SUL_006	1.74	2.28	2.69	3.12	3.79	4.38	5.06	7.33	1.91	2.51	2.95	3.43	4.17	4.82	5.57	8.06
SUL_007	1.73	2.27	2.68	3.11	3.78	4.36	5.03	7.24	1.90	2.50	2.94	3.42	4.15	4.79	5.53	7.96
SUL_008	2.04	2.67	3.13	3.64	4.41	5.08	5.87	8.46	2.24	2.93	3.44	3.99	4.83	5.58	6.44	9.28
TNP_001	0.24	0.31	0.37	0.43	0.52	0.60	0.68	0.91	0.26	0.34	0.40	0.47	0.57	0.65	0.73	0.98
TNP_002	0.50	0.67	0.79	0.92	1.11	1.29	1.45	1.97	0.54	0.72	0.85	1.00	1.21	1.39	1.57	2.13
TNS_004	0.95	1.19	1.35	1.53	1.79	2.02	2.27	3.10	1.04	1.29	1.47	1.66	1.95	2.19	2.47	3.38
TNW_001	0.57	0.70	0.80	0.90	1.05	1.18	1.33	1.81	0.62	0.76	0.87	0.98	1.14	1.28	1.44	1.96
TNW_002	0.70	0.86	0.98	1.10	1.27	1.42	1.60	2.19	0.76	0.93	1.06	1.19	1.38	1.54	1.74	2.37
TNW_003	0.84	1.04	1.19	1.34	1.57	1.77	2.00	2.72	0.91	1.13	1.29	1.46	1.70	1.92	2.16	2.95
TUL_001	6.14	7.68	8.72	9.79	11.30	12.55	14.14	19.29	6.65	8.32	9.45	10.60	12.24	13.59	15.32	20.90

G Analysis of rainfall data

Introduction to Rainfall event summary sheets

This appendix provides results from analysis of rainfall events. Most of the analysis has been carried out using daily rainfall data as there are very few sub-daily gauges in the study area. However, some more simplified sheets show analysis of sub-daily data to aid in understanding the characteristics of short-duration rainfall events.

Information provided in the summary sheets



Map of rainfall depths

The map shows the total accumulated rainfall for the range of dates given in the heading of the sheet. Gauges included on the map are those that are within or near to catchments in the initial list of Areas for Further Assessment (AFAs) provided at the start of the project. A small number of extra AFAs in other catchments were identified during the flood risk review, but this was completed after the rainfall analysis had been carried out.

The map identifies ten key gauges, spread throughout the study area, for which long records are available.

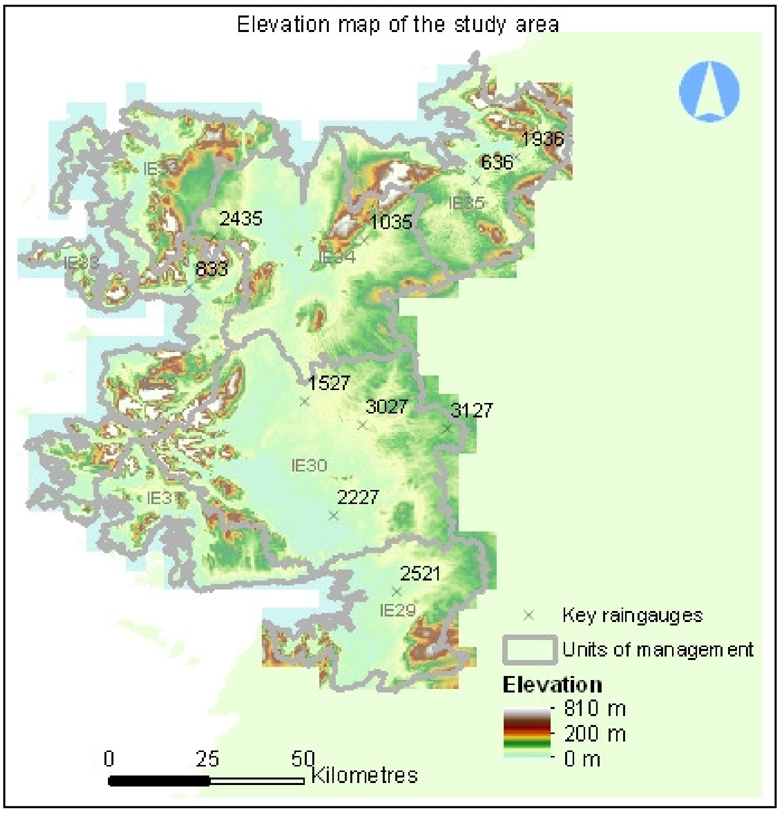
In interpreting the map it is important to bear in mind the general tendency for higher rainfall in the upland areas. The map below shows the topography of the area in relation to the key raingauge locations.

Time series

Series of daily rainfalls at each of the key gauges for which data is available

Commentary

Comments on the characteristics of the event, including any synoptic information available from Met Éireann reports.

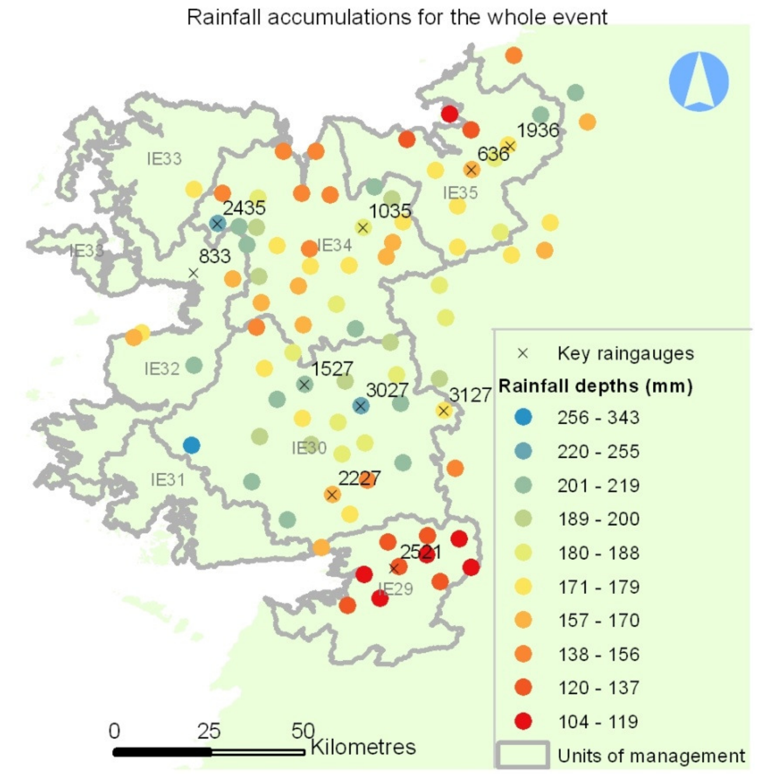


Depth duration frequency analysis

Table of rainfall depths and corresponding annual exceedance probabilities (AEPs) for the maximum rainfall accumulated over a range of durations at selected raingauges. The gauges included in this analysis are those where the rainfall was most notable, i.e. the AEPs were the lowest. The durations have been chosen to be appropriate to the nature of the event, with up to 14 days used for prolonged periods of rainfall. AEPs are calculated from the FSU rainfall frequency statistics.

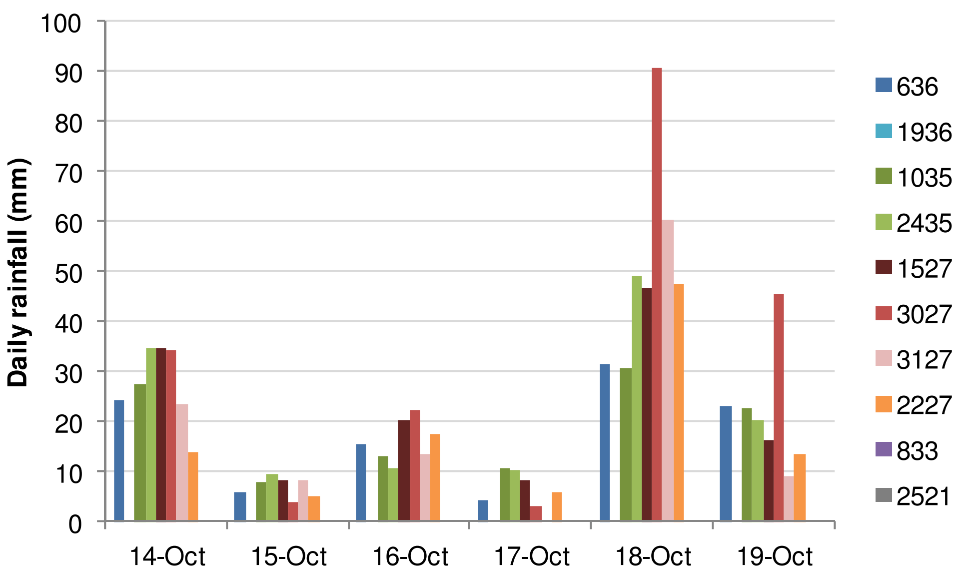
Rainfall event summary sheet

14 to 19 October 1954



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
1527	1	46.8	31.3
	2	63.3	20.0
	4	92.3	10.0
	6	135.6	1.8
3027	1	90.8	1.4
	2	136.3	0.3
	4	161.9	0.2
	6	200	0.13
3127	1	60.3	7.1
	2	69.6	8.3
	4	83.1	12.0
	6	115	4.3

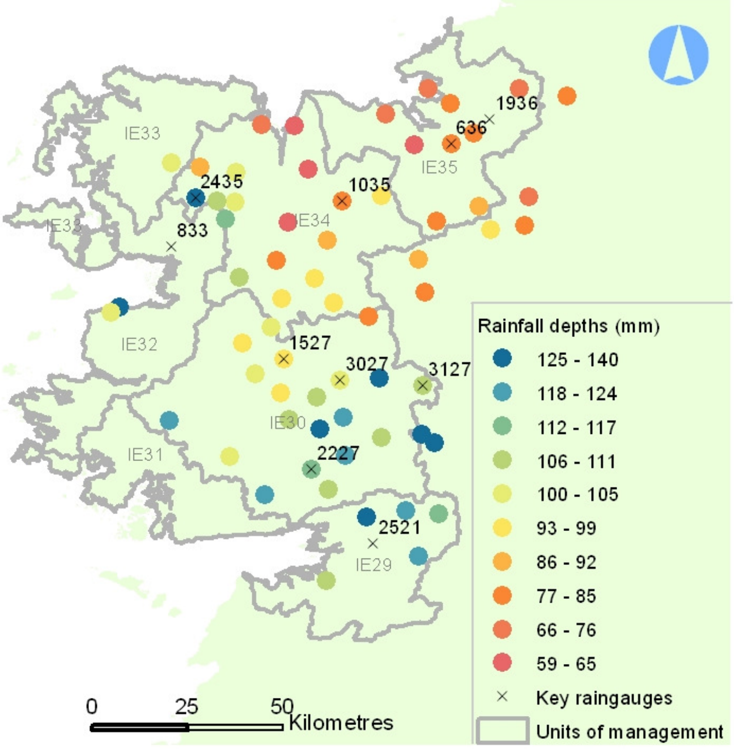


Several days of rainfall culminated in large daily totals on 18 October 1954. The rain affected the whole of the Western RBD although it was most severe in hydrometric area 30, with an AEP below 1% at gauge 3027, Milltown (between Tuam and Claremorris), for durations over 1 day. For a duration of 6 days, the AEP at Milltown was as low as 0.13% (a return period of 800 years).

Rainfall event summary sheet

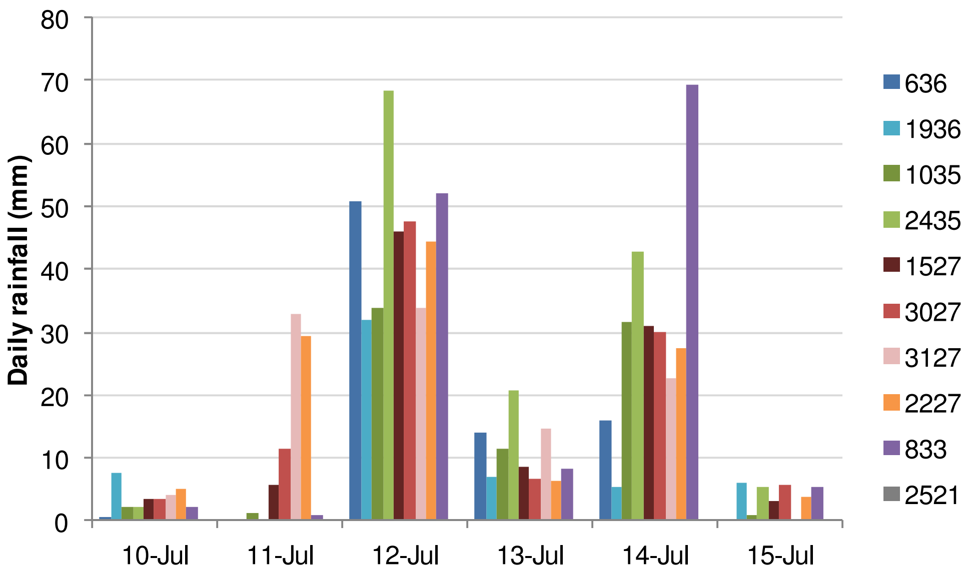
10 to 15 July 1961

Rainfall accumulations for the whole event



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
3127	1	33.9	59
	2	66.9	10
	3	81.7	7
	4	104.4	3
2227	1	44.3	26
	2	73.7	3
	3	80.1	5
	4	107.5	1
833	1	69.4	15
	2	77.8	24
	3	129.8	3
	4	135.3	5

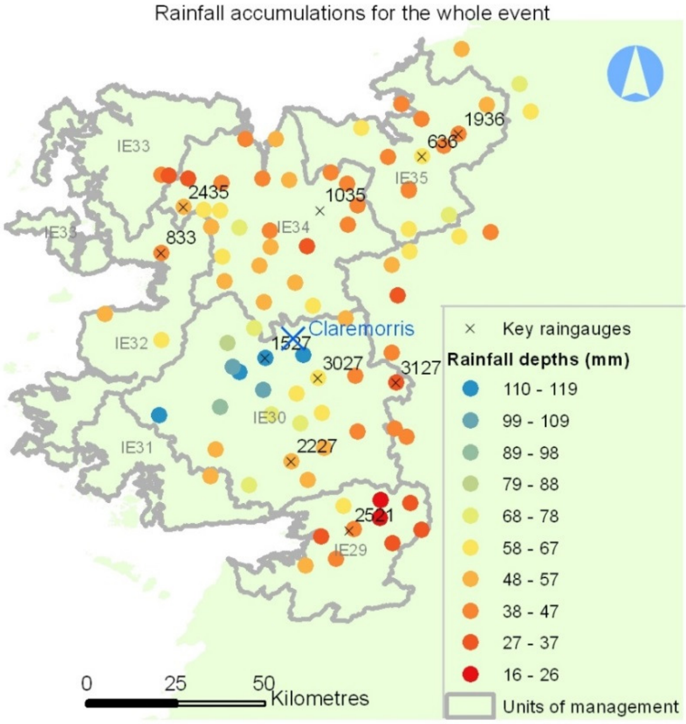


This summer event affected the whole of the Western RBD, although the largest 6-day accumulations were in hydrometric areas 29 and 30, in the area between Athenry and Claremorris. The majority of the rainfall fell on 12 and 14 July. AEPs were as low as 1% over a duration of 4 days.

Rainfall event summary sheet

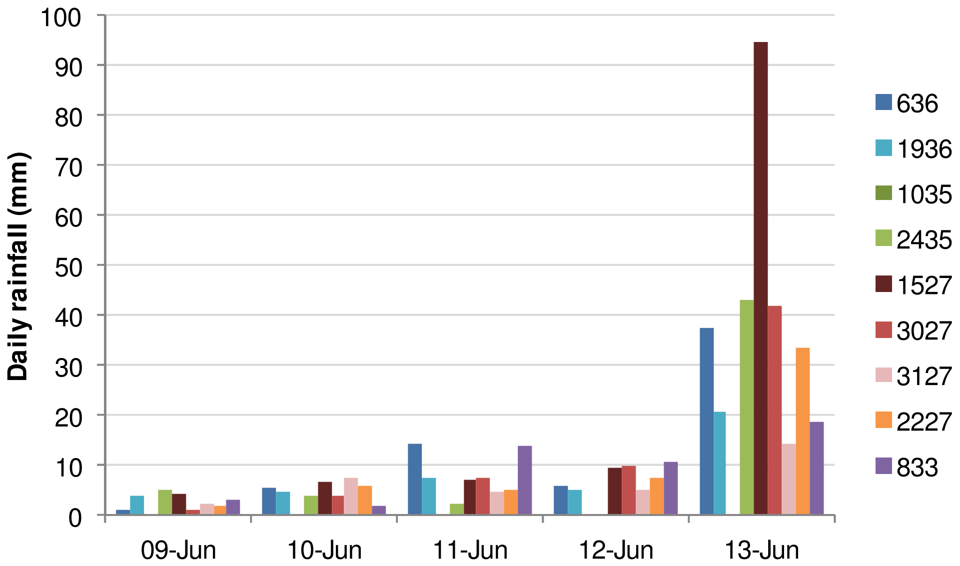
10 to 14 June 1964

Rainfall accumulations for the whole event



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
1527	1	94.7	0.9
	2	104.4	1.1
	3	111.5	1.4
	4	118.4	1.7
3027	1	41.8	37.0
	2	51.6	37.0
	4	59.3	37.0
	6	63.1	45.5



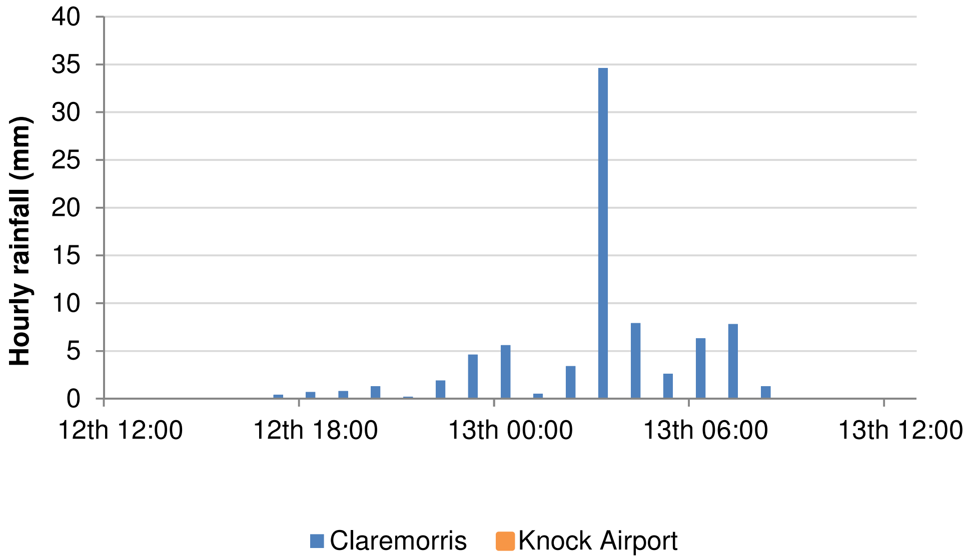
This summer event occurred during a period of light to moderate rain across the whole Western RBD, but the intense rainfall on 13 June was concentrated in the north of hydrometric area 30, between Lough Corrib and Claremorris. At gauge 1527 (Hollymount) the AEP of the 1-day total was 1%. At other key gauges the event was much less extreme. The next page summarises analysis of sub-daily rainfall data.

Analysis of hourly rainfall data

The short, intense nature of this event indicates that analysis of sub-daily rainfall data is worthwhile. Data is available from one gauge in the study area, Claremorris (see the map on the previous page).

Depth duration frequency at Claremorris		
Duration (hours)	Depth (mm)	AEP (%)
1	34.6	1.2
2	42.5	1.2
3	55.1	0.7
4	61.4	0.6
6	72.6	0.5
9	83.3	<0.5
12	86.7	0.6

Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.



During an event which lasted around 10 hours at Claremorris there was an exceptionally heavy burst of rainfall, 34.6mm in 1 hour between 0200 and 0300 on 13 June. Over all accumulation durations from 1 to 24 hours this is the highest rainfall recorded to date at Claremorris (1950-2010).

The AEP of the 1-hour total was 1.2%, i.e. a return period of 80 years. Over the full duration of the event, the AEP was just under 0.5, i.e. a return period over 200 years. This is consistent with the analysis of the daily rainfall data in the vicinity, for example at gauge 1527. It is likely (although hard to be sure without any other recording raingauge data) that the duration of the event was similar at other nearby locations which recorded large daily totals. Rainfall of this intensity is likely to have resulted in local flooding.

Sub-daily rainfall event summary sheet

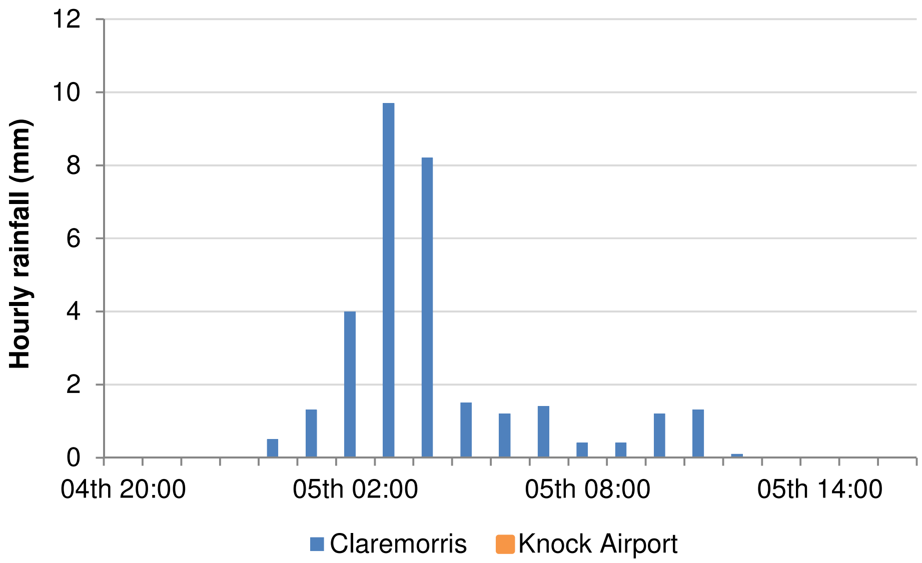
5 October 1964

Hourly rainfall data is available from one gauge in the study area, Claremorris.

Depth duration frequency at Claremorris

Duration (hours)	Depth (mm)	AEP (%)
1	9.7	High
2	17.9	31.1
3	21.9	26.5
4	23.4	29.7
6	24.7	39.0
9	27.3	44.8
12	29.3	49.5

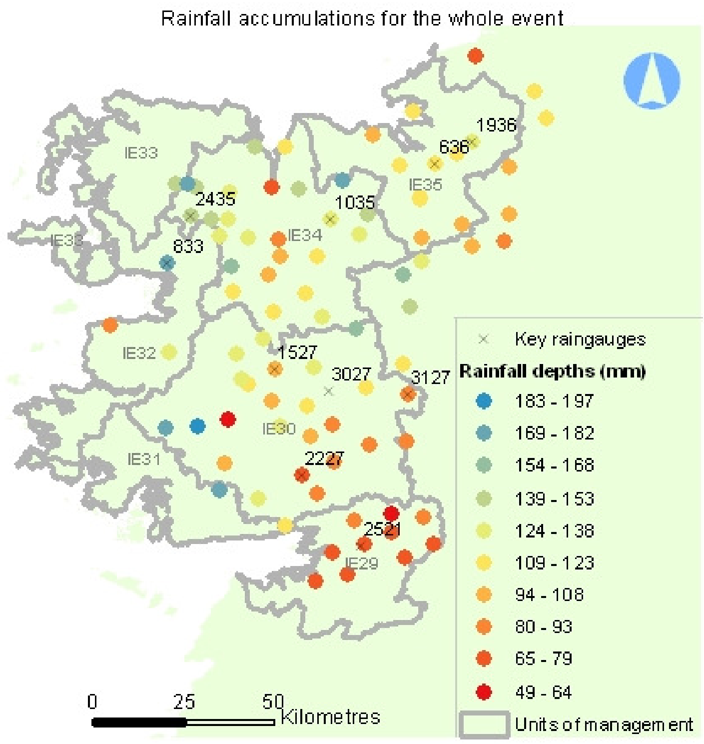
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.



Heavy rainfall was recorded in the early hours of 5 October. Over a duration of 2-4 hours the AEP was around 30%, i.e. a return period of 3 years.

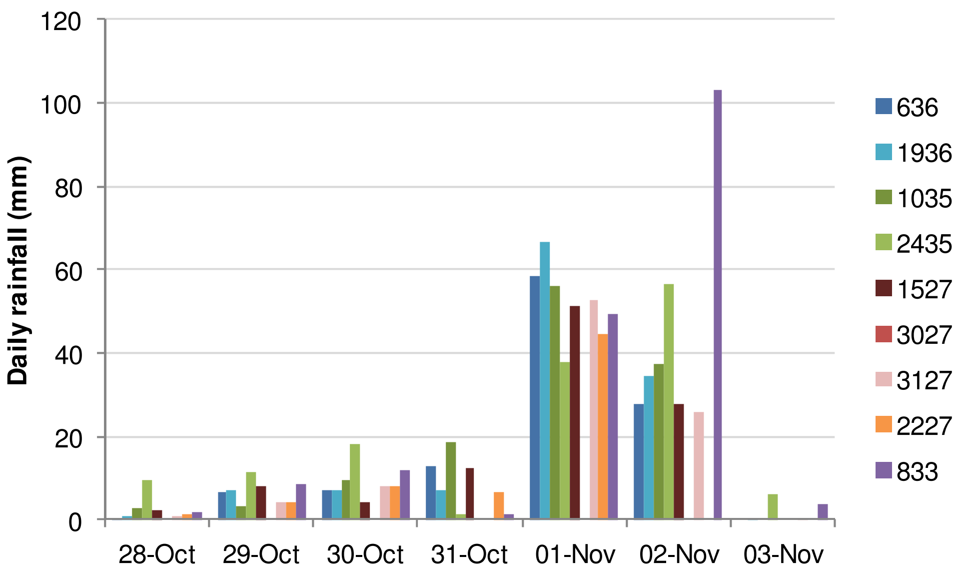
Rainfall event summary sheet

29 October to 2 November 1968



Depth duration frequency at selected gauges with the most extreme rainfalls

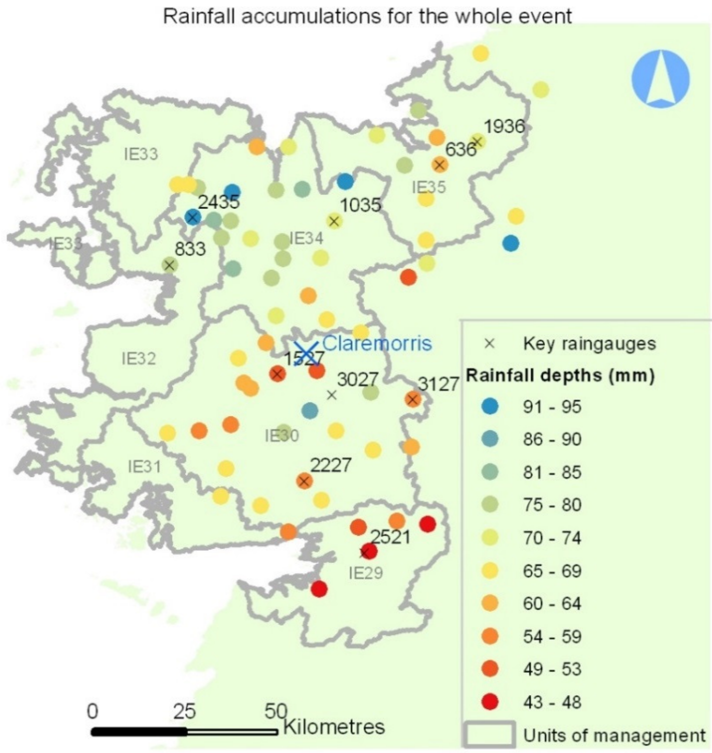
Raingauge number	Duration (days)	Depth (mm)	AEP (%)
636	1	58.4	8.8
	2	86.4	2.6
	4	106.7	2.5
	6	113.8	4.7
833	1	103	2.2
	2	152.5	0.6
	4	165.7	1.4
	6	177.9	2.6
1035	1	56.3	14.1
	2	93.9	1.7
	4	121.9	1.2
	6	128	2.8



Several days of moderate rainfall in late October were followed by two days of heavy rainfall, 1 and 2 November, affecting all parts of the Western RBD although with much larger totals to the west and north. Rainfall rarities were most notable over a duration of 2-4 days, with AEPs as low as 0.6% (a return period of 160 years) at Newport, north of Westport.

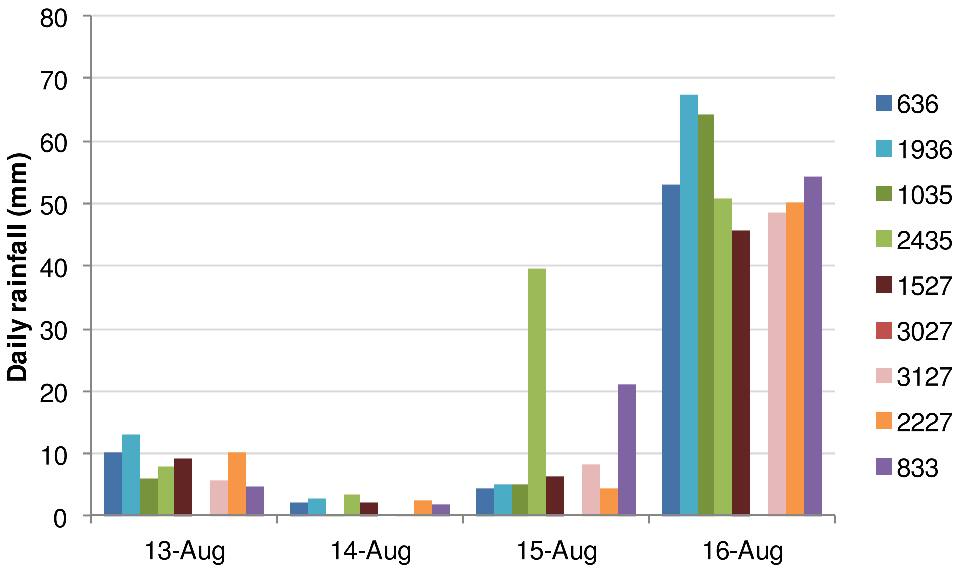
Rainfall event summary sheet

13 to 16 August 1970



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
636	1	53	14.1
	2	57.4	24.4
	3	59.7	40.0
	4	69.9	34.5
1035	1	64.1	6.7
	2	69.2	12.2
	3	69.9	26.3
	4	75.8	31.3
2227	1	50.1	12.3
	2	54.5	25.6
	3	56.9	45.5
	4	67.2	37.0



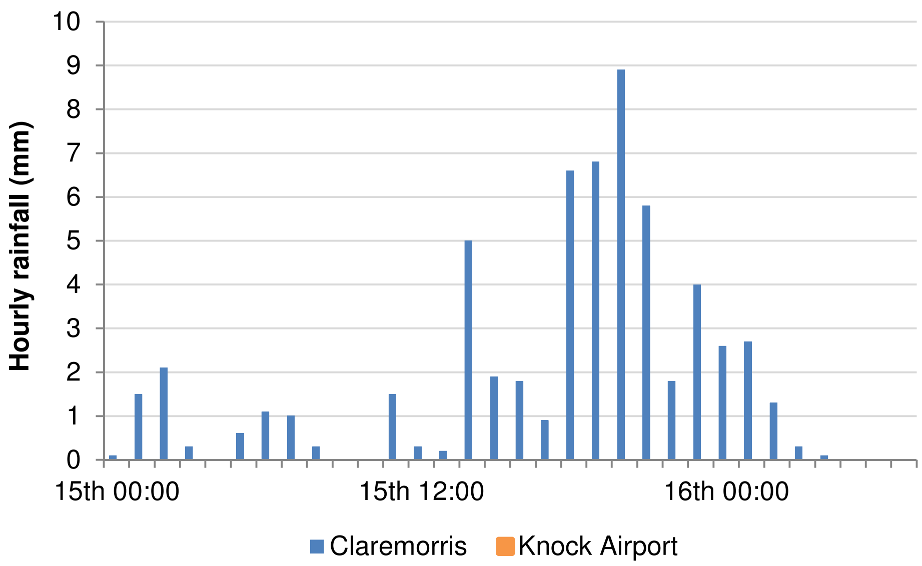
Moderate rainfall on 13 and 15 August was followed by a heavy fall on 16th. The rainfall was heaviest in hydrometric areas 32 and 34 and the northern part of area 30. High rainfall totals were recorded in the Nephin Beg mountains of Mayo (e.g. at gauge 2435) but the event rarity was most severe further east. At gauge 1035 (Aclare, north of Swinford) the 1-day AEP was 7%, a return period of 15 years.

Analysis of hourly rainfall data

The short, intense nature of this event indicates that analysis of sub-daily rainfall data is worthwhile. Data is available from one gauge in the study area, Claremorris (shown on the map on the last page).

Depth duration frequency at Claremorris		
Duration (hours)	Depth (mm)	AEP (%)
1	15.7	22.0
2	22.3	15.5
3	28.1	11.2
4	29.9	12.8
6	36.5	10.1
9	43.5	8.7
12	50.1	7.2

Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.

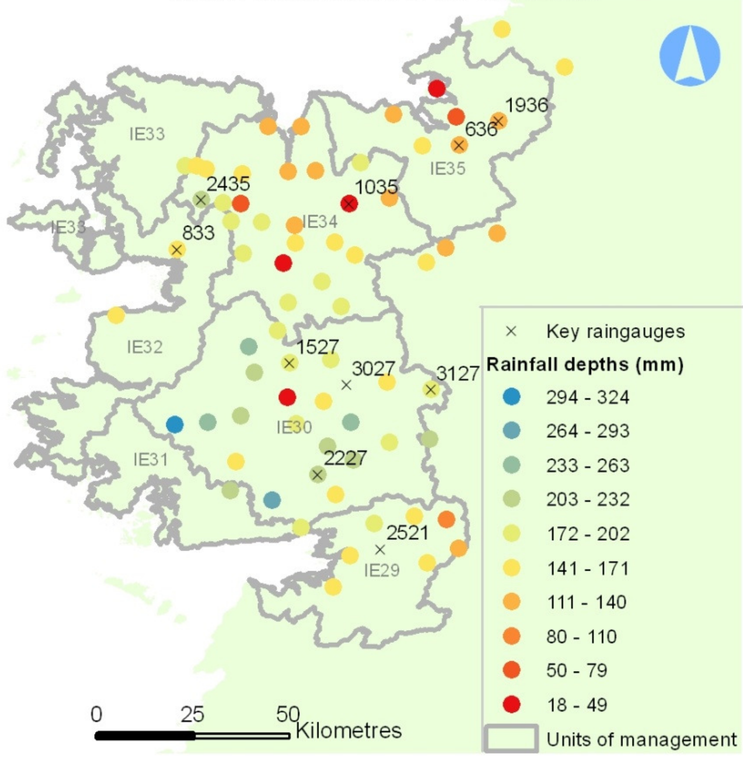


After light rain on the morning of 15 August, heavy rain fell during the afternoon and overnight into 16 August. The AEPs indicate that the rainfall was not particularly extreme at Claremorris. It can be seen from the map that the rainfall was heavier further north and also to the south.

Rainfall event summary sheet

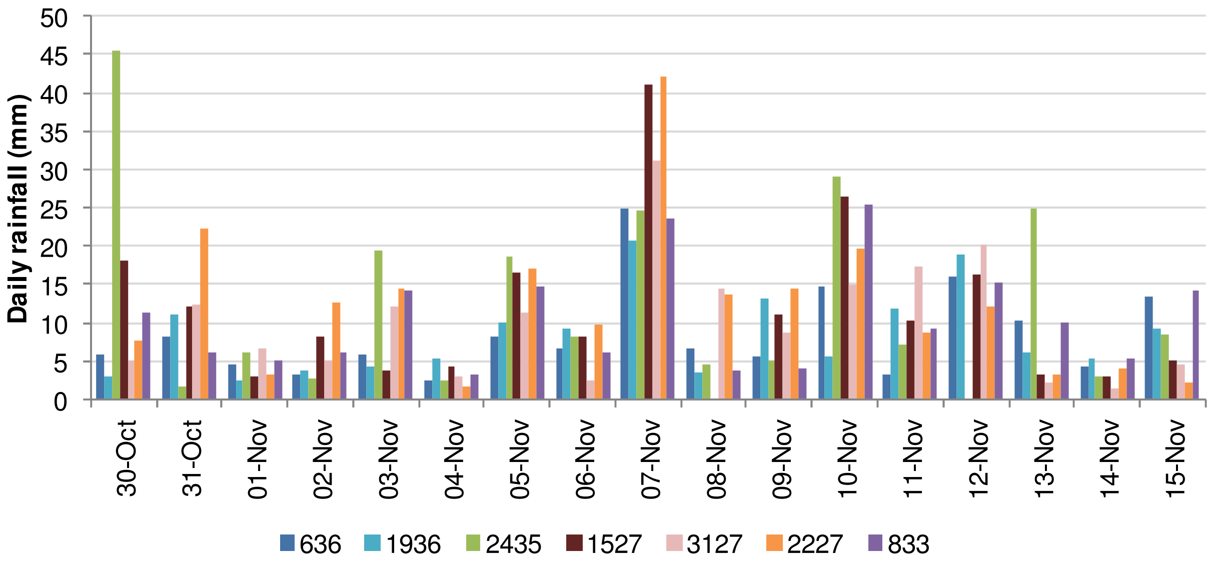
29 October to 14 November 1977

Rainfall accumulations for the whole event



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
1527	1	46.9	31.3
	4	78.7	24.4
	7	113.7	11.2
	14	179.5	4.3
3127	1	31.2	71.4
	4	69.5	32.3
	7	109.3	9.8
	14	165.1	5.6
2227	1	42.1	33.3
	4	89.8	4.7
	7	125.4	2.2
	14	199.6	0.7



Prolonged rainfall frequently occurs in late Autumn. In 1977 there was some rain every day from late September to late November. The highest falls were in early November, particularly over hydrometric area 30 and the south of 34. The map shows a few raingauges in this area with much lower rain but this is probably due to missing data. Further north, around Sligo, there was much less rain. The maximum accumulation over a 2-week period was not particularly extreme at most gauges, but at 2227 (Carndolla, between Galway and Headford) the AEP was as low as 0.7% (a return period of 150 years).

Sub-daily rainfall event summary sheet

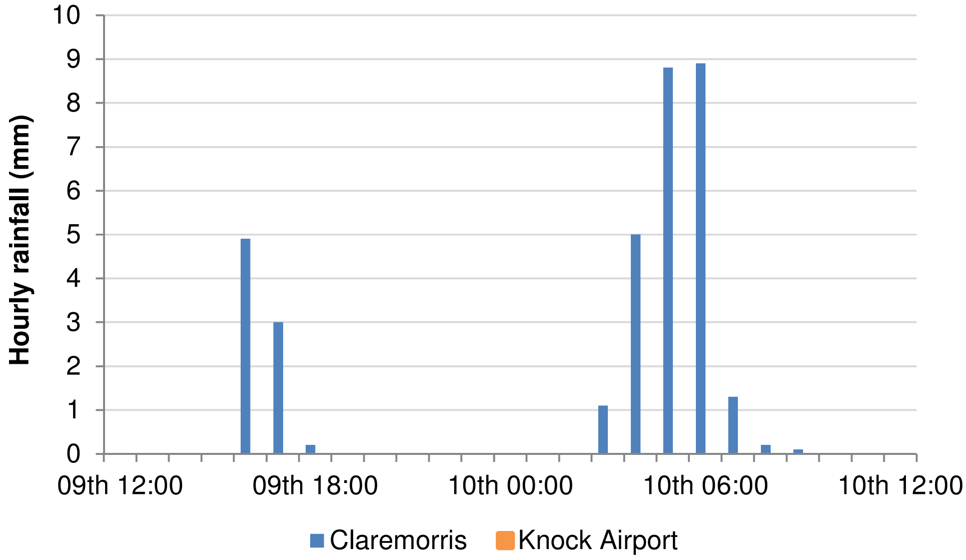
10 September 1981

Hourly rainfall data is available from one gauge in the study area, Claremorris.

Depth duration frequency at Claremorris

Duration (hours)	Depth (mm)	AEP (%)
1	8.9	High
2	17.7	32.1
3	22.7	23.7
4	24	27.5
6	25.1	37.3
9	25.4	High
12	25.4	High

Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.



After a brief shower on the afternoon of 9 September, heavy rainfall was recorded early in the morning on 10 September. The lowest AEP was for the 3-hour accumulation of 22.7mm, which has an AEP of 24%, i.e. return period of 4 years.

Sub-daily rainfall event summary sheet

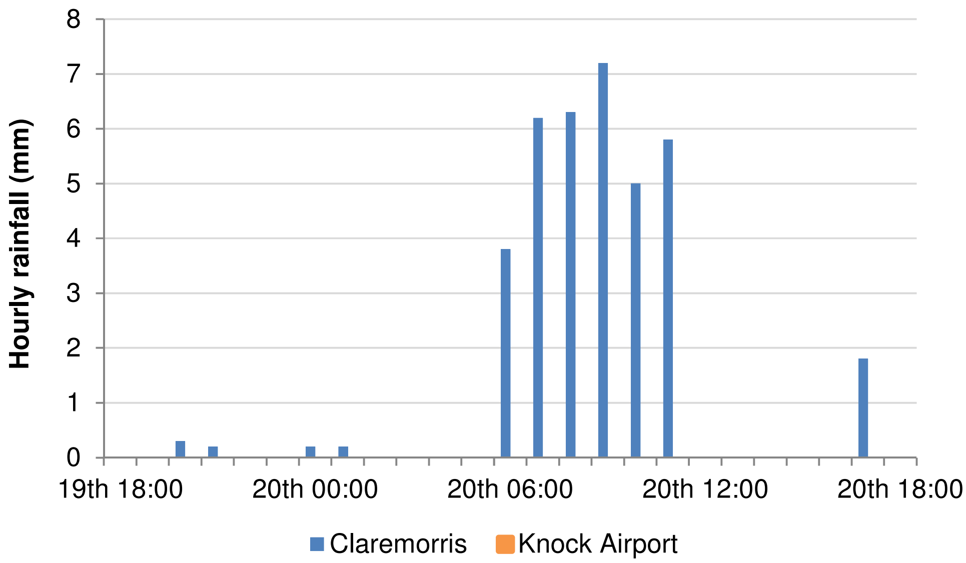
20 August 1987

Hourly rainfall data is available from one gauge in the study area, Claremorris.

Depth duration frequency at Claremorris

Duration (hours)	Depth (mm)	AEP (%)
1	7.2	High
2	13.5	High
3	19.7	36.2
4	24.7	25.1
6	34.3	13.0
9	34.3	22.1
12	36.1	26.4

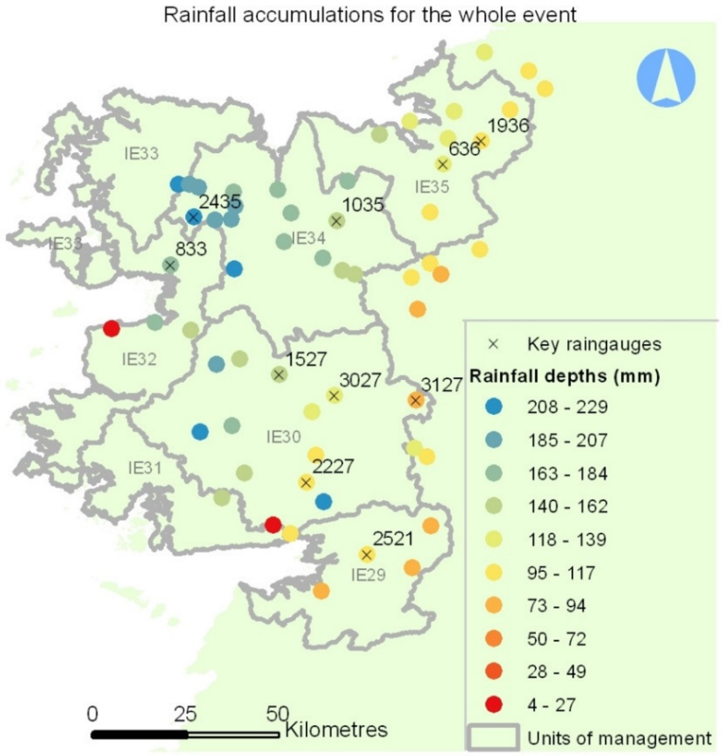
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.



Warm and humid weather, associated with southerly winds, brought periods of heavy rainfall during mid-August. This short rainfall event lasted for 6 hours on the morning of 20 August. The 6-hour accumulation at Claremorris had an AEP of 13%, i.e. a return period of 8 years.

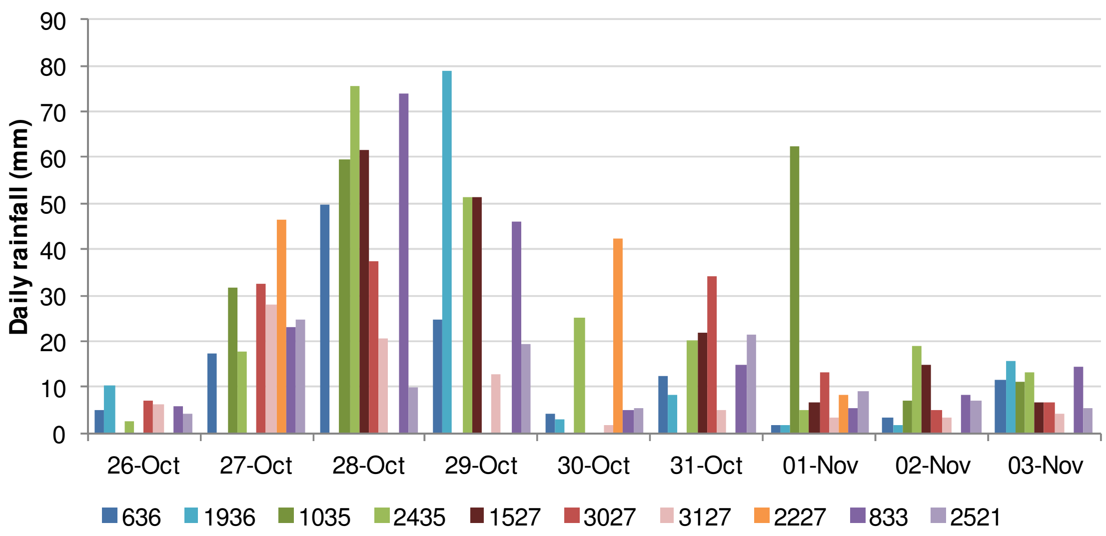
Rainfall event summary sheet

26 October to 2 November 1989



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
1035	1	62.5	7.8
	4	96.3	6.8
	6	153.7	0.6
	8	172.1	0.7
1527	1	61.4	9.2
	4	134.4	0.7
	6	155.7	0.6
	8	173.1	0.6
833	1	73.7	11.6
	4	148.6	2.8
	6	168.4	3.8
	8	190.5	4.2

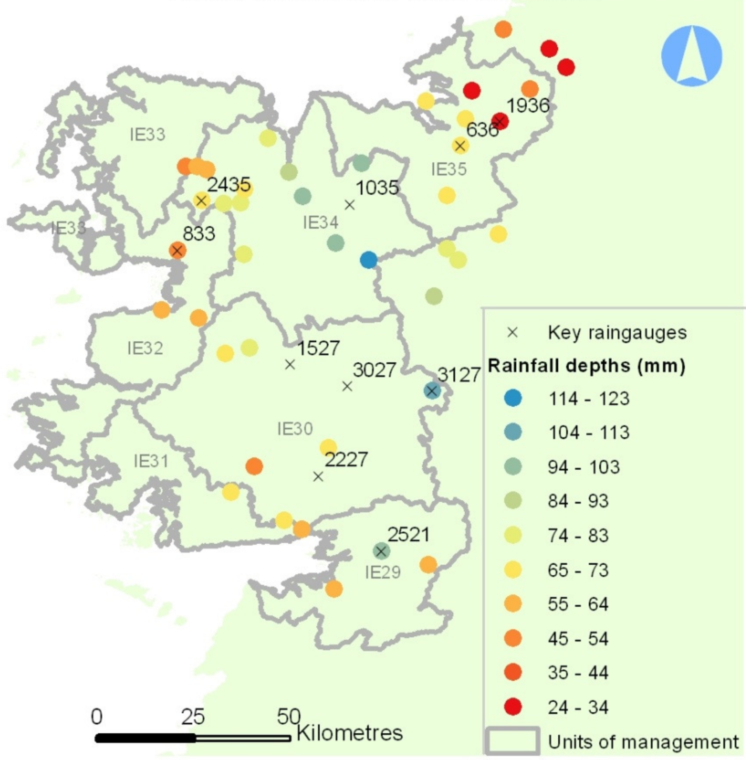


Rainfall affected all of the study area from 5 October to mid-November 1989 and was most severe in late October when a depression approached the extreme SW of Ireland and then moved east, resulting in a slow-moving band of rain associated with a warm front. The largest falls were over the Galway and Mayo mountains and over much of hydrometric areas 30, 32, 33 and 34. The two red spots on the map are probably due to periods of missing data. At Belmullet (NW corner of County Mayo) it was the wettest October since records began, with 129mm recorded in a 36- hour period. AEPs were below 1% for accumulations over several days at gauges 1035 (Aclare) and 1527 (Holymount).

Rainfall event summary sheet

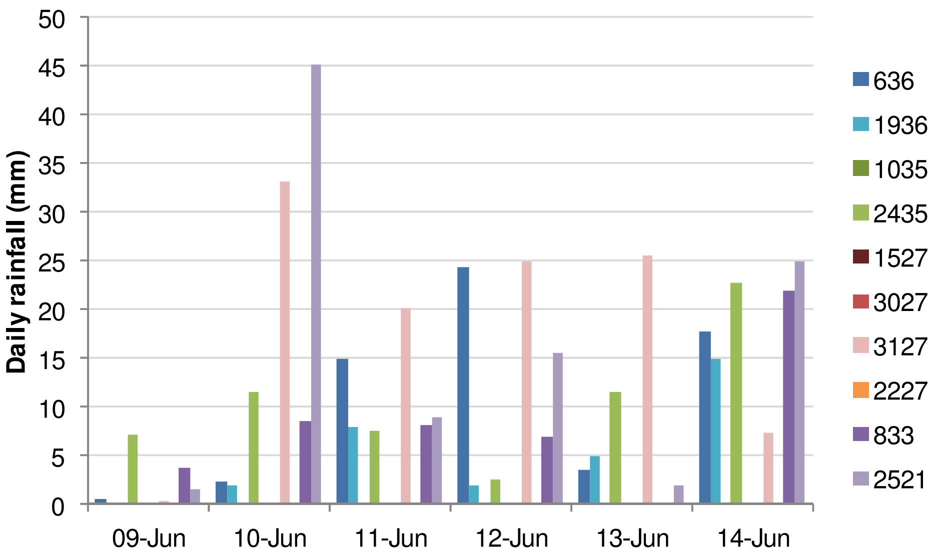
9 to 14 June 1993

Rainfall accumulations for the whole event



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
3127	1	33.2	62.5
	2	53.4	30.3
	3	78.3	9.0
	4	103.8	2.9
2521	45.2	25.6	45.2
	54.2	28.6	54.2
	69.7	14.7	69.7
	71.6	25.0	71.6

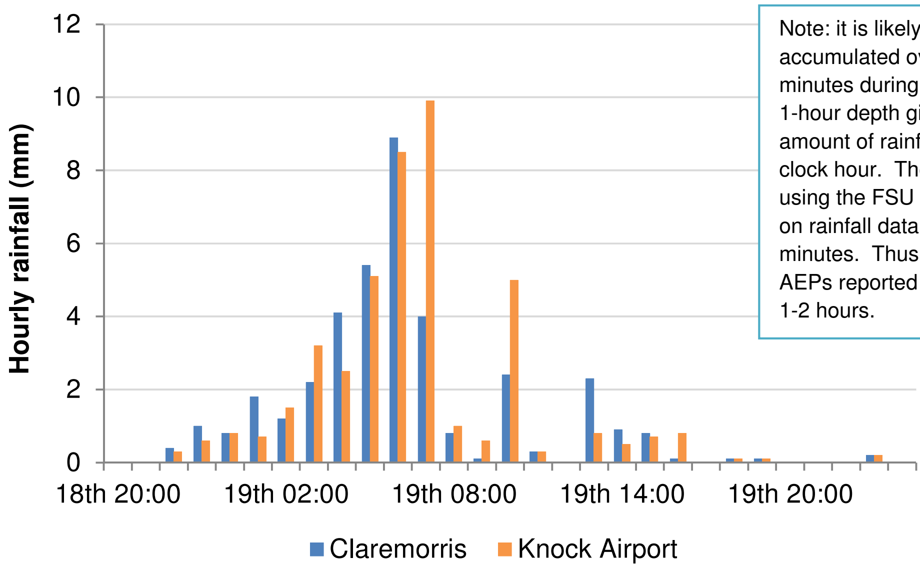


Note that data is missing from several of the key gauges during this event. Rain was caused by a cool northerly airflow due to a depression centred over England and Wales. On 11 June there was very heavy rain in the east midlands and north of Ireland. In the Western RBD, the rainfall over this period was heaviest inland, in the east of hydrometric areas 29, 30 and 34. At gauge 3127 (Glenamaddy, north-east of Tuam) there were four days of notable rainfall, totalling 104mm, with an AEP of 3% over the 4 days (a return period of 30 years).

19 July 1998

Hourly rainfall data is available from two gauges in the study area, Claremorris and Knock Airport.

Depth duration frequency at Claremorris			Depth duration frequency at Knock Airport		
Duration (hours)	Depth (mm)	AEP (%)	Duration (hours)	Depth (mm)	AEP (%)
1	8.9	High	1	9.9	High
2	14.3	High	2	18.4	33.1
3	18.4	43.4	3	23.5	24.9
4	22.4	33.7	4	26	25.1
6	25.8	34.4	6	30.7	23.4
9	29.4	36.2	9	37.3	19.8
12	32.7	36.2	12	39.4	23.2



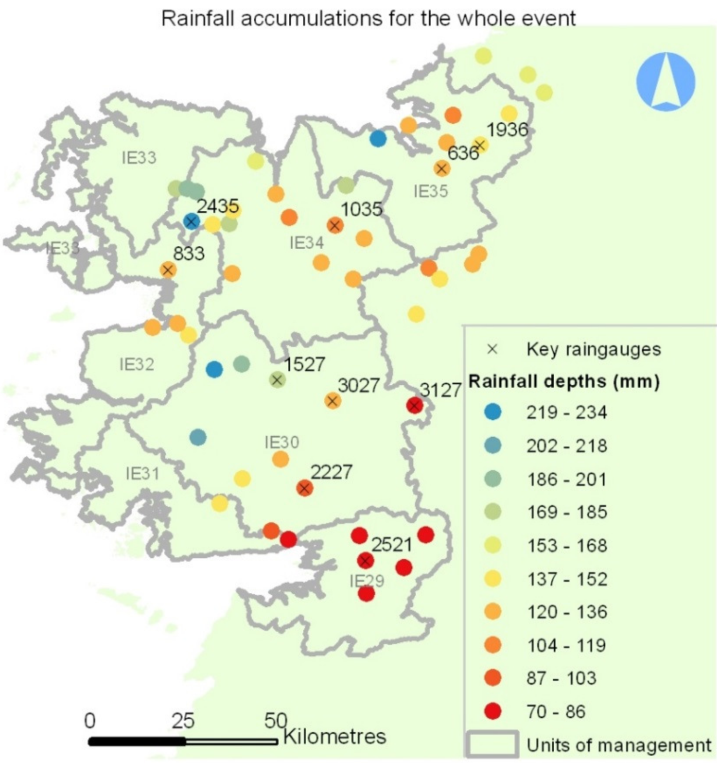
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.

19 July was a cloudy day with close to normal temperatures. There were spells of rain, some heavy and thunder, across much of Ireland apart from the east coast.

At both raingauges, the event started around midnight on 19 July and continued through the morning. The heaviest rainfall was recorded from 0400 to 0700. The depth of rainfall was similar at the two gauges, and the AEPs indicated that the rainfall was not particularly extreme: typical AEPs were 30-40% at Claremorris and 20-25% (i.e. return periods of 4-5 years) at Knock Airport.

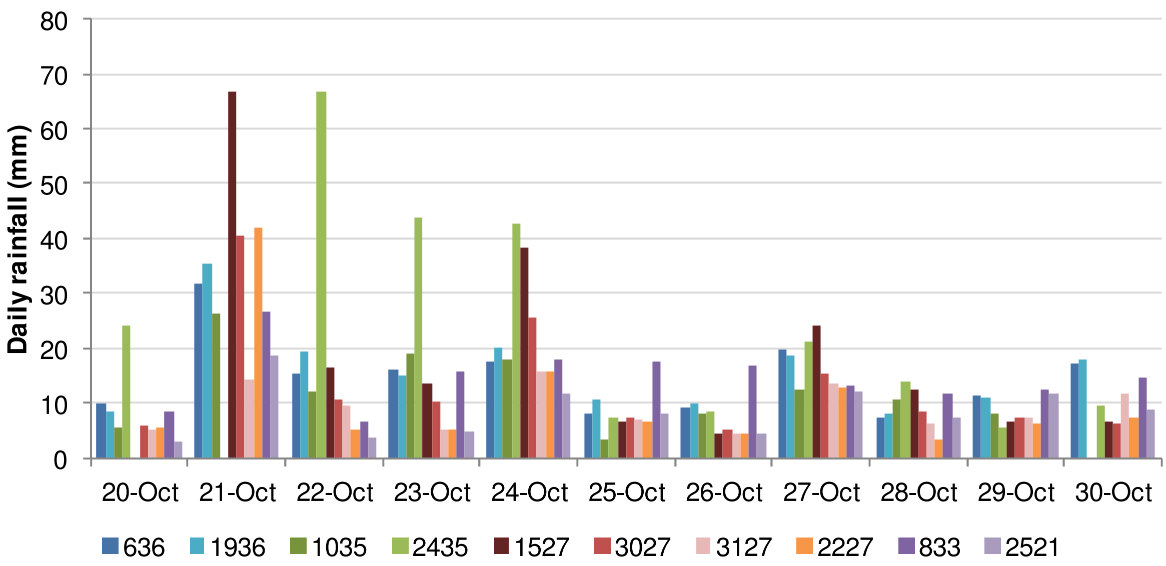
Rainfall event summary sheet

20 to 28 October 1998



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
636	1	31.6	71.4
	2	46.8	52.6
	4	80.5	16.1
	7	117.8	5.9
2435	1	66.8	38.5
	2	110.5	8.5
	4	160.7	3.7
	7	204.3	4.0
1527	1	66.6	6.0
	2	82.9	4.3
	4	134.8	0.7
	7	170.2	0.5



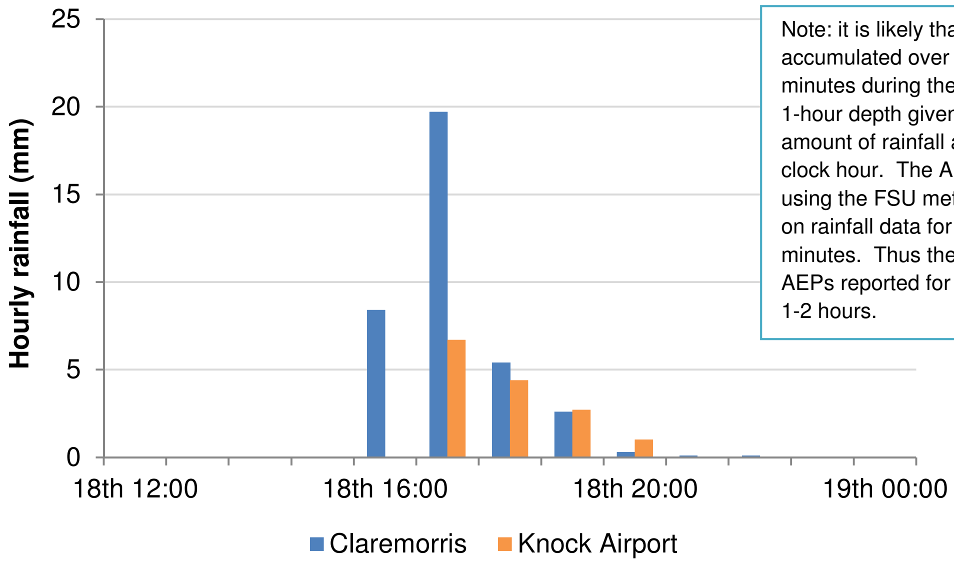
On 20-21 October a deepening depression moved northwards to the west of Ireland bringing heavy frontal rainfall driven by south-easterly gales. There was more widespread and heavier rainfall on 25th. Total October rainfall was near-normal for the western RBD whereas in the SW of Ireland it was the wettest October since 1940. The event impacted all of the Western RBD although totals were lower in hydrometric area 29. It was most extreme at gauge 1527, Hollymount, where the AEP was as low as 0.5% over 1 week of rain – although this may be exaggerated by a possible 2-day accumulation of rain recorded on 21 Oct.

Sub-daily rainfall event summary sheet

18 August 2000

Hourly rainfall data is available from two gauges in the study area, Claremorris and Knock Airport.

Depth duration frequency at Claremorris			Depth duration frequency at Knock Airport		
Duration (hours)	Depth (mm)	AEP (%)	Duration (hours)	Depth (mm)	AEP (%)
1	19.7	10.2	1	6.7	High
2	28.1	6.5	2	11.1	High
3	33.5	5.5	3	13.8	High
4	36.1	6.0	4	14.8	High
6	36.5	10.1	6	14.8	High
9	36.6	17.5	9	14.8	High
12	36.6	25.2	12	14.8	High



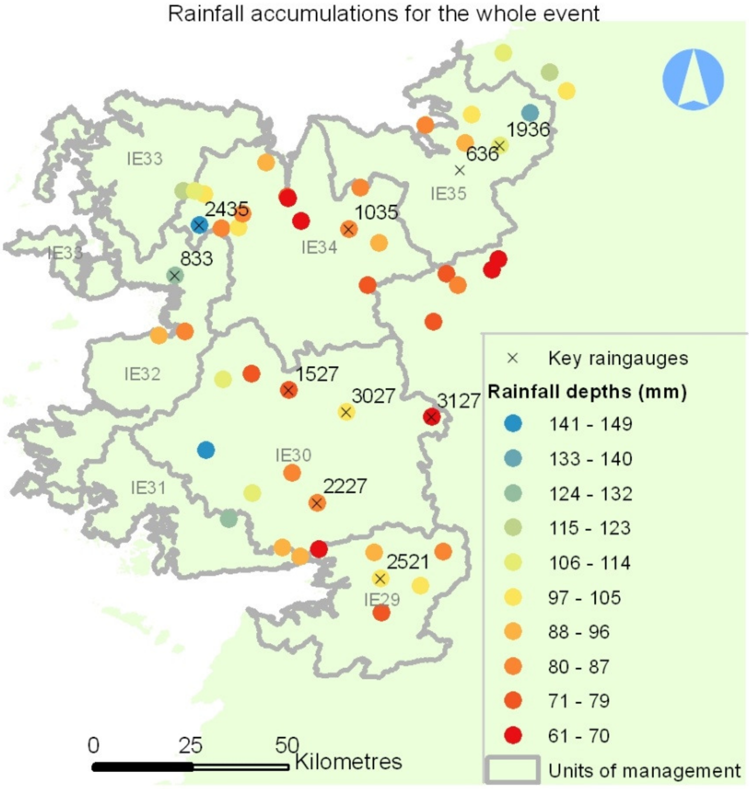
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.

August 2000 was warm and there were frequent thunderstorms between 16th and 21st. On 18th thunder showers were confined to the north-west of Ireland, with temperatures between 16° and 19° C.

This event was a brief burst of rainfall which lasted for a few hours in the late afternoon and early evening of 18 August. At Knock Airport the totals were not noteworthy but at Claremorris the rainfall was intense, resulting in AEPs around 6% for durations 2-4 hours (i.e. return periods around 17 years).

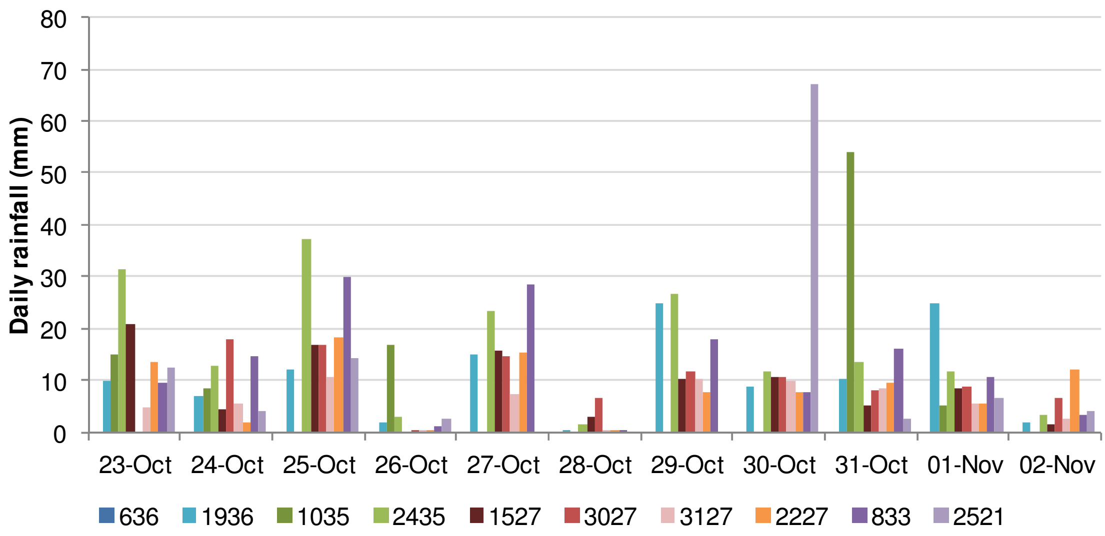
Rainfall event summary sheet

24 October to 2 November 2000



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
2521	2	n/a	n/a
	4	80.8	12
	7	92.5	24
	14	142.3	15
2435	2	58.2	>50
	4	87.4	>50
	7	135.8	>50
	14	239.2	28

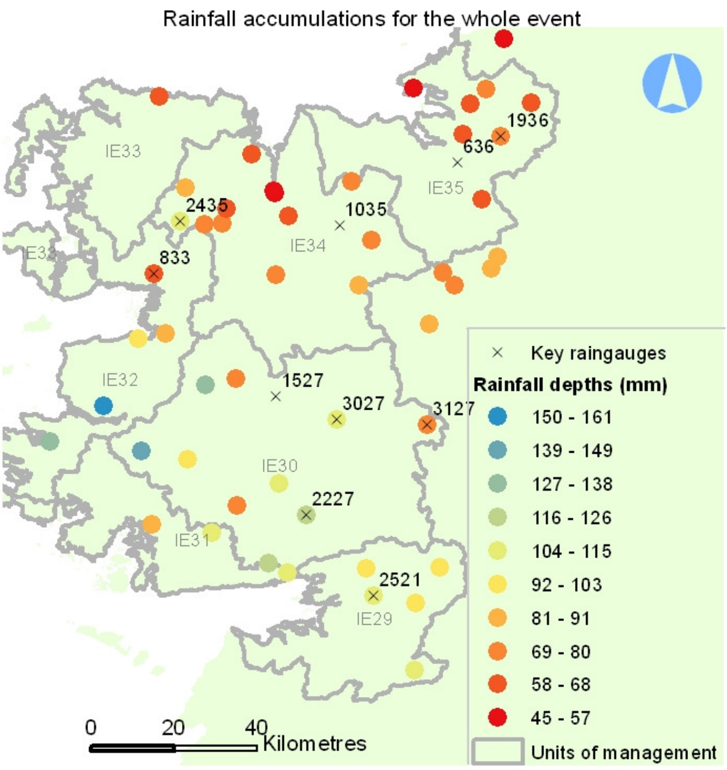


This event affected all of the Western RBD. A succession of Atlantic depressions brought rain almost every day from late August to mid December 2000. The highest totals were observed in late Oct and early Nov, although the event was not particularly severe at any of the key gauges analysed. The lowest AEP was at gauge 2521, Craughwell. In England and Wales the event was much more severe. Over the whole of October, rainfall was highest of any October on record at Galway Airport and Maam Valley.

Note: the reported depth of 67.3mm at gauge 2521 on 30 October was probably in fact an accumulation over four days, as zero rainfall was reported at this gauge for the preceding three days.

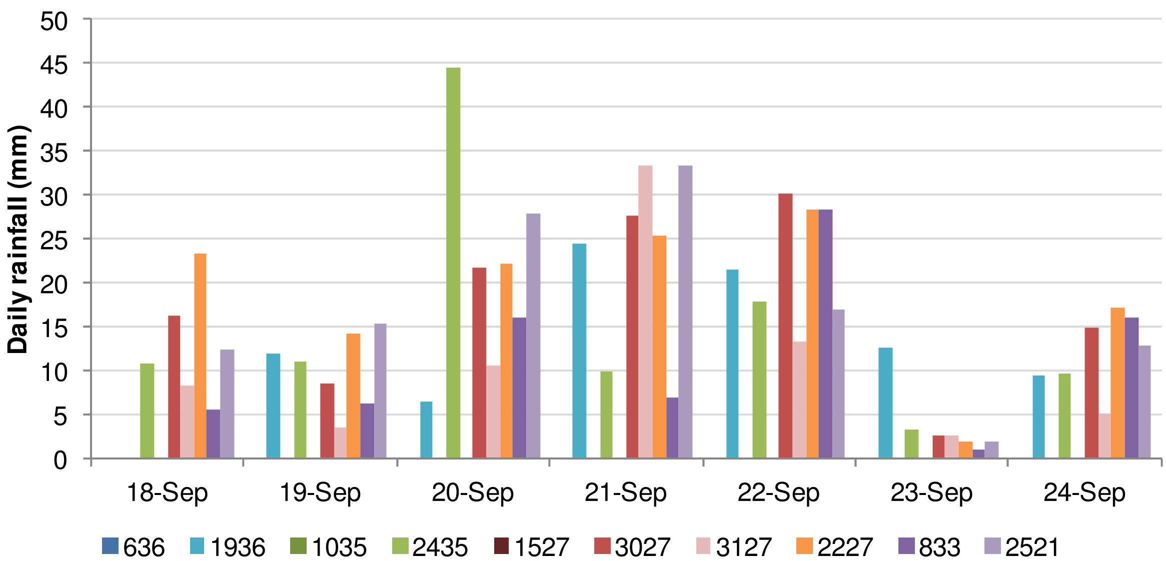
Rainfall event summary sheet

17 to 23 September 2006



Depth duration frequency at selected gauges with the most extreme rainfalls

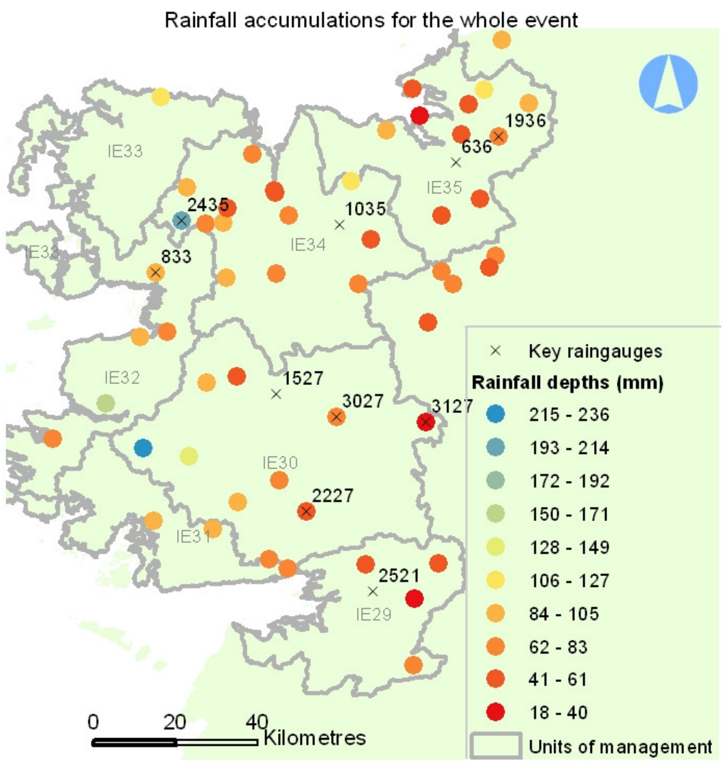
Raingauge number	Duration (days)	Depth (mm)	AEP (%)
3027	1	30.2	76.9
	2	57.9	23.3
	4	88.1	9.6
	7	121.7	5.2
2227	1	28.4	90.9
	2	53.8	27.8
	4	90.1	4.6
	7	132.4	1.3
2521	1	33.4	76.9
	2	61.3	13.7
	4	93.6	4.0
	7	120.7	3.5



This was the warmest September on record in many parts of Ireland. Deep Atlantic depressions brought wet and windy weather. The rain on 20th-21st was caused by the remnants of Hurricane Gordon. This event was more severe in the south of the RBD, with multi-day accumulations having AEPs around 5% in hydrometric areas 29 and 30. The lowest AEP was at gauge 2227, Carndolla, between Galway and Headford, where the maximum 7-day accumulation had an AEP of 1.3% (a return period of 70 years).

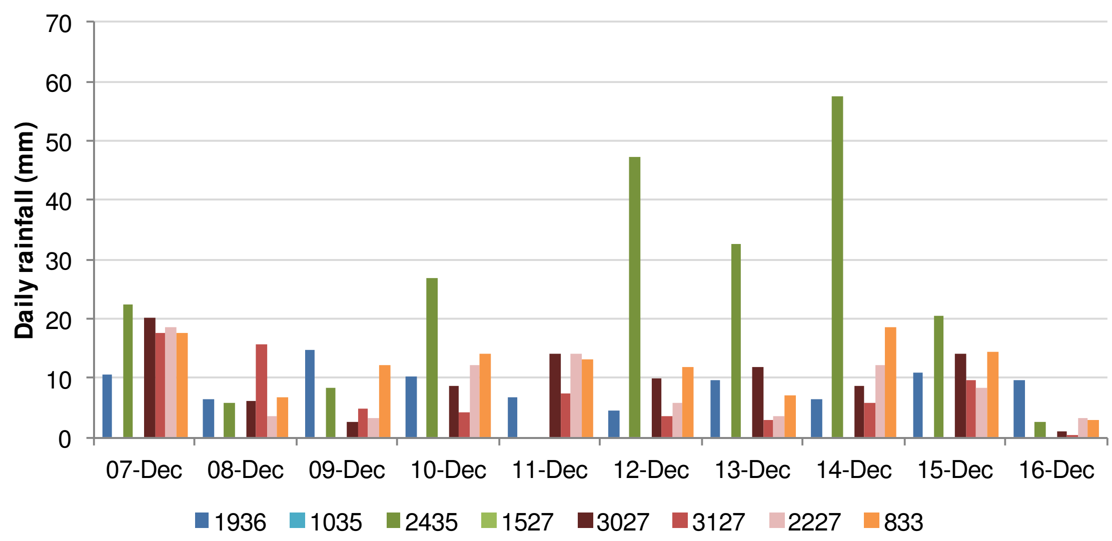
Rainfall event summary sheet

9 to 15 December 2006



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
2435	2	101.3	14.7
	4	157.7	4.3
	7	192.8	6.6
	14	368.1	0.4
3027	2	89.4	2.8
	4	118.7	1.7
	7	136.1	2.5
	14	196.6	1.5
2227	2	41.3	76.9
	4	76.4	16.4
	7	118.1	3.7
	14	173	3.0



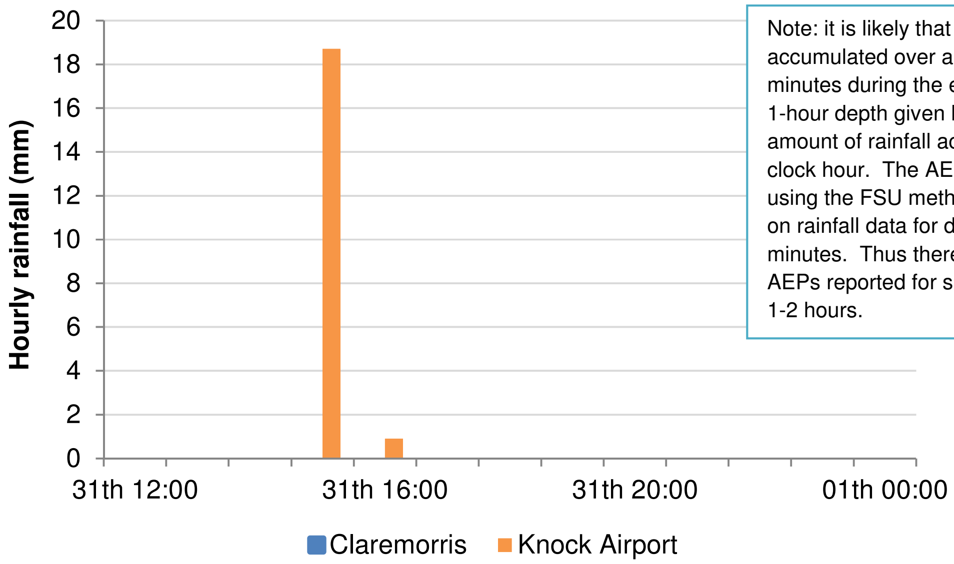
A series of very deep depressions passing to the northwest of Ireland brought rain, accompanied by strong south-westerly winds. There was rain almost every day from 7 November to mid-December. During 9-15 Dec there were exceptionally high totals in the western mountainous areas, particularly at gauge 2435 (Keenagh Beg, in the Nephin Beg hills above Crossmolina) where the AEP over 2 weeks was 0.4%, i.e. a return period of 400 years. The event was also notable in hydrometric area 30, with AEPs of 1-3% at gauges 3027 and 2227. It is possible that some of the low rainfall totals shown on the map are due to missing data.

Sub-daily rainfall event summary sheet

31 May 2008

Hourly rainfall data is available from two gauges in the study area, Claremorris and Knock Airport.

Depth duration frequency at Claremorris			Depth duration frequency at Knock Airport		
Duration (hours)	Depth (mm)	AEP (%)	Duration (hours)	Depth (mm)	AEP (%)
1	0.1	n/a	1	18.7	15.0
2	0.1	n/a	2	19.6	27.7
3	0.1	n/a	3	19.6	41.2
4	0.1	n/a	4	19.6	High
6	0.1	n/a	6	19.6	High
9	0.1	n/a	9	19.6	High
12	0.1	n/a	12	19.6	High



Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.

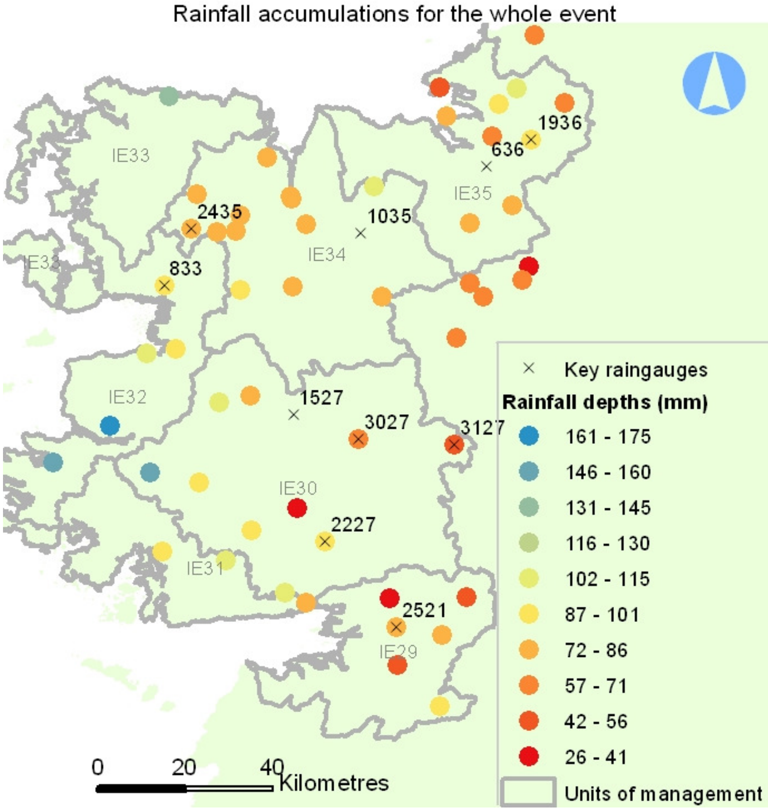
May 2008 was sunny, dry and warm. On May 31st, a thunderstorm in County Mayo resulted in a brief intense fall of rain which was recorded at Knock Airport. 25km to the south-west at Claremorris there was no rain. From the daily rainfall data it appears that the highest rainfall was 25mm at Strade, north-east of Castlebar.

The 1-hour fall of 18.7mm is the highest on record to date at Knock Airport (1996-2010) and had an AEP of 15% (i.e. a return period of 7 years).

Rainfall event summary sheet

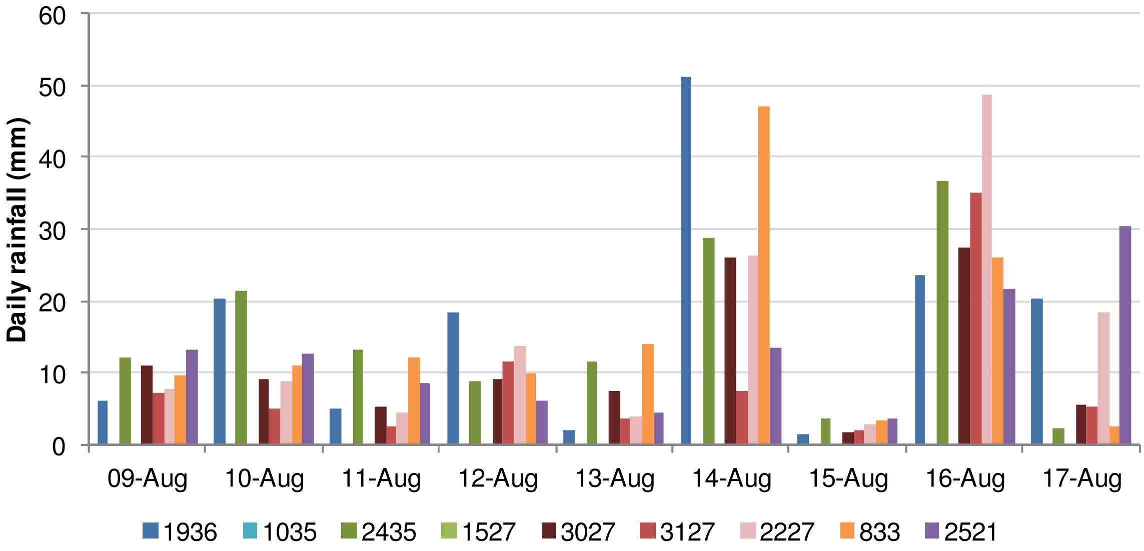
14 to 16 August 2008

Rainfall accumulations for the whole event



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
1936	1	51.2	17.2
	2	53.2	43.5
	4	96.4	15.2
	7	121.9	20.4
2227	1	48.6	14.9
	2	66.9	6.5
	4	96	2.7
	7	118.1	3.7
2521	1	30.4	83.3
	2	52.1	34.5
	4	69.2	30.3
	7	88.3	32.3



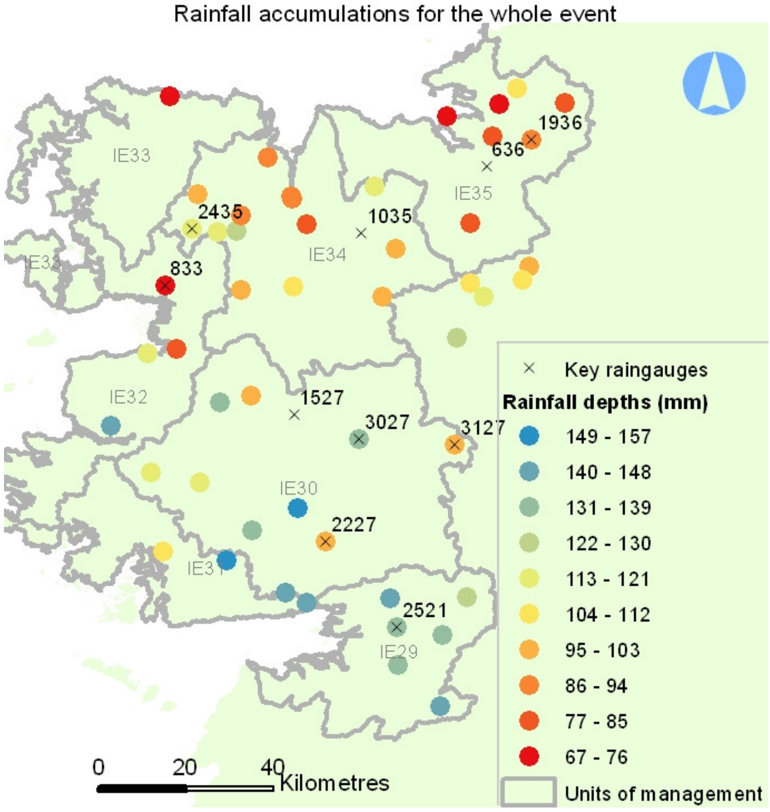
Low pressure close to or over Ireland brought a succession of Atlantic frontal systems across the country, giving some significant falls on 14th and 16th. It was the wettest August in some parts of Ireland. The event affected all of the Western RBD. It was not particularly severe, with an AEP exceeding 30% at most gauges. The lowest AEP was 3% for the 4-day total at gauge 2227, Carndolla.

Further information on this event is available in Met Éireann’s Climatological Note No. 11.

Note: some of the low rainfalls shown on the map are due to periods of missing data.

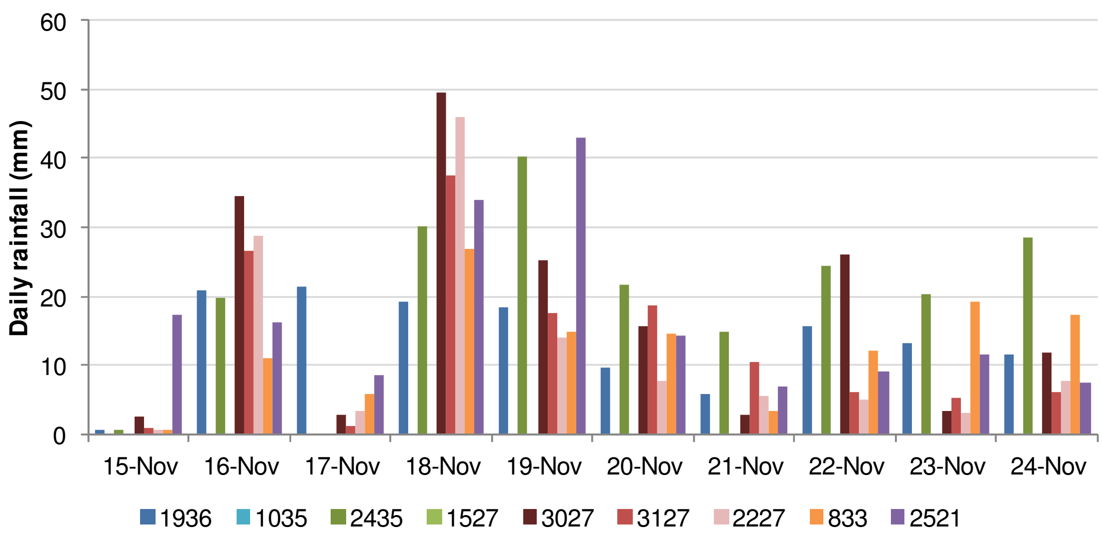
Rainfall event summary sheet

15 to 20 November 2009



Depth duration frequency at selected gauges with the most extreme rainfalls

Raingauge number	Duration (days)	Depth (mm)	AEP (%)
3027	2	74.6	7.1
	4	111.9	2.4
	7	156.2	1.0
	14	210.8	0.9
3127	2	55.1	26.3
	4	84.3	11.1
	7	118.4	5.5
	14	174.4	3.4
2521	2	76.8	2.9
	4	101.4	2.2
	7	146.9	0.7
	14	212.9	0.5



Atlantic depressions passing close to Ireland brought wet and windy conditions throughout almost all of November, continuing a pattern of very unsettled weather over Ireland that began in mid-October. Rainfall totals for November were the highest on record at most stations. In the Western RBD rain fell almost every day from 18 October to 28 November. The highest totals were in the south of the RBD, in hydrometric areas 29 to 31, particularly in the vicinity of Galway. The AEP was below 1% (a return period of 150-200 years) for 1 and 2-week accumulations at gauge 2521, Craughwell, south of Athenry.

Further information on this event is available in Met Éireann’s Climatological Note No. 12.

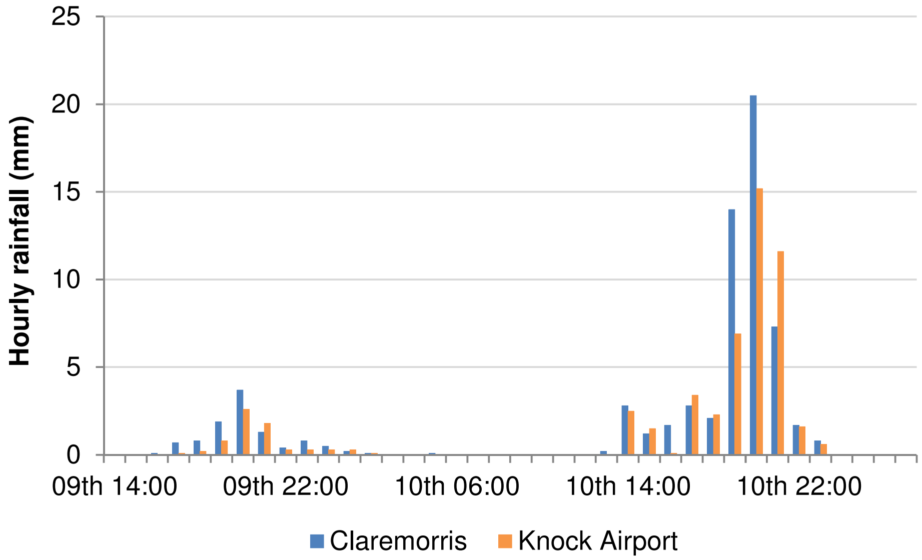
Sub-daily rainfall event summary sheet

10 July 2010

Hourly rainfall data is available from two gauges in the study area, Claremorris and Knock Airport.

Depth duration frequency at Claremorris			Depth duration frequency at Knock Airport		
Duration (hours)	Depth (mm)	AEP (%)	Duration (hours)	Depth (mm)	AEP (%)
1	20.5	8.9	1	15.2	28.1
2	34.5	2.9	2	26.8	9.7
3	41.8	2.2	3	33.7	6.9
4	43.9	2.6	4	36	7.8
6	48.4	3.1	6	41	8.0
9	54.1	3.3	9	45.1	9.5
12	55.1	4.7	12	45.7	13.4

Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based on rainfall data for durations as short as 15 minutes. Thus there may be a bias in the AEPs reported for short durations, particularly 1-2 hours.



Rain fell across Ireland most days of July 2010, associated with frontal systems moving eastwards over Ireland, as unusually deep depressions for July tracked close to the west coast. On 10 July maximum temperatures were 16-20℃ and winds became stronger through the day. A band of persistent rain in the south of the country during the morning spread northwards to affect all areas by afternoon. Further heavy thundery pulses moved up from the south during the afternoon and evening, producing extremely heavy falls in the west. The rain cleared from the southwest by evening.

The highest rainfall in the country during this event was recorded at Claremorris. At both Claremorris and Knock Airport rain was particularly heavy from 6-9pm. Over a 3-hour duration the AEP was 2.2% at Claremorris (a return period of 50 years) and 7% at Knock Airport.

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