

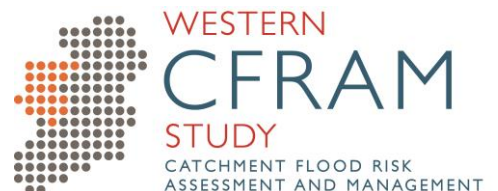


Western CFRAM Unit of management 34 - Moy and Killala Bay Hydrology Report

Final report

September 2017

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



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Contract

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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) 34, the Moy catchment and Killala Bay.

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 34 there are five AFAs, all subject to fluvial flood risk. These are Ballina, Foxford, Charlestown, Swinford and Castlebar. Flood risk in Ballina is also influenced by tide levels, as the Moy is tidal to the centre of Ballina.

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, although some of these measure only river level and do not have a rating equation that enables conversion of level into flow. At two gauging stations in UoM 34, Rahans and Ballycarroon, 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 been completed at the time of writing and so it has been necessary to develop software to apply some of the methods.

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.

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.

Further refinement of the hydrological analysis has been carried out for Foxford, following the December 2015 flooding in Foxford. A flow relationship between Foxford and Rahans gauges has been developed as a means of extending the AMAX record series to include peak flows after the Foxford gauge became inactive in 2013, and so include an estimate of the December 2015 peak flow in the flood frequency estimation process. The December 2015 peak flow was the highest recorded at a number of gauges on the River Moy catchment and also on other gauges in the Western River Basin District. It is likely to also be the highest flow since the Foxford gauge was commissioned and so has a significant bearing on the flood growth curve used to derive design event peak flows. The purpose of extending the length of the AMAX record series is to reduce the uncertainty in these hydrological estimates.

Methods used to estimate design flood hydrographs at each AFA

AFA	Name	QMED method	Growth curve method	Distribution	Hydrograph shape
Ballina	Moy	Data Transfer – Pivotal using 34001 at some HEPs and 34001 and 34003 at others	Pooled	General Logistic	Hydrograph Width Analysis - 34001
	Ardnaree, Glenree, Ballina, Bunree, Quignamanger, Quignalecka, Knockanelo, Knockleitaugh, Tullyegan	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
Castlebar	Castlebar	Data Transfer – Pivotal 34018	Pooled	General Logistic	FSR subject to re-assessment during modelling
	Knockthomas, Springfield, Saleen, Saleen lake stream, Knockrawer, Milebush	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
Charlestown	Mullaghanoë	Catchment Descriptors (altered from Data Transfer – Pivotal 34031)	Pooled	General Logistic	FSR rainfall-runoff with $T_p(0)$ adjusted from lag analysis
	Sargirra	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
	Black River	Catchment Descriptors (altered from Data Transfer Pivotal 34031)	Pooled	General Logistic	FSR rainfall-runoff
Foxford	Moy	Data Transfer – Pivotal 34003, with extension of Foxford AMAX record series with Rahans gauged data,	Single Site	General Logistic	Hydrograph Width Analysis - 34003
	Foxford, Rinnananny	Catchment Descriptors	Pooled	General Logistic	FSR rainfall-runoff
Swinford	Swinford, Newpark	Catchment Descriptors (altered from Data Transfer – Pivotal 34031)	Pooled	General Logistic	FSR rainfall-runoff with $T_p(0)$ adjusted from lag analysis

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.

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 34 should be read in conjunction with the Hydraulic Modelling Report for UoM 34, 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 Western 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 (including historic event analysis) 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 Unit of management (UoM) 34. Chapter 7 summarises the approach that has been taken for design flow estimation at each AFA in UoM 34. 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 34 should be read in conjunction with the Hydraulic Modelling Report for UoM 34, 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 34, as will the assessment of the joint probability of fluvial and coastal flooding.

1.4 Unit of management 34 - Moy and Killala Bay

Unit of management 34, shown in Figure 2-1, also referred to as Moy and Killala Bay, covers an area of 2,314 square kilometres of the Western RBD. The area is predominantly within County Mayo but there are also some small areas of County Sligo and north County Galway included.

Figure 1-1: Unit of management 34: Moy and Killala Bay - overview map



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The Flood Risk Review identified five Areas for Further Assessment (AFAs) in UoM 34. These are:

1. Ballina
2. Castlebar
3. Charlestown
4. Foxford
5. Swinford
6. Crossmolina

The CFRAM for UoM 34 is focusing predominantly but not exclusively on the first five of these areas, along with the MPW reaches which are principally on the Moy and Castlebar Rivers. The flood relief scheme for Crossmolina has been advanced ahead of the wider Western CFRAM study since this area has suffered from recent high flood damages. The scheme works began with an OPW feasibility study which is being expanded upon to include hydraulic modelling and options appraisal. Should a cost-beneficial scheme be identified, the works will advance to construction stages. The Crossmolina study, being undertaken by JBA under subcontract to Ryan Hanley, is

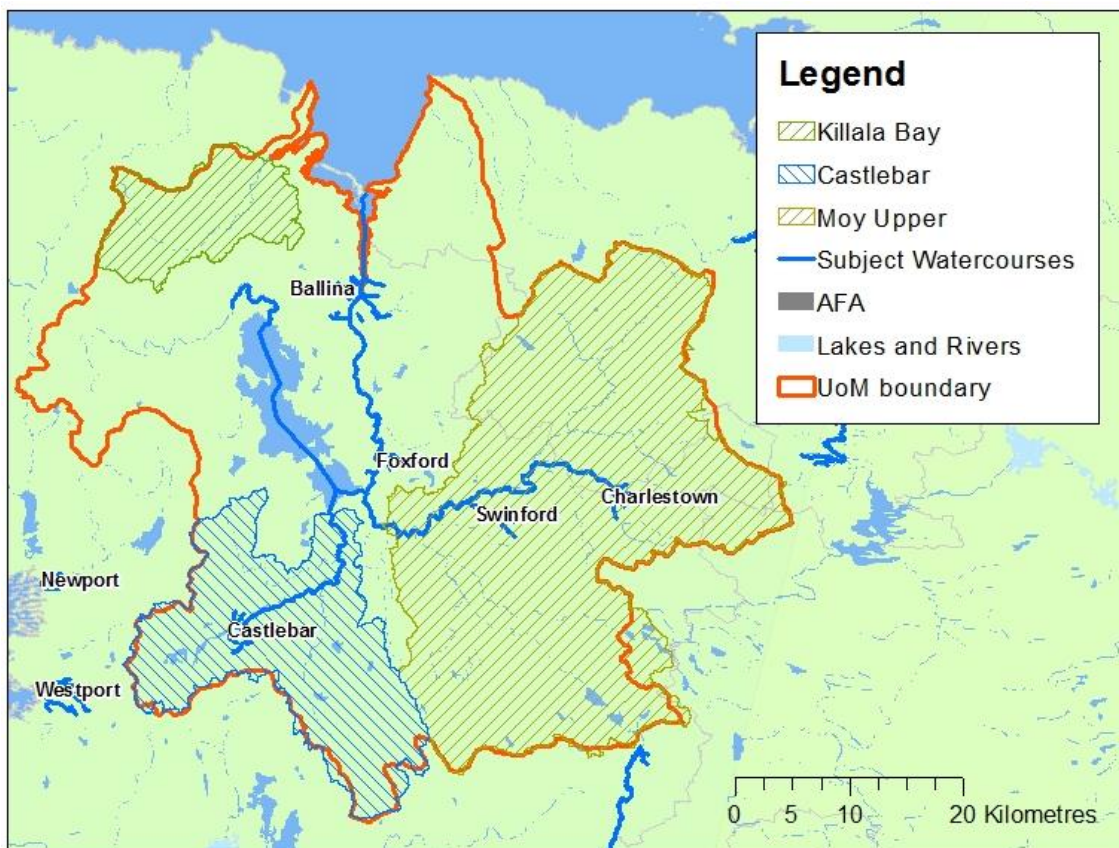
using methods developed for the Western CFRAM, and in particular the hydrology calculations prepared by the OPW in the feasibility report have been revised in accordance with the methods detailed in this Hydrology Report.

2 Hydrology of the study area

2.1 Catchments

The whole unit of management 34 forms a single catchment, the Moy, with the exception of a number of small catchments to the north, which drain into Killala Bay; these catchments are not subject to assessment under the CFRAM. Ballina lies at the mouth of the River Moy where it enters Killala Bay. The map below illustrates two of the main sub-catchments plus the northern catchment at Killala Bay.

Figure 2-1: Subject catchments in UoM34



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. 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 Report¹ and further details of each specific watercourse can be found in the WCFRAM Hydraulic Modelling Report for UoM 34.

The following maps detail the watercourses within each of the AFAs within the catchment.

¹ 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 2-2: Castlebar AFA watercourses



Figure 2-3: Charlestown AFA watercourses



Figure 2-4: Swinford AFA watercourses

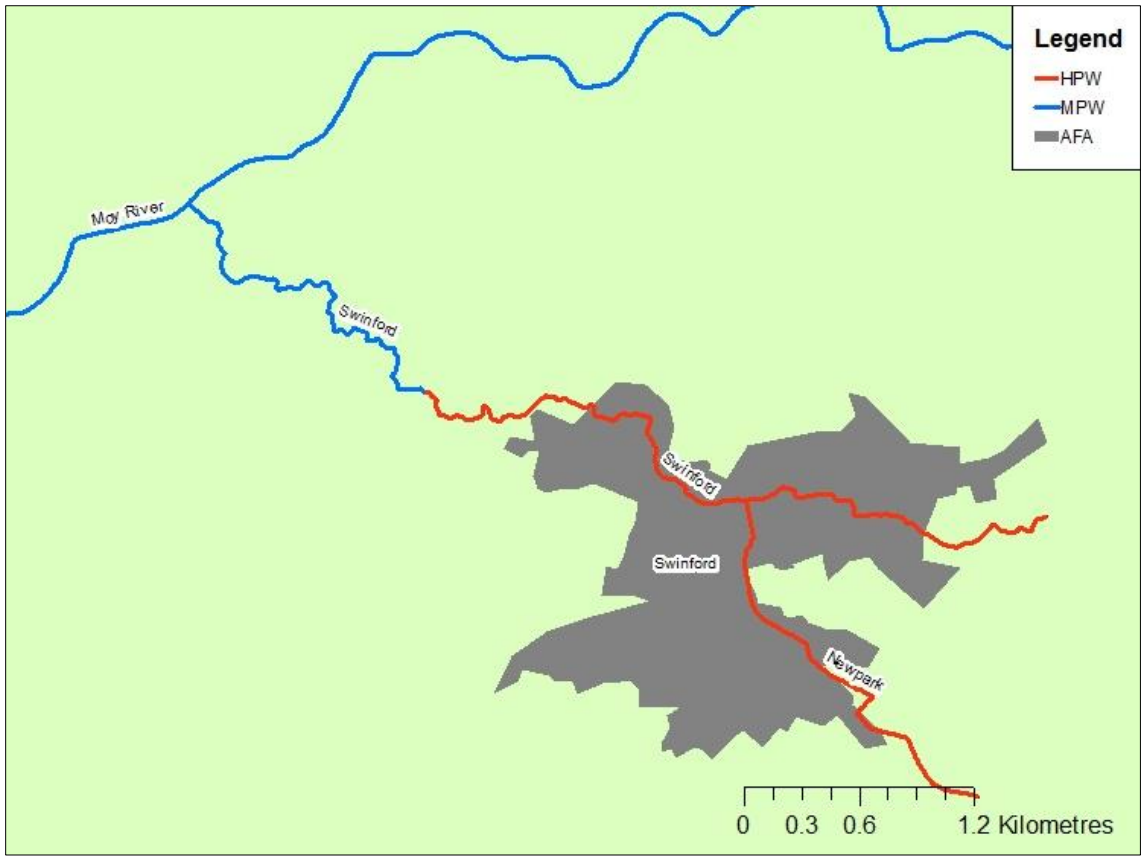


Figure 2-5: Foxford AFA watercourses

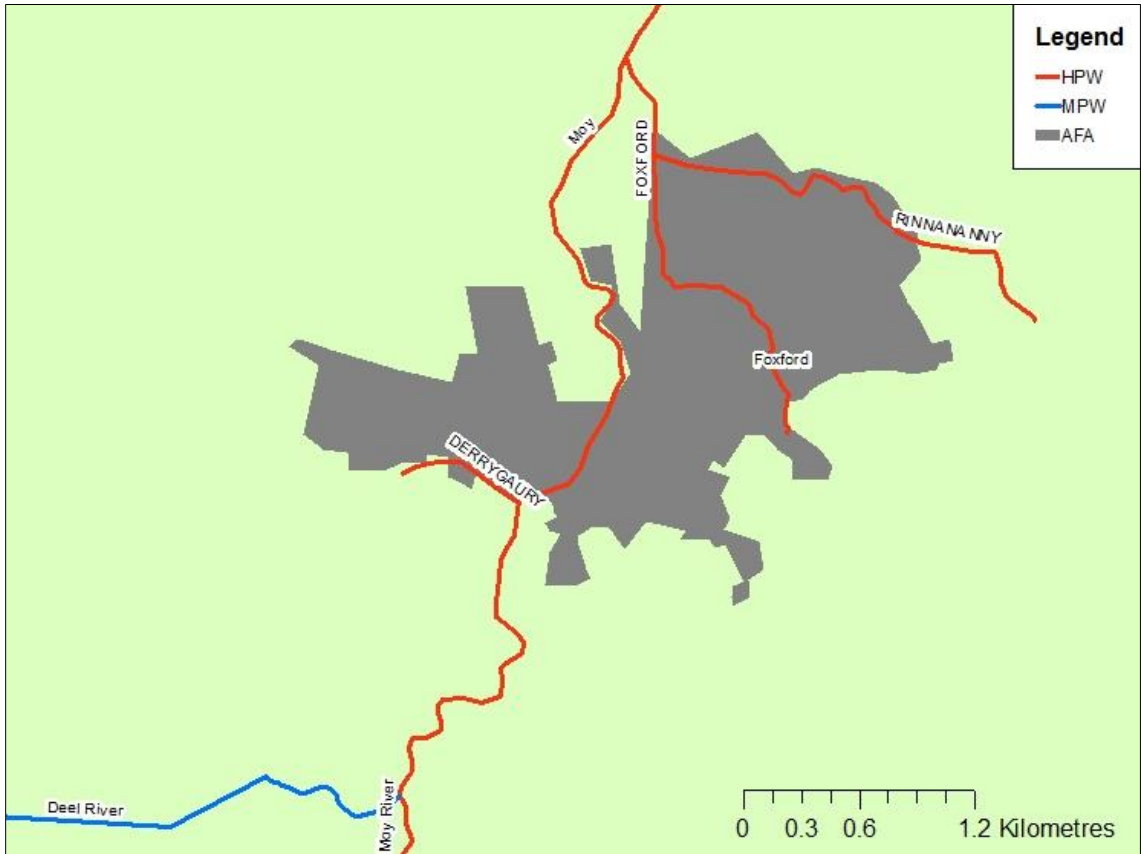
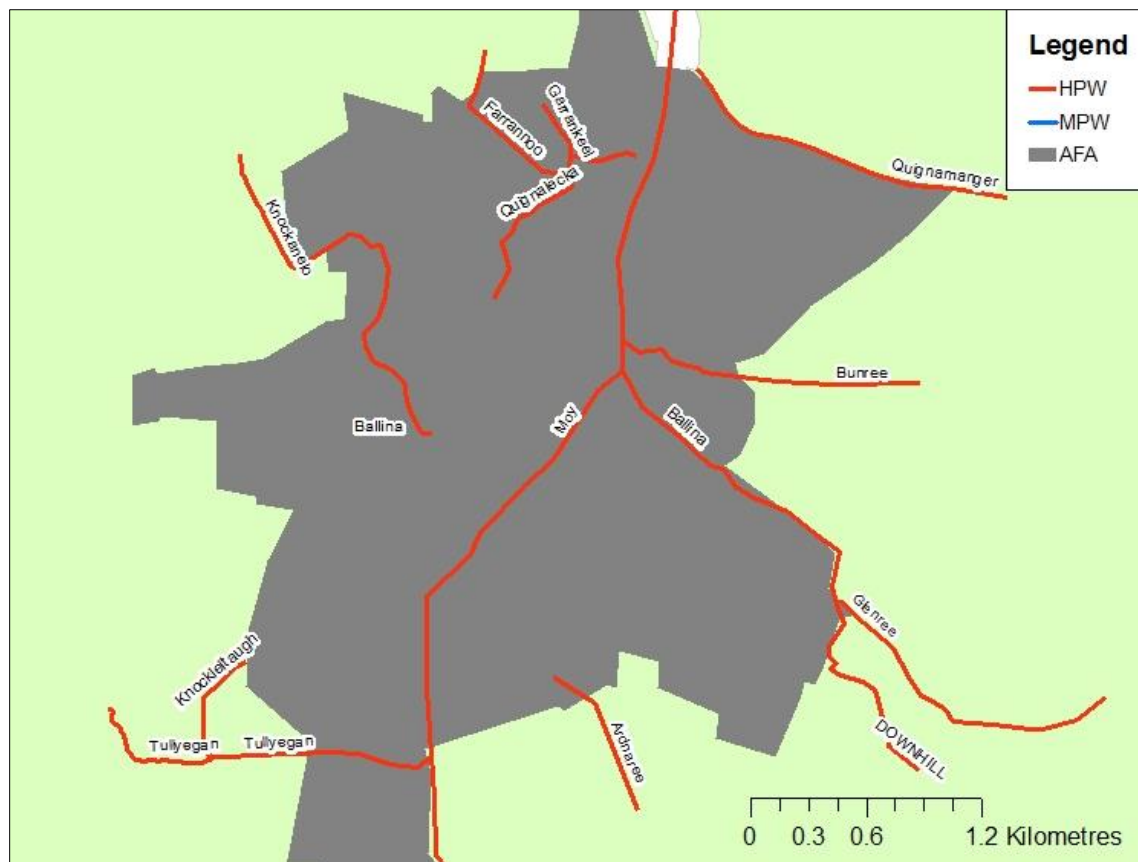


Figure 2-6: Ballina AFA watercourses



The Moy River forms the majority of unit of management 34; its catchment is approximately 1,980km², which is around 85% of the UoM. The catchment areas to the downstream limit of the Castlebar, Charlestown, Swinford, Foxford and Ballina AFAs are 89km², 38km², 18km², 1,795km² and 1,980km² respectively. The catchment includes numerous areas of higher elevation, including the Ox Mountains to the east and the Nephin Beg Range and Croaghmoyle to the south west. The gradient of the watercourse as a whole (S1085) is 0.73 m/km, which is low, despite the influence of steeper channels in the upper reaches of the catchment. In the upper catchment north of Ballyhaunis there is a discrepancy between the UoM boundary and the catchment boundaries supplied by OPW (as shown in Figure 2-1). In this location it appears that the UoM boundary is correct, as explained in the Inception Report.

The Moy River has a reach length of approximately 52 kilometres from its confluence with the Mullaghanoe River. Upstream of this point the Moy rises in the Ox Mountains above the town of Cloonacool. Its principal tributaries from its source are the Mullaghanoe River, the Swinford River and the Clydagh River.

The mean annual rainfall is 1300mm. The rainfall is generally higher in the mountainous areas as would be expected. Mean annual rainfall at Cloonacool is 1640mm. The mean annual rainfall to the downstream limit of the Castlebar, Charlestown, Swinford, Foxford and Ballina AFAs is 1565mm, 1280mm, 1235mm, 1340mm and 1320mm respectively.

The bedrock geology of the Moy catchment is a complex mixture of various geology types. Two areas, around Castlebar and Ballina are predominately Carboniferous Limestone. Between these two locations runs a band of complex geology including Precambrian rocks, Ordovician to Devonian Granite, Carboniferous Sandstone and Shale and also Cambrian Sandstone and Slate. Most of the upper catchment is covered with deep poorly drained mineral soils, with some areas of peat and deep well drained mineral soils. The lower catchment around Ballina is mostly underlain by deep well drained mineral soils. The BFI as predicted from soil characteristics is 0.78, indicating a significant degree of soil permeability.

The catchment includes two major water bodies; Lough Conn and Lough Cullin, the latter of which the Clydagh River drains into. The FARL value of the entire catchment is 0.823.

The catchment is rural but has a number of larger settlements including Ballina, Castlebar and Charlestown.

An arterial drainage scheme for the Moy catchment was carried out in 1960-71. The effects of the scheme can be seen in some of the longer flood peak records, as discussed in Section 5.1.

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 1km².

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

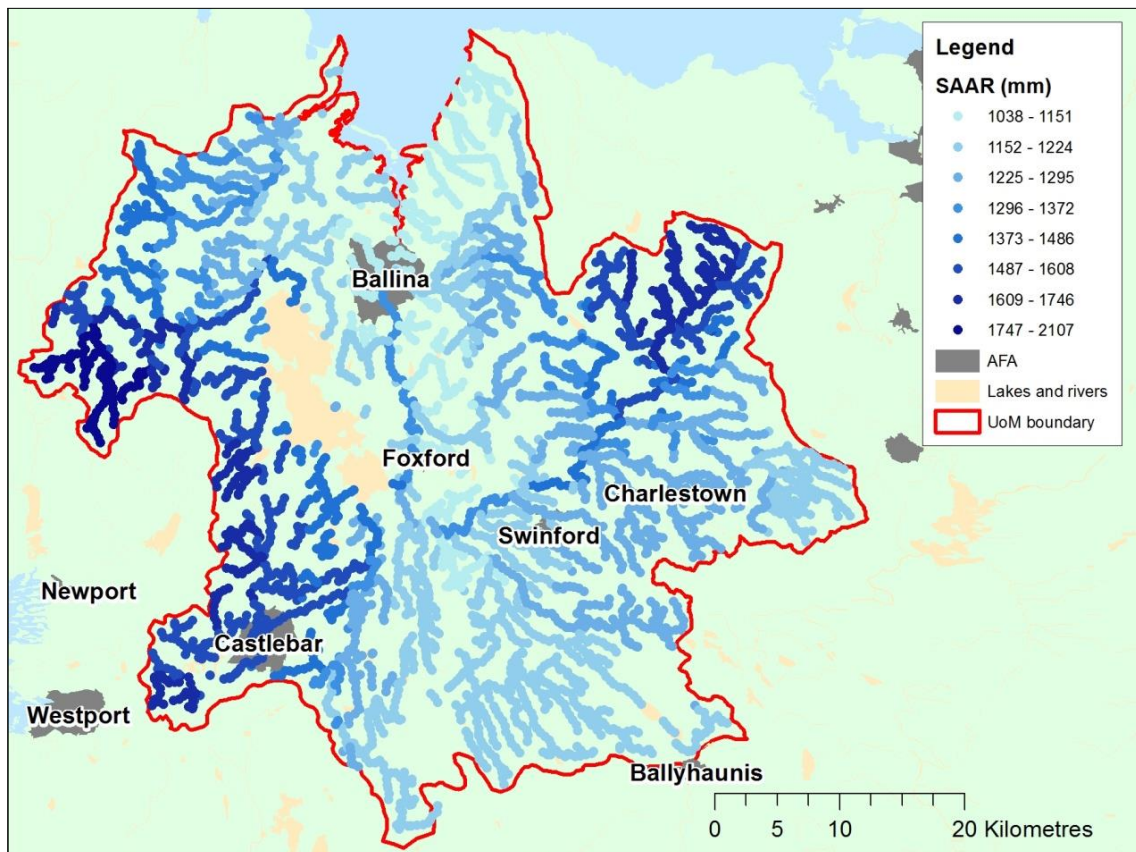


Figure 2-8: Baseflow index estimated from soil properties, BF_{SOIL}

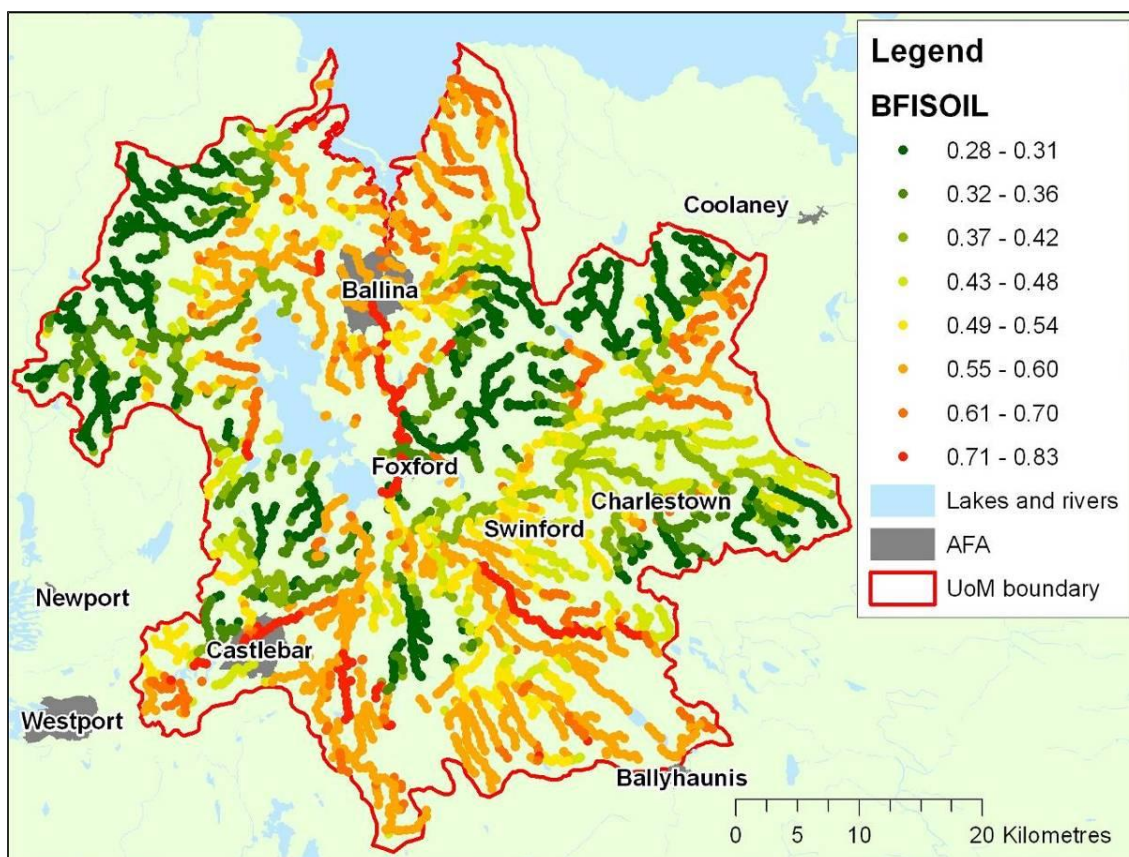


Figure 2-9: Slope of the main watercourse in the catchment, S1085

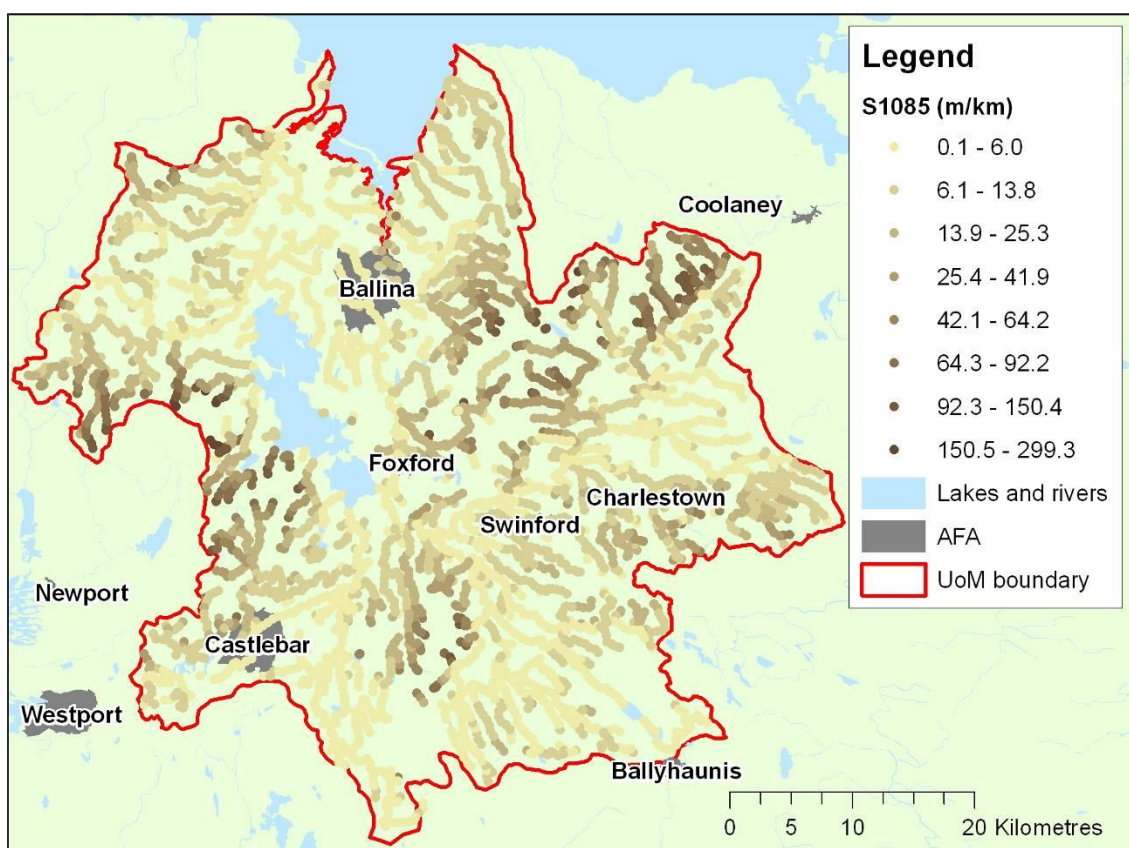
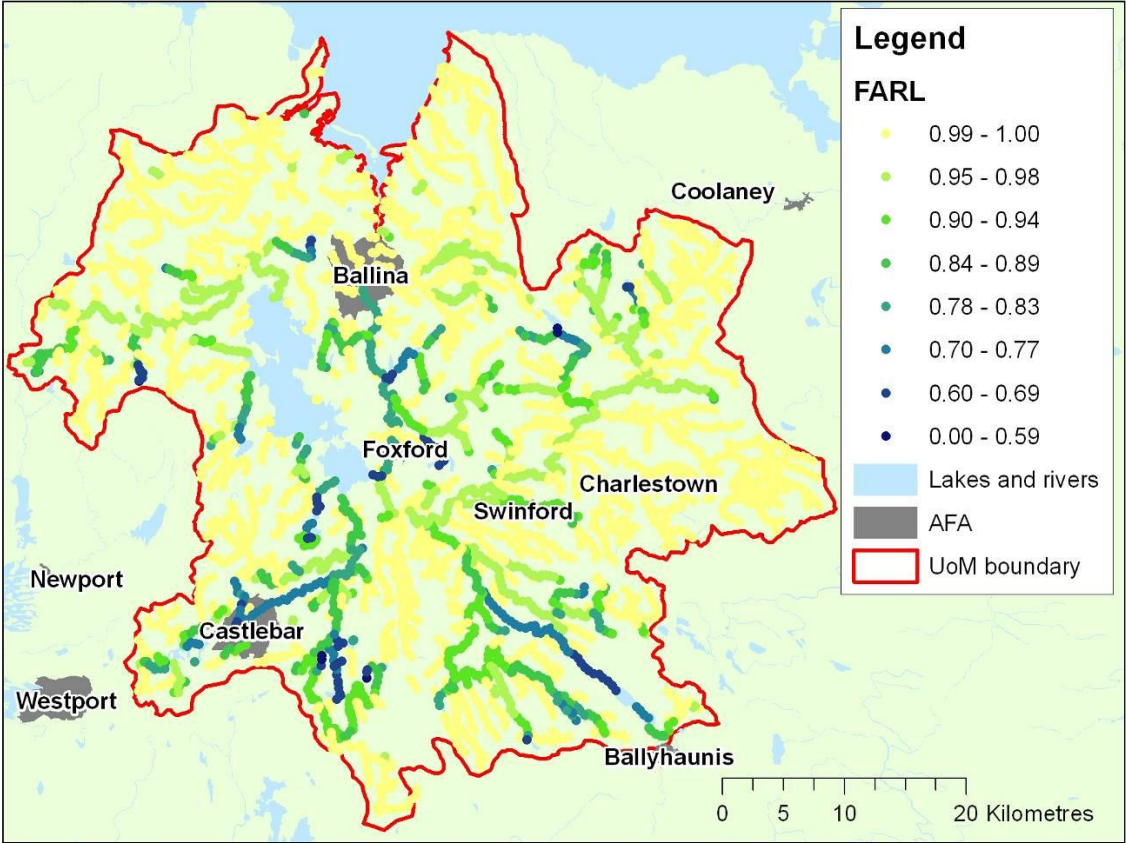


Figure 2-10: Flood attenuation by reservoirs and lakes, FARL



3 Hydrological data

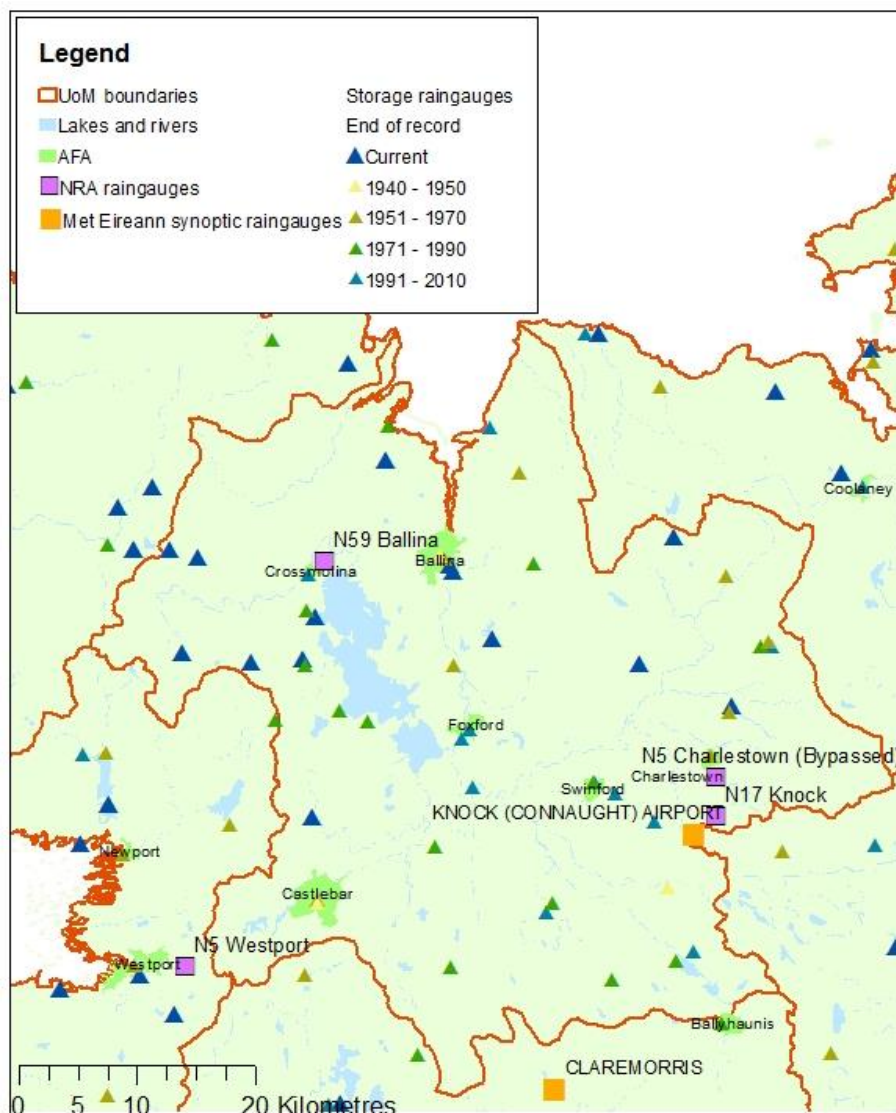
3.1 Meteorological data

Figure 3-1 shows raingauges (past and present) for which digital data is held by Met Éireann or the National Roads Authority (NRA) within this unit of management. Met Éireann operate just one synoptic raingauge (i.e. a recording gauge that measures rainfall at a sub-daily time step), at Knock Airport on the eastern boundary of the area. There is also another just outside the study area to the south, at Claremorris. Data from both gauges has been analysed for this study.

Data from all the gauges shown has been provided by Met Éireann. Some of the gauges have digital data available from the 1940s. Knock Airport synoptic gauge has data from 1996.

Analysis of the rainfall data, from synoptic sources, tipping bucket gauges and storage gauges, is described in Appendix G (taken from 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, including at sensors within UoM 34 near Ballina and Charlestown. Information on this dataset was provided after completion of the inception phase and so it has not been incorporated in the analysis of rainfall events. It was, however, 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 16 river level gauges that have been judged as potentially useful for this study, i.e. either on rivers that are to be modelled or nearby gauges with good quality flood peak datasets that represent potential donor sites. At nine of these gauges it is possible to calculate flow from the observed water levels using a rating equation. At a tenth gauge, Banada (34013), it has been possible to develop an approximate rating using available flow gaugings. Two of the gauges, Rahans (34001) and Ballycarroon (34007), have been identified for review and extension of rating equations within this study, as described in the following section. 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	Catchment Area (km ²)	Start of record	End of record	Flow available ?	FSU quality class	Comments
34001	RAHANS	1974	1968	-	Y	A2	Rating review site
34003	FOXFORD	1805	1976	2013	Y	A2	Relationship between Foxford and Rahans gauge established to extend AMAX record series beyond the end of the record in 2013.
34004	BALLYLAHAN	935	1954	-	Y	C	FSU spreadsheet has pre-drainage AMAX from 1954-59. These are not relevant to present-day conditions.
34005	SCARROW-NAGEERAGH	309	1952	-	Y	A1	AMAX available only for 1952-64. No rating for recent period and most level data for 1978-2007 is missing.
34007	BALLY-CARROON	152	1952	-	Y	B	Rating review site but no AFA here now.
34010	CLOONACAN NANA	484	1953	-	Y	B	AMAX available only to 1966, pre-drainage.
34011	GNEEVE BRIDGE	143	1975	-	Y	A2	AMAX only to 2003, when the weir was removed. No post-2003 rating developed yet.
34013	BANADA	174	1952	-	Approx.	n/a	Approximate rating fitted to gaugings - see text below.
34018	TURLOUGH	95	1976	-	Y	A1	
34021	SWINFORD	18	2002	-	N	n/a	
34031	CHARLES-TOWN	23	1997	-	Y	n/a	Gauged up to 6m ³ /s. QMED is 11m ³ /s and highest flow on record is 19m ³ /s so considerable extrapolation.
34061	BALLINA	1984*	1968	-	N	n/a	Continuous data from 2007.
34071	POLLAGH	976	2007	-	N	n/a	
34072	ISLANDEADY L.	59*	1983	1996	N	n/a	
34073	LANNAGH	80*	1976	1990	N	n/a	
34074	CORLUMMIN	819	1976	2009	N	n/a	

*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.
4. All gauges with flow available have rating equations and check gaugings. All gauges listed have annual maximum series.
5. 34001, 34004, 34005, 34007, 34010, 34011, 34013, 34018, 34061, 34071 are operated by OPW. Others are operated by local councils.

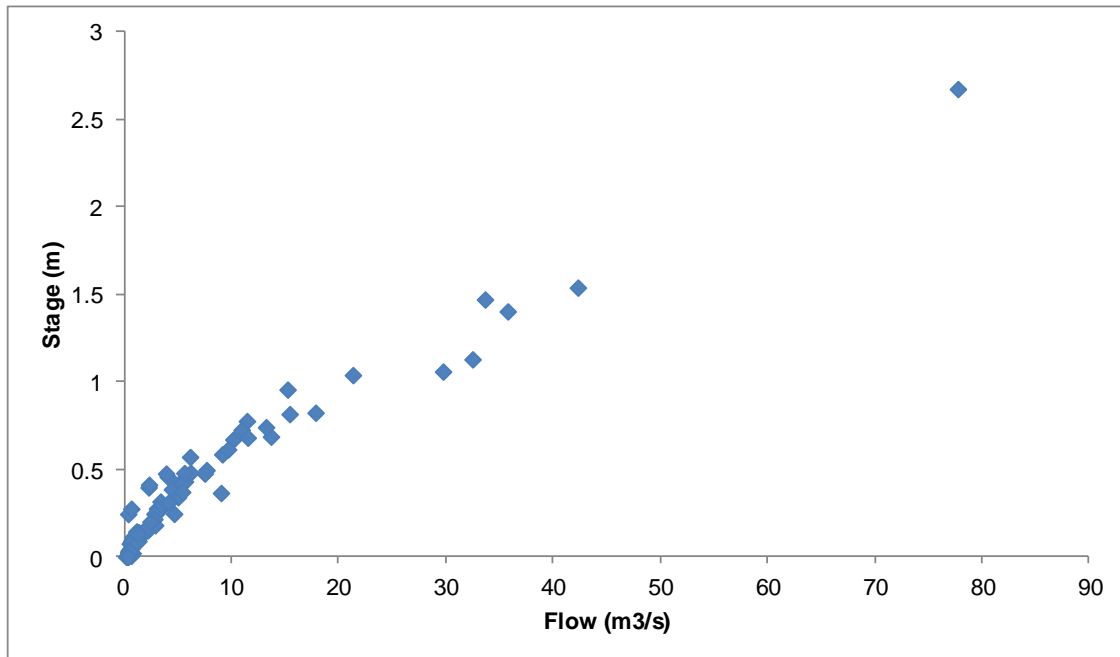
At gauge 34013, Banada, there is a long record of annual maximum levels from 1952. No rating has been developed for this site, but there are flow gaugings available (Figure 3-3). They show little scatter up to around 1.5m stage. The median annual maximum level is around 2.5m. An

indicative annual maximum flow series has been developed by fitting a line through these gaugings:

$$\log(\text{flow}) = 1.6909 \log(\text{stage}) + 2.9985$$

This must not be regarded as a formal rating equation, but it at least enables comparisons of peak flows at Banada with those measured at other gauges on the River Moy. The resulting peak flow series was used to help characterise the hydrology of the catchment but was not included in the analysis underpinning the estimation of design flows.

Figure 3-3: Flow gaugings at Banada



All AFAs benefit from nearby gauging stations although not all gauges provide flow data:

- At Ballina there is gauge on the River Moy, Rahans (34001), with peak flows available from 1968 to date. The rating has been derived from a large number of check gaugings up to around QMED but shows considerable scatter at high flows. It was not possible to improve on this rating during the rating review.
- At Foxford there is a gauge on the River Moy (34003), with peak flows from 1976 until 2013. The rating equation was classed as A2 in the FSU research so it had been assumed that the flood peak data are reliable for flood frequency analysis. This gauge is part of the EPA monitoring network and since the FSU analysis a more recent rating review has been carried out by the EPA which incorporates high flow gauging during November 2009. At the time of writing the FSU data has not been updated with the latest rating relationship.
- At Castlebar there is a level-only gauge at Lannagh (34073) with data available between 1976 and 1990. Rather more usefully, 7km downstream of the town is a flow gauge at Turlough (34018) with data from 1976. The rating equation was classed as A1 in the FSU research so it has been assumed that the flood peak data are reliable for flood frequency analysis.
- At Swinford there is a level gauge (34021) with data from 2002. There is no flow data available for this watercourse. However, there are several nearby gauged catchments which may provide useful donor sites.
- At Charlestown there is a flow gauge (34031) with a short record from 1997 to date. The rating is extrapolated considerably even for QMED so the flow data has limited value for flood estimation. However, it will not be rejected out of hand because it may be that QMED estimated from the flow data is less uncertain than a generalised estimate made from catchment descriptors.

Thus there are no major data gaps associated with the hydrometric network. However, for any future improvements to the design flood estimates it would be useful if flood flow ratings could be developed for the gauges at Swinford and Charlestown.

3.3 Review of rating equations

Two gauges in UoM 34 have been identified by OPW for rating reviews: Rahans (34001) and Ballycarroon (34007). The results of the rating reviews can be found in Appendix A.

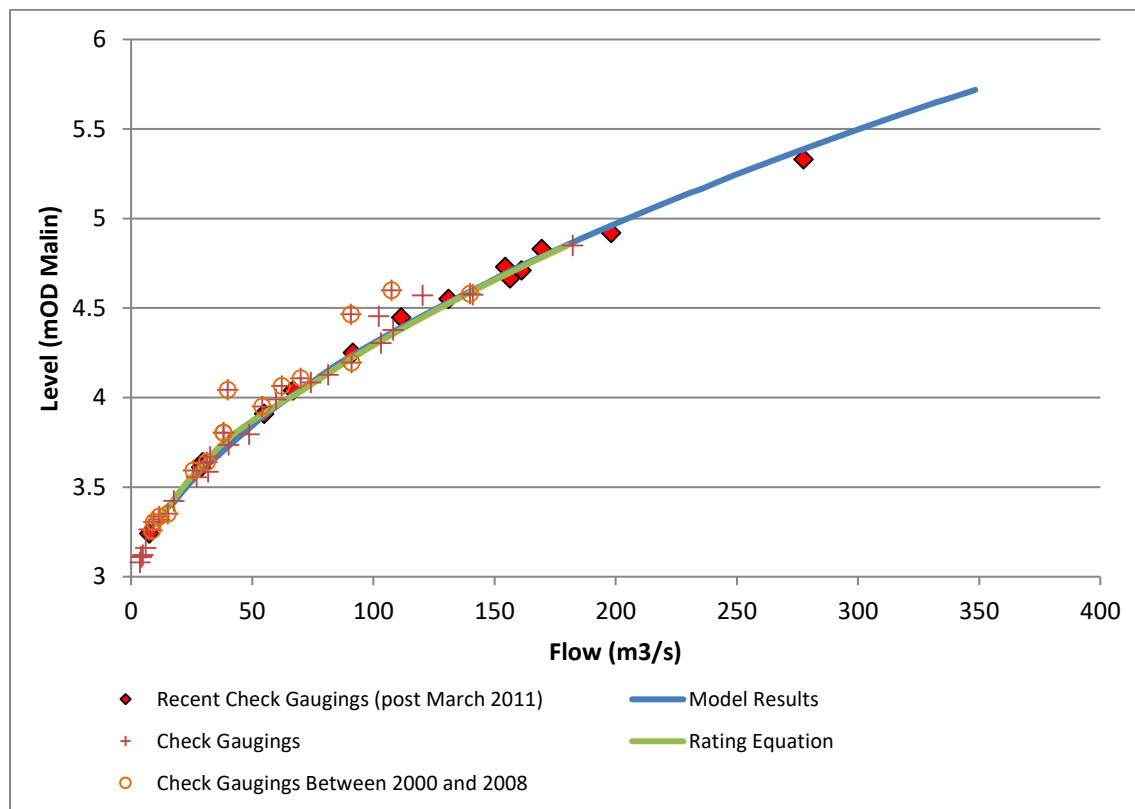
Both gauging stations have been visited in order to assess the physical characteristics of the river channel and floodplain such as hydraulic controls on water level (at low and high flows), hydraulic roughness and potential bypass routes in flood conditions. Existing rating equations have been assessed 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.

The development of hydraulic models generally provides an opportunity to extend the rating equations above the range of flows for which check gaugings are available. However, it was not possible to extend either the Rahans or Ballycarroon ratings.

At Rahans there is not enough confidence in the understanding of the hydraulics to enable extension of the rating using the model. Safety concerns have prevented survey of the crest of the large natural weir located immediately upstream of the salmon weir. This structure is the largest control of upstream water levels. There are also uncertainties over the implications of blockage through the three channels by the side of the salmon weir. As explained in Appendix A, it is recommended that the existing rating is used for calculation of flows for the purposes of the Western CFRAM. Spot flow gaugings since 2011 at Rahans (34001) have been plotted against the stage discharge relationships adopted in the CFRAM hydrology. For Rahans (Figure 3-4) the highest spot flow gaugings are marginally below the flow rating curve as extended by the calibrated hydraulic model. The highest spot flow gauging was carried out on 06 December 2015 close to the peak of the flood event which affected Foxford.

There is no evidence to suggest that the flow ratings for Rahans should be revised.

Figure 3-4: Rahans (34001) Rating equation and recent spot flow gaugings



The development of a hydraulic model at Ballycarroon was substantially delayed due to the presence of freshwater pearl mussels which held up the survey. The rating review, along with estimation of design flows at Ballycarroon and Crossmolina, was carried out separately to the CFRAM within the Crossmolina Flood Relief Scheme study. However, for completeness, the results of the rating review are included in Appendix A. The impact of the 2015 flooding and recent spot flow gauging at Ballycarroon is being addressed in the development of the Crossmolina Flood Relief Scheme. This work is in progress and any change will not have any influence on the flood hydrology on the River Moy, due to the influence of Lough Conn and Lough Cullin.

Rating equations at other gauging stations are available from the operator of the station, i.e. OPW or local authorities.

Revisiting the gauge records following the December 2015 flooding in Foxford, highlighted that the EPA had updated a rating review for the Foxford gauge (ref 34003) to incorporate flow gauging during November 2009. This flow gauging was the highest flow gauged and has subsequently been applied to the full AMAX record series. This EPA rating review had not been used to update FSU data and so was not applied to the CFRAM hydrological analysis or hydraulic modelling until the update that included the data available from the 2015 event.

At Ballylahan (34004), the OPW have collected recent spot flow gaugings since 2011. These have been used to develop a provisional flow rating, which is the same as the flow rating developed in 2001 by the OPW. There is out of bank flow which bypasses the gauge and so there is significant uncertainty in the flow estimates.

3.4 Tidal data

Figure 3-5 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 3-5: Tidal gauge locations

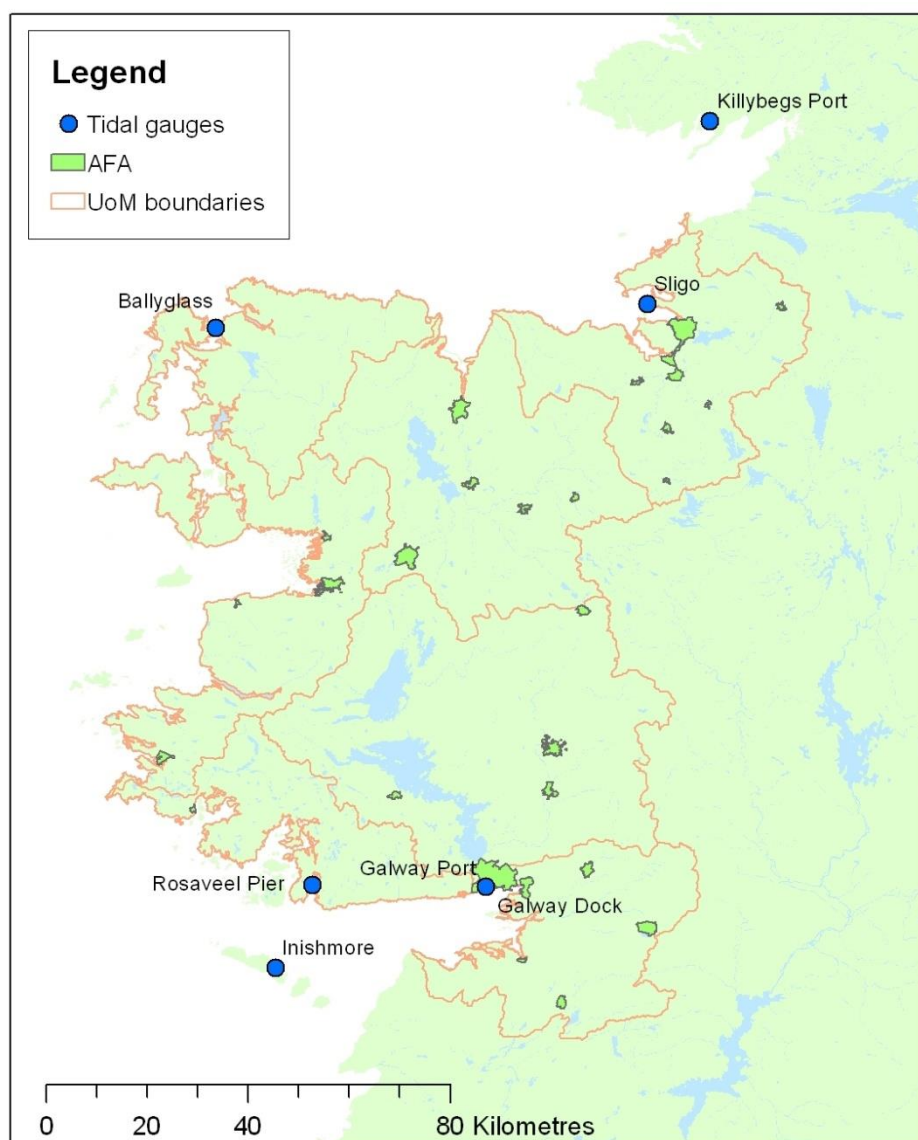


Table 3-2 Summary of tidal gauges

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.

Only limited information on historical flooding, which includes some indication of the magnitude and/or extent of the flood, was available for UoM 34. 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 Tribute 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 the Inception Report.
- 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.

Mayo County Council have provided information on recent flooding through the CFRAM public consultation process. This has been incorporated into the hydrological and hydraulic analysis.

Analysis of the historical 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%. There are five AFAs in UoM 34.

Design flows are needed at a number of locations on rivers, called Hydrological Estimation Points (HEPs). Estimation of design groundwater conditions is not required, as groundwater flood mapping is being covered in a separate nationwide study. Design flows are needed for:

- The River Moy at Ballina, Foxford and other locations within the extent of the MPW model. Flows will also be needed for small ungauged tributaries of the Moy within Ballina and Foxford. The most significant tributary is the Glenree/Brusna River in Ballina. Refer to Figures in Section 2.
- The Castlebar River and tributaries at Castlebar. There are several small tributaries, some affected by lakes and some with fairly urban catchments. Refer to Figures in Section 2.
- The Swinford River and small tributaries at Swinford. Refer to Figures in Section 2.
- The Mullaghanoe River and tributaries at Charlestown. Refer to Figures in Section 2.

The specification calls for hydrological estimation points (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 has been completed it is not possible to ascertain which tributaries contribute 10% of main channel flow; therefore HEPs 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 as 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 several quite distinct types of catchment for which design flows are needed. On the Lower Moy, 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., 1993²). A consequence is that accurate estimates of flood durations and volumes will be important on these catchments.

On the Castlebar River it appears possible (subject to the findings of the hydraulic model) that flood flows and levels may be affected by backwater effects from Lough Cullin, resulting in prolonged hydrographs despite the small size of the catchment. It is also possible that there are groundwater influences which may also contribute to the prolonged hydrograph.

In contrast, the tributaries of the Upper Moy at Swinford and Charlestown are 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.

There are gauging stations in or near to most AFAs, so 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 by data transfer using upstream or downstream gauges as donor sites where possible. Since the flow data at Foxford and Ballina implicitly account for the effects of the major lakes in the catchment it is not necessary to carry out flood routing calculations in order to estimate design flows on the River Moy.

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 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 removing the adjustment to QMED in Charlestown and Swinford, where the appropriateness of donor sites was less clear. In addition, for all HEPs, the FSR rainfall-runoff method was applied to estimate the gradient of the upper portion of the growth curve for return periods in excess of 100 years.

A variety of methods for defining characteristic flood hydrographs have been tested. These included:

- Deriving a characteristic hydrograph using the parametric method from FSU Work Package (WP) 3.1, in which a hydrograph (standardised to a 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.

Section 6 describes how the preferred methods have been chosen.

² Anderson, R.J., dos Santos, N. and Diaz, H.F. (1993) An analysis of flooding in the Parana/Paraguay River Basin. Dissemination Note 5. Latin America and Caribbean Tech. Dept. Environment Div. 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 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 a peak flow record with a good quality. Some additional gauges were analysed during the inception phase, but the analysis described in Appendix B focuses primarily on gauges that have been used to estimate design flows for AFAs, excluding Ballycarroon which is being covered in the separate Crossmolina Flood Relief Scheme study.

The magnitude of estimated design flows is 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), summary statistics for the largest floods and discussion of the data. The appendix also includes an analysis of flood volume data at one gauge, Rahans (34001), chosen because of the extremely long-duration floods that are observed on the Lower River Moy. The volume frequency analysis has been used to provide an additional check on design flood hydrograph shapes at Ballina (Section 6.6).

Most flood peak records in the Moy catchment date back to the 1970s or late 1960s. There are three longer records, back to the early 1950s, on the Moy at Ballylahan (near the centre of the catchment), the Moy at Banada (in the eastern headwaters) and the Deel at Ballycarroon (a tributary of Lough Conn, in the west of the catchment). The project included an investigation of the feasibility of extending the flood peak record in the lower catchment back to earlier years. For example, the gauge on the Moy at Rahans was apparently installed in 1939 and yet flow data are available only from 1968, when the gauge was automated. There is also a gauge shortly downstream of Rahans at Bachelors Walk, Ballina, which has level data on charts available from 1952. The value of this record is limited because the gauge is affected by tides and there is only one flow gauging available for the period before 1968. It was concluded that the value of earlier flood data on the Moy would be limited because of the arterial drainage scheme which covered the whole catchment between 1960 and 1971. Evidence of the effects of the scheme can be seen in the flood peak records at Banada and Ballylahan. At Banada there is a distinct increase in annual maximum flows after approximately 1966.

The December 2015 flooding was the highest peak flow recorded at Rahans (34001) and Ballycarroon (34007). The Foxford gauge has been inactive since 2013, but it is estimated that the highest peak flow since the gauge opened would also be the December 2015 event. The peak flow in Rahans in December 2015 is in the order of the 2% to 1% AEP.

At many gauges, the second highest peak flow on record was that of late October 1989. It was the most significant in the lower Moy, where at Rahans the estimated AEP was just under 2%. On the Castlebar River at Turlough, the November 2009 event was more severe than October 1989. In the headwaters of the Moy, at Banada, the August 2008 flood was the highest. The catchment at Banada is small and relatively steep and so will be sensitive to shorter-duration intense rainfall of the type which occurred in August 2008. Please refer to Appendix B for estimated AEPs of notable floods at a range of gauging stations.

In terms of flood volumes, the most severe event on record at Rahans was in February 1990 when the analysis is carried out using volumes accumulated over either four or eight weeks.

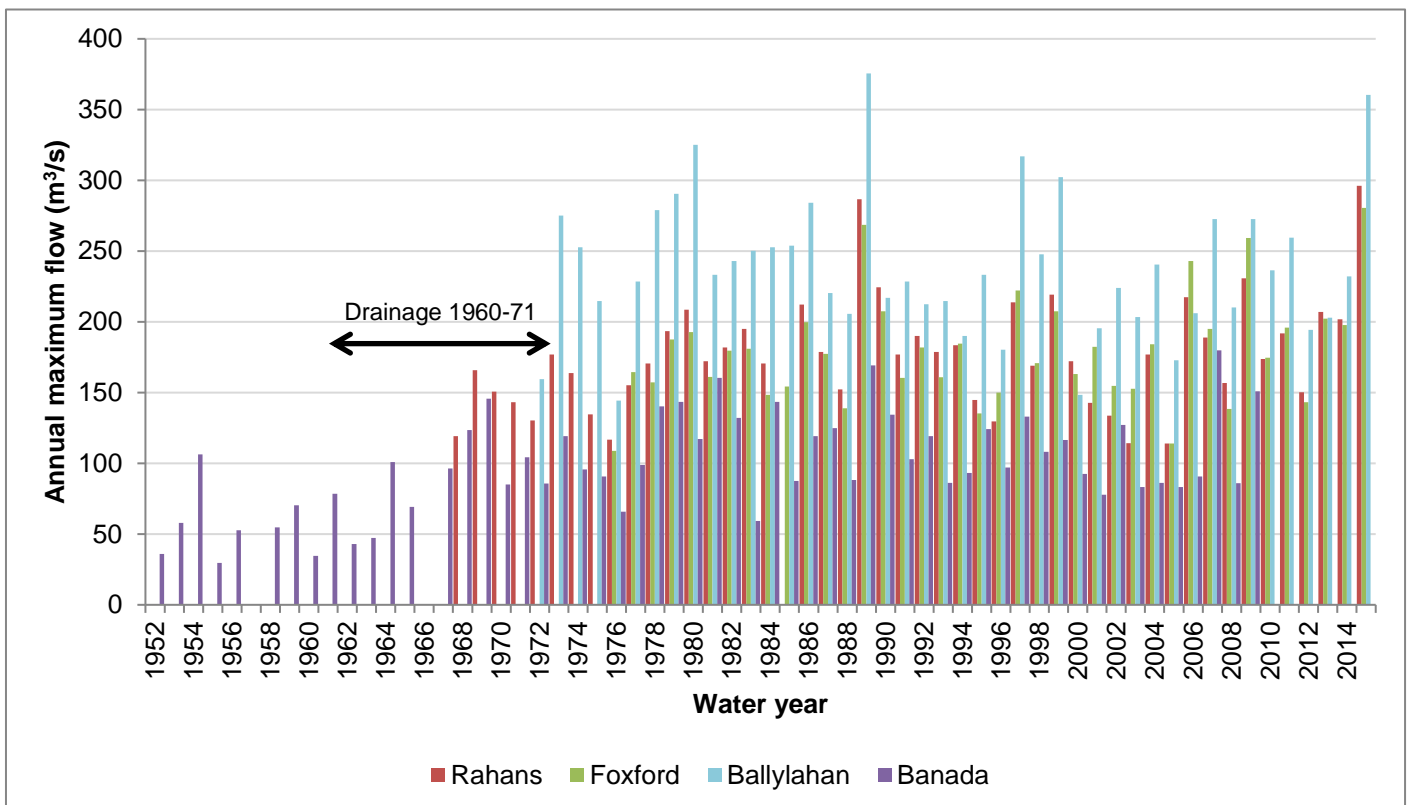
The vast majority of annual maximum floods occur in the autumn and winter. The gauges at Banada and Ballycarroon, on smaller and steeper catchments, show a wider seasonality with some major floods in the summer or early spring.

A comparison of flood peak series at several gauges is shown in Figure 5-1 (note Foxford AMAX data since 2013 is based upon the relationship between flows at Foxford and Rahans gauges). The stations are ordered from the downstream end of the River Moy, at Rahans, up to Banada in the headwaters. Foxford is shortly downstream of the outlet of Lough Cullin and Ballylahan is shortly upstream. The dominance of the 1989 flood can be seen at most gauges. The flow values

shown on the graph for Banada are derived from an approximate rating fitted to the 90 flow gaugings available since 1952, since no official rating has been developed for this gauge. The Banada peak flows have not been extended for the latest data.

Peak flows at Rahans are generally similar to those at Foxford. This makes sense because the catchment area increases by just 10% between these gauges, and there will be some attenuation of peaks via floodplain storage along the Lower Moy. In some events Foxford has a higher peak than Rahans and in other events this reverses with higher flows recorded at the upstream Foxford gauge. Preceding catchment conditions and wetness are likely to influence conveyance and storage. There is sufficient length of record for a relationship to be derived. Further details on the relationship between these two gauges and the extension of the Foxford AMAX series can be found in Appendix B.

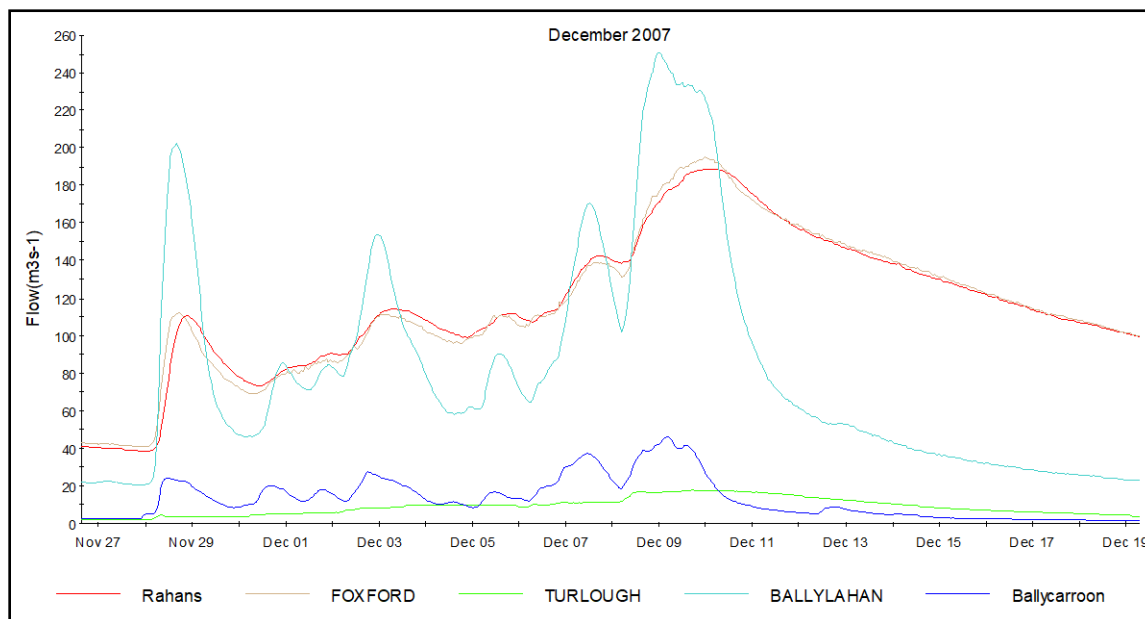
Figure 5-1: Flood peak series at gauges on the River Moy



A surprising feature of the above plot is that peak flows at Foxford are nearly always lower than those at Ballylahan. Foxford is 7km downstream of Ballylahan and the catchment at Foxford is nearly twice the size of that at Ballylahan because it includes the area draining through Loughs Conn and Cullin. The drop in peak flows is most likely due to an unusual feature of the outlet channel from Lough Cullin which is that its flow can reverse direction when the River Moy is high³. The result is that flood water from the River Moy is stored in Lough Cullin (and potentially Lough Conn too). The storage is off-line in that the river does not flow through the lough. This can be seen from OSi mapping and is confirmed by inspection of the timing of flood peaks (Figure 5-2): there is typically only a few hours delay between the peak flows at Ballylahan and Foxford, whereas a long delay would be expected if the storage was on-line.

This feature has been accounted for implicitly in the design hydrology by basing the design flood hydrograph for Foxford on analysis of flood peak data and flood hydrographs at Foxford.

³ Personal communication from Miriam Mulligan, OPW and confirmed by the Moy MPW hydraulic model.

Figure 5-2: Example flood hydrographs for gauges on the River Moy and tributaries

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 3.4. 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 available 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 "Flooding created a lake with 1 mile diameter around Foxford" or "River Moy burst its banks, highest flood in 4 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 record that has been analysed for the study area is for the Rahans gauge (34001) at Ballina, starting in 1968. The highest flood recorded by the gauge was in December 2015 at 8.45 mOD (Malin). The next highest in October 1989. The peak level was 8.11 mOD and the level stayed over 7.8m (higher than most other floods) for nearly five days. The historical review has found a mention of flooding at Ballina in 1989 although little details are available. The flood of 1968 in Ballina appears to have been just as serious, if not more so, as it is described as affecting several houses and the fire station. This flooding presumably occurred before September 1968 and thus is not included in the annual maximum flow series for the gauge at Rahans, which starts in the water year 1968-69. Another significant flood occurred at Ballina in 1948, and there are other references to earlier floods.

From the single site flood frequency curve fitted to annual maximum flows at Rahans (see Appendix B), the AEP of the 1989 flood was previously estimated to be less than the 1%. With the extension of the AMAX series to include the December 2015 flood the estimate of the 1989 flood reduces to between the 5% and 2% AEP. The December 2015 flood is estimated to be less than the 1% AEP. This is primarily down to the plotting position of the AMAX series on the single site growth curve plot.

If the 1968 flood was indeed similar or larger than that of 1989, then it is likely that the true AEP of the 1989 event is rather higher (i.e. the flood was less extreme) than that estimated solely from the gauged data. This finding has been helpful in making a choice between the single-site and pooled flood growth curves at Rahans. The latter curve, adopted as the preferred estimate, implies an AEP of just under 2% for the 1989 flood. The value of the historical analysis is, however, limited by the fact that the drainage scheme (1960-71) altered the hydrology of the catchment. This assumption does not change when considering the December 2015 flood.

5.3 Overview of method for flood peak estimation

At all HEPs in UoM 34, 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:

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

Where

DS area = Catchment area of downstream gauge (km²)

US area = Catchment area of upstream gauge (km²)

HEP area = Catchment area at HEP (km²)

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 supplied by OPW or EPA.

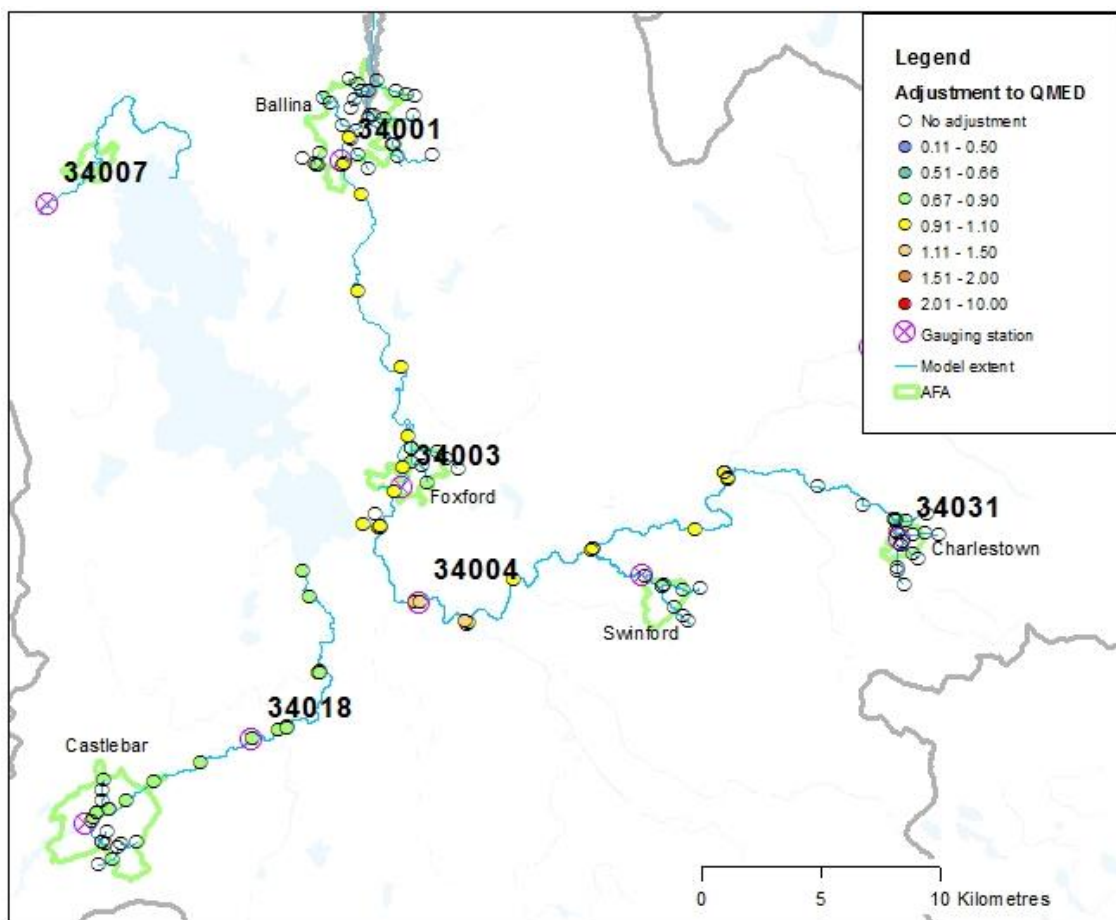
Some adjustment factors for QMED were removed after review of the flood outlines generated using the initial design flows. This decision was taken in cases where QMED was reduced by an adjustment factor derived from a donor site whose validity was questionable, due either to the quality of the flood peak data, the length of the record or the dissimilarity of the donor and subject catchments. In UoM 34, QMED adjustment factors were removed for Charlestown and Swinford due to the uncertain quality of flood peak data at the Charlestown gauge (see Table 3-1).

Figure 5-3 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 34 show only moderate QMED adjustments, generally in the range 0.91 to 1.10. On the Castlebar River at Turlough, QMED from flood peak data is more substantially lower than that predicted from catchment descriptors, resulting in an adjustment factor of 0.68. The reasons for this are not known but may be associated with the prolonged hydrographs seen on this watercourse, possibly due to backwater effects or groundwater interactions.

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.

Figure 5-3: Adjustment factors for QMED for gauges and hydrological estimation points



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 34 single-site growth curves were deemed the most representative of the gauged catchment at Foxford (34003) and Ballylahan (34004). At Foxford there is less than five times the number of years record for the 1% AEP event. The approach to select a single site above a pooling group which provides a sufficient length of record is appropriate given the nature of the upstream catchment and influence of Lough Cullin on flood flows.

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, i.e. 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 Western 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)⁴ 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.

Catchment descriptors AREA and SAAR are available for gauges in Northern Ireland, the latter being calculated over the same period 1961-90 as used for the FSU descriptor SAAR. One assumption has been made to enable the inclusion of Northern Ireland 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.

⁴ 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.

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. Although there is some evidence from research on UK data⁵ 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 content 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 catchments from Northern Ireland in most groups, the exceptions being groups created for the smallest catchments such as Charlestown (21km²), where 12 of the 16 gauges in the group are in Northern Ireland. Most groups contain more gauges from the Western RBD than from other RBDs, thus focusing the analysis on catchments that are local as well as hydrologically similar.

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 2 helps to 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 34 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.

⁵ 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)⁶ and Environment Agency (2012)⁷. 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)⁸. 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. 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 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.

⁶ Gaume, E. (2006) On the asymptotic behaviour of flood peak distributions. *Hydrol. Earth Syst. Sci.*, 10, 233-243.

⁷ Environment Agency (2012) Flood estimation guidelines. Operational instruction 197_08, issued June 2012.

⁸ 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

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.

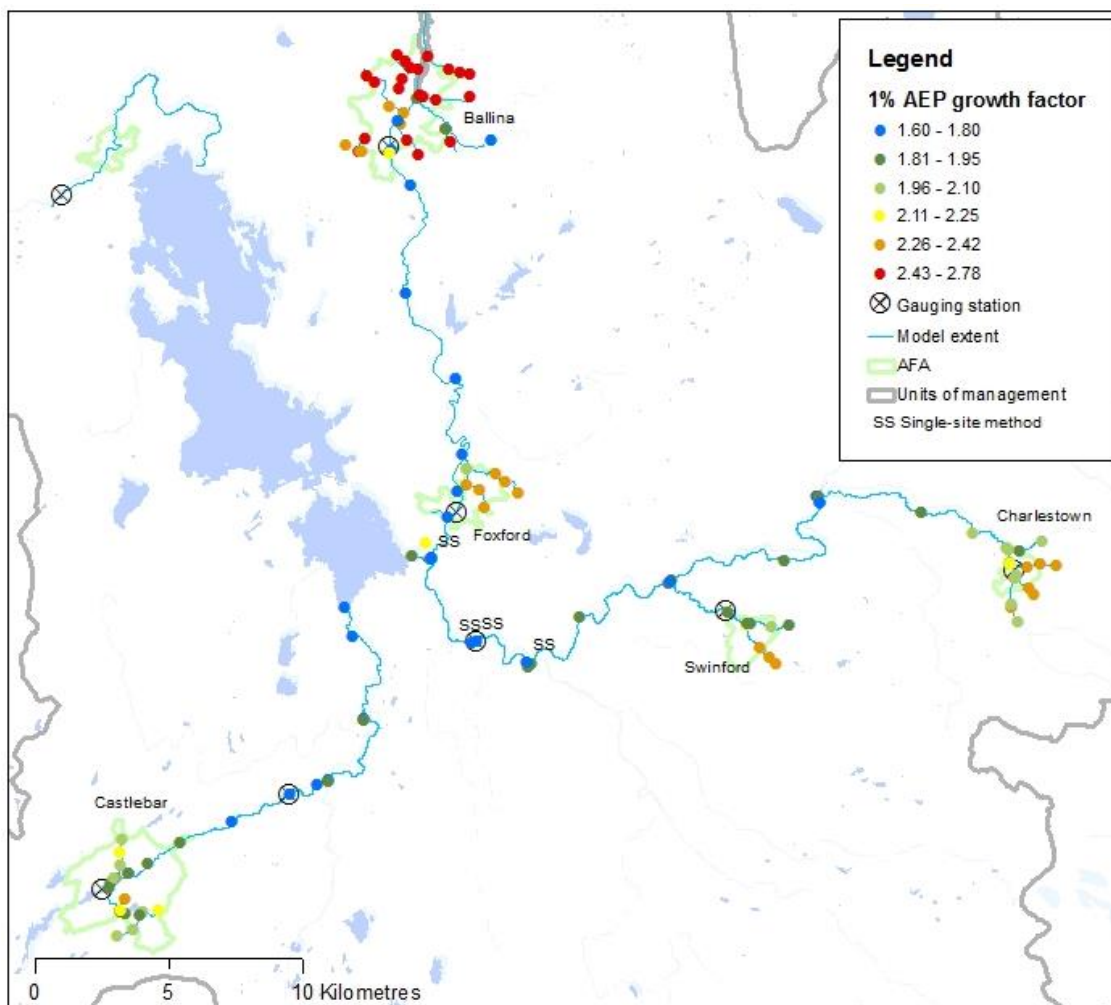
For sites on watercourses where there is a gauging station nearby at which the single-site curve is preferred, it is not appropriate to use a pooled growth curve as this may result in a sudden jump in the growth factor, leading to spatial inconsistency in the design flows. For this reason, single-site 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 5-4 shows the resulting growth factors for an AEP of 1%, i.e. the ratio of the 1% AEP flood to QMED. The major rivers (Moy and Castlebar) show low growth factors, in the range 1.6 to 1.9, due to attenuation in lakes and/or floodplains. Steeper growth curves are found on the smaller tributary catchments, with 1% AEP growth factors up to 2.4 in most places and up to 2.8 on small urban watercourses in Ballina.

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 5-4: 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 growth 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 guidelines⁹

⁹ Environment Agency (2012) Flood Estimation Guidelines.

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 AEPs 0.2% and 0.1%.

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 34 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 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 of 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 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 use in 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 34 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 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, $T_p(0)$, and the duration of the design storm. The recommended storm duration D depends on $T_p(0)$ 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 by 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 in the WCFRAM study.

The main influence on the duration of the design hydrograph is thus the value of $T_p(0)$. 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 $T_p(0)$:

- 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:

$$\text{URBAN} = 1.567 \text{ URBEXT}$$

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 T_p from hydrometric data is not possible across all of the study area due to the sparse density of Met Éireann's recording raingauges. T_p 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, and these were included in the estimation of T_p for catchments where beneficial; no such catchments were identified in UoM34.

Some of the river gauges selected for estimation of T_p are not on watercourses to be modelled but were considered as potential donor sites for adjusting $T_p(0)$ 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:

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

The resulting $T_p(0)$ values for UoM 34 are shown in Table 6-1 below. The results are very variable. At only five stations is the catchment descriptor estimate of $T_p(0)$ 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 $T_p(0)$ 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
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	
<p>* 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.</p>					

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 34, at three gauges in the upper parts of the catchment (34007, 34029 and 34031) and also the Moy at Ballylahan (34004), there is a close match between the two hydrograph shapes. On catchments with a major lake influence (the Moy at Rahans and Foxford and the Castlebar at

Turlough) 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 $T_p(0)$ (“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.

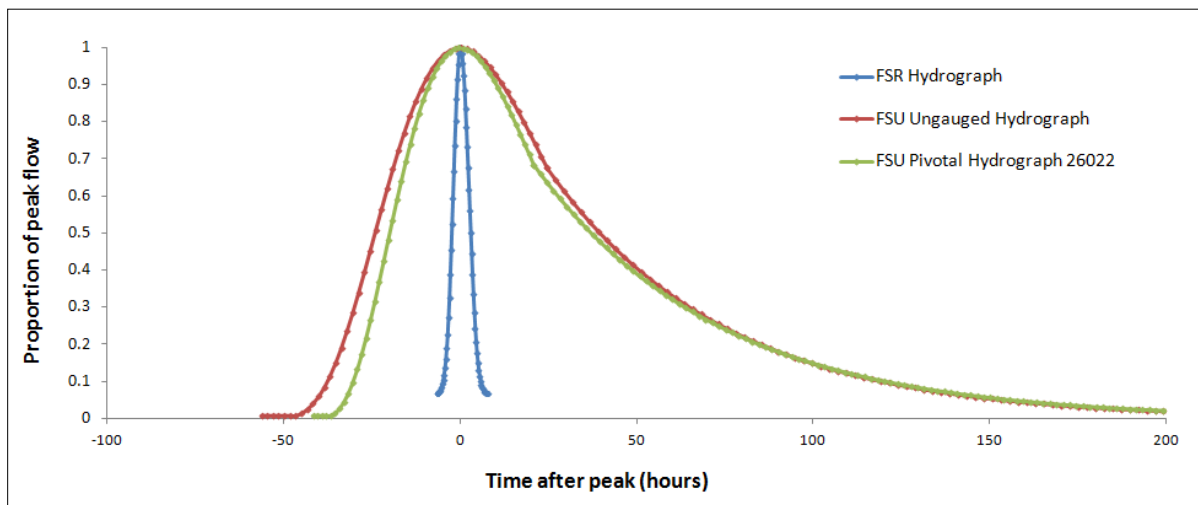
Out of the five gauged catchments, the FSU ungauged method appears to give the best fit to observed hydrographs at two gauges, and the FSU pivotal method (implemented using the OPW spreadsheet) at another two gauges. At all four of these gauges, the FSR method gives a fit that is judged to be acceptable. 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 2.5km². 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 $T_p(0)$ 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 $T_p(0)$ 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 $T_p(0)$ 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 $T_p(0)$ 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 is currently being carried out using the MPW hydraulic model. This will be reported in the hydraulic modelling report.

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 21 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 (Section 6.4.2).
- 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 data from recorded raingauges) 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.

Within UoM 34, where results from lag analysis are available (Section 6.3), the time to peak has been adjusted for model inflows close to the gauging stations at Swinford (1.08) and Charlestown (1.19).

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 the following AFAs in UoM 34:

- Castlebar – the method for producing hydrograph shapes at Castlebar will be finalised once the hydraulic model of the Castlebar River is complete. Observed hydrographs at Turlough gauging station are remarkably prolonged, but the AFA at Castlebar is well

upstream of Turlough and so hydrograph shapes at Castlebar may not be well represented by observed shapes at Turlough. IBIDEM shows that the FSR hydrograph combined with the FSU peak flow is impossibly short-duration at Turlough (the implied SPR parameter is negative), but the FSR shape may be suitable as an inflow upstream of Castlebar, if routed through Lough Lannagh.

- Foxford – the FSR hydrograph is much narrower than the observed due to attenuation in Loughs Conn and Cullin. The HPW model reach on the River Moy at Foxford starts just downstream of the outflow from Lough Cullin (River Deel). It is appropriate to use the median observed hydrograph derived at Foxford as an inflow to this model. There is a small tributary in Foxford which joins the Moy downstream of the urban area so the relative timing of hydrographs will not be critical for flood risk (and the tributary is tiny compared with the Moy – the flood on the Moy will last for weeks so we can expect peaks from the tributary to occur sometime during that period).
- Ballina – the issue here is the same as at Foxford; the hydrograph shape on the River Moy will be represented using the median observed hydrograph at Rahans gauging station.

6.6 Checks against volume frequency analysis

A statistical analysis of annual maximum flood volumes has been carried out using flow data from Rahans gauging station (34001). The results are shown in Appendix B. They indicate the expected volume of flood water over a given duration for a given AEP. This analysis 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 Rahans, the preferred hydrograph shape is derived by averaging observed hydrographs (see Appendix B, B16). When scaled to match the estimated 1% AEP peak flow of 304m³/s, the design hydrograph gives a volume of 410Mm³ (million cubic metres) accumulated over the 4-week period containing the maximum volume during the design event. The corresponding 4-week volume from the volume frequency analysis is a little over 500Mm³.

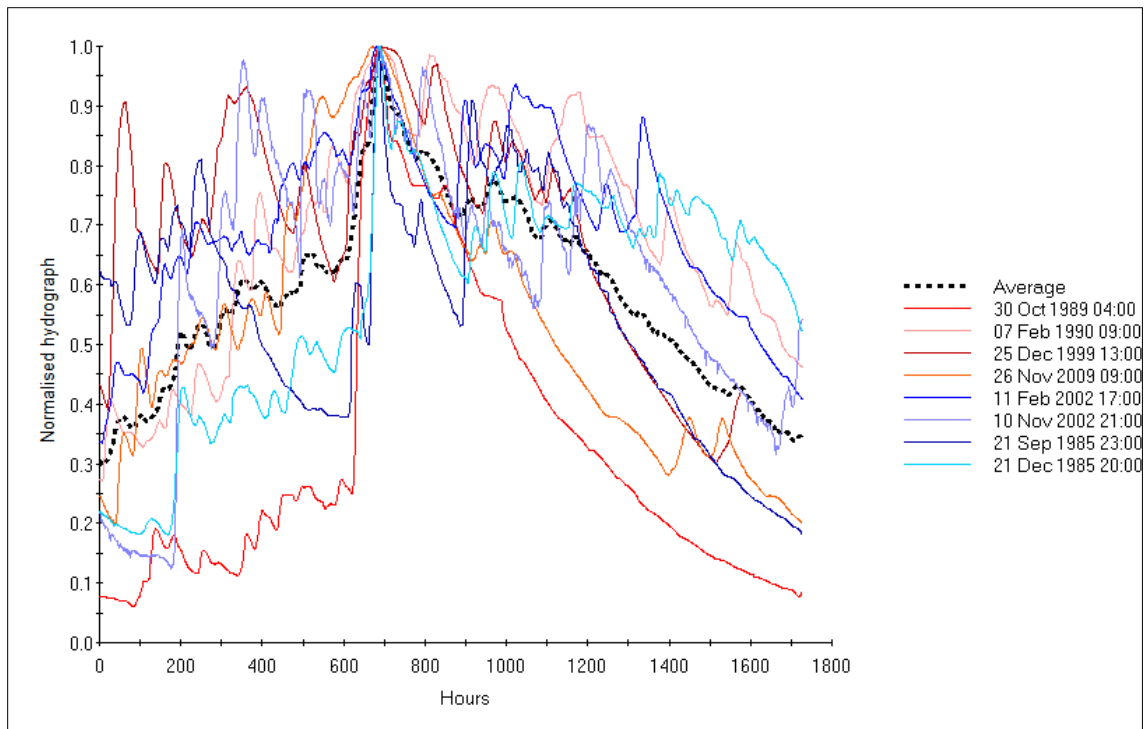
For the 10% AEP flow of 226 m³/s, the design hydrograph has a 4-week volume of 310Mm³. The corresponding 4-week volume from the volume frequency analysis is 400Mm³.

For both AEPs the design hydrograph has a volume around 20% 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 Ballina – this may be considerable for an AEP of 1% but lower for the 10% probability.
- Uncertainty in the statistical estimates of flood peak frequency.
- Systematic variation in flood volume with flood magnitude. This does not appear to be the case at Ballina. Figure 6-2 plots four of the highest floods (in shades of red) and four of the lowest (in shades of blue), centred on their peaks and normalised. There is no obvious consistent difference in shapes or durations between large and small floods.
- 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.

In the light of the above uncertainties, it is concluded that the comparison of the two approaches gives results that are within expected tolerances, and thus the volume frequency analysis helps to confirm that the method implemented at Rahans results in design hydrographs with the expected order of magnitude of volume.

Figure 6-2: Normalised flow hydrographs for some of the largest and smallest floods at Rahans



7 Summary of flood estimation process

7.1 Summary of steps leading to design flood hydrographs

The previous chapters 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 T_p 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	Watercourse	QMED method	Growth curve method	Distribution	Hydrograph shape
Ballina	Moy	DT – Pivotal using 34001 at some HEPs and 34001 and 34003 at others	P	GL	HWA - 34001
	Ardnaree, Glenree, Ballina, Bunree, Quignamanger, Quignalecka, Knockanelo, Knockleitaugh, Tullyegan	CD	P	GL	RR
Castlebar	Castlebar	DT – Pivotal 34018	P	GL	FSR subject to re-assessment during modelling
	Knockthomas, Springfield, Saleen, Saleen lake stream, Knockrawer, Milebush	CD	P	GL	RR
Charlestown	Mullaghanoe	CD (altered from DT – Pivotal 34031)	P	GL	RR-LAG
	Sargirra	CD	P	GL	RR
	Black River	CD (altered from DT – Pivotal 34031)	P	GL	RR
Foxford	Moy	DT – Pivotal 34003 (with AMAX series extended by Rahans-Foxford relationship)	S	GL	HWA - 34003
	Foxford, Rinnananny	CD	P	GL	RR
Swinford	Swinford, Newpark	CD (altered from DT – Pivotal 34031)	P	GL	RR-LAG

Meaning of codes:

QMED methods - Data Transfer (DT)¹⁰ / Catchment Descriptors (CD)

Growth curve method - Pooled (P) / Single Site (SS)¹¹

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

Hydrograph shape – FSR rainfall-runoff (RR) / FSR rainfall-runoff with $T_p(0)$ adjusted from lag analysis (RR-LAG) / FSR rainfall-runoff with $T_p(0)$ adjusted to match HWA results (RR-ADJ) / hydrograph width analysis from observed events (HWA)¹²

¹⁰ DT – If data transfer method adopted, pivotal station chosen is detailed

¹¹ SS – If single site method adopted, station number for which the growth factors have been derived is detailed

¹² 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 Western 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). This is the case for many model reaches covering HPWs flowing through AFAs, including most if not all AFAs in UoM 34. 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 corrected 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 FSU guidance, the first step will be to model a design run of the entire River Moy MPW 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. For this reason, inflows to models will be adjusted in order to match design flows along reaches that contain gauging stations.

When there are confluences within model reaches where both watercourses contribute a significant proportion of the downstream flow (as occurs along the length of the Moy), design flows will be set initially using the exceedance 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.

The relative timings of inflows will be specified using the FSR rainfall-runoff method since it has been found that the latter 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 Castlebar River due to backwater from Lough Cullin. On this reach the preferred approach will be to use the hydraulic model to work out the flow in the river given suitable input hydrographs and a downstream boundary at the lough. This may also be a consideration on parts of the River Moy due to the very low channel gradient and influence of off-line storage in Lough Cullin.

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 the rating review gauges, it has been possible to check the quality of the flow measurement and (for most gauges) extend the rating up to high flows. However, rating reviews were not carried out at all gauges:

- At Rahans gauge, it is assumed that the existing rating can be relied on up to the highest observed flows. This is because it was not possible to safely survey the weir and the weir structure has changed in recent years and thus there was not enough information to extend the rating using the hydraulic model.
- At Ballylahan and Turlough, it is assumed that the existing ratings can be relied on up to the highest observed flows. These gauges were not identified for rating reviews. The updates to the hydrological assessment for Foxford following the analysis of the December 2015 event remains uncertain given the inactive gauge status since 2013 and the need to infer data from the downstream gauge. The Foxford gauge flow rating equation adopted is the latest EPA rating equation. This has been validated by the hydraulic model analysis.
- At Charlestown, it is assumed that the existing rating can be relied on up to QMED.

The potential negative effect of the above assumptions has been reduced by estimating all growth curves in UoM 34 via pooled analysis, which helps to dissipate the effect of any errors in individual flood peak series.

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 on 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 Western 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 34 is 41 years, at Rahans. 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 publication¹³ 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 Western 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:

¹³ Framework for Assessing Uncertainty in Fluvial Flood Risk Mapping, Flood Risk Management Research Consortium Research Report SWP1.7, 2011.

- 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 34 is 1.06, increasing to 1.10 at Charlestown 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, 2006¹⁴). 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 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 will be limited to the consideration of factorial errors in QMED.

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 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) most gauging stations
- 0.83Q to 1.20Q for HEPs at (or very close to) Charlestown gauging station
- 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.

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).

¹⁴ 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.

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 hydraulic model of Ballina, which is the only modelled watercourse in UoM 34 to discharge into the sea.

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

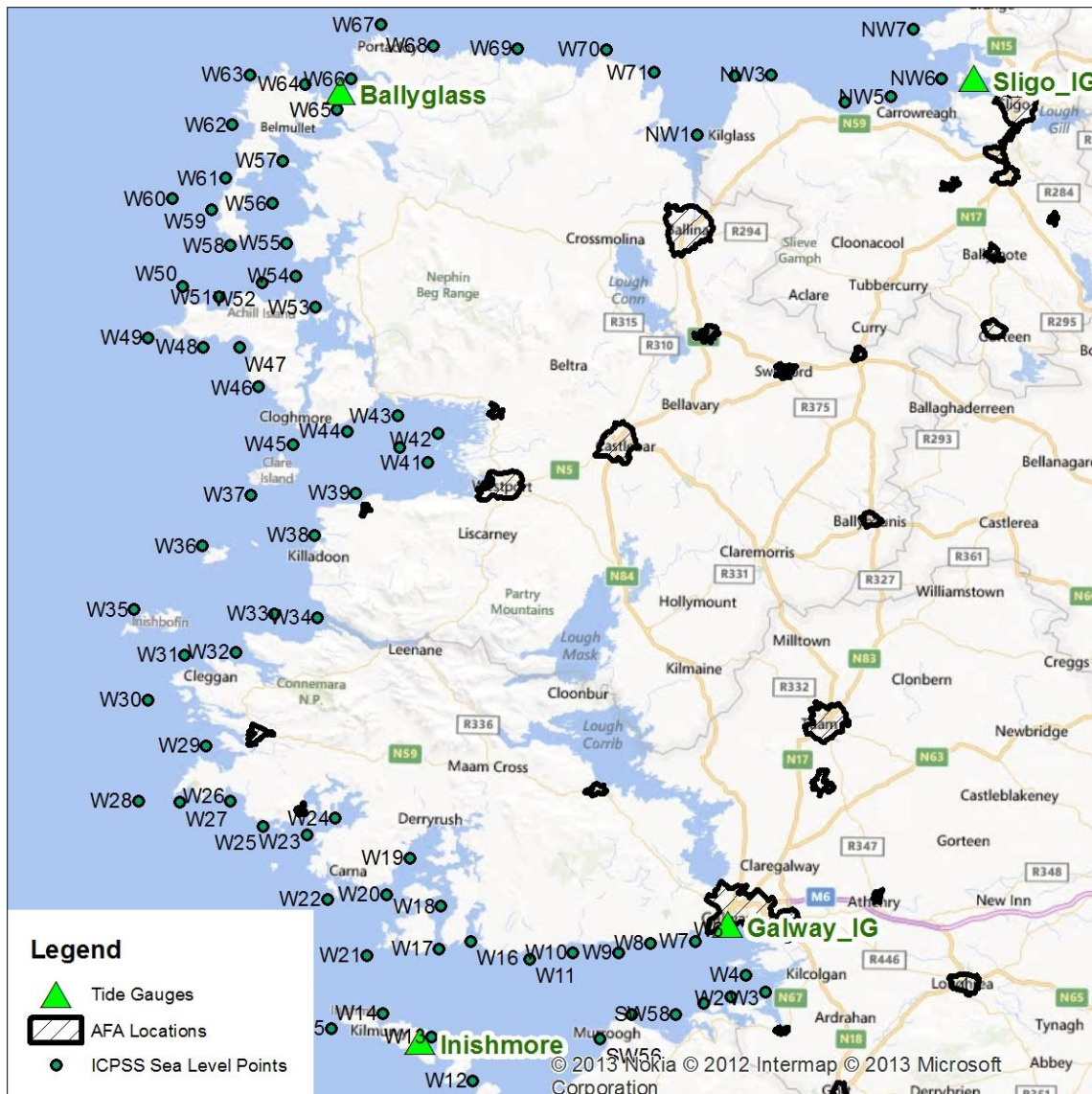
The ESLs used in the derivation of the design tidal-graphs were taken from the Irish Coastal Protection Strategy Study Phase 3 - West Coast¹⁵ report; shown in Table 10-2 and Figure 10-1. These were based on a global tidal model developed by Kort and Matrikelstryeslen in Denmark.

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

Location	Return Period (years)							
	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

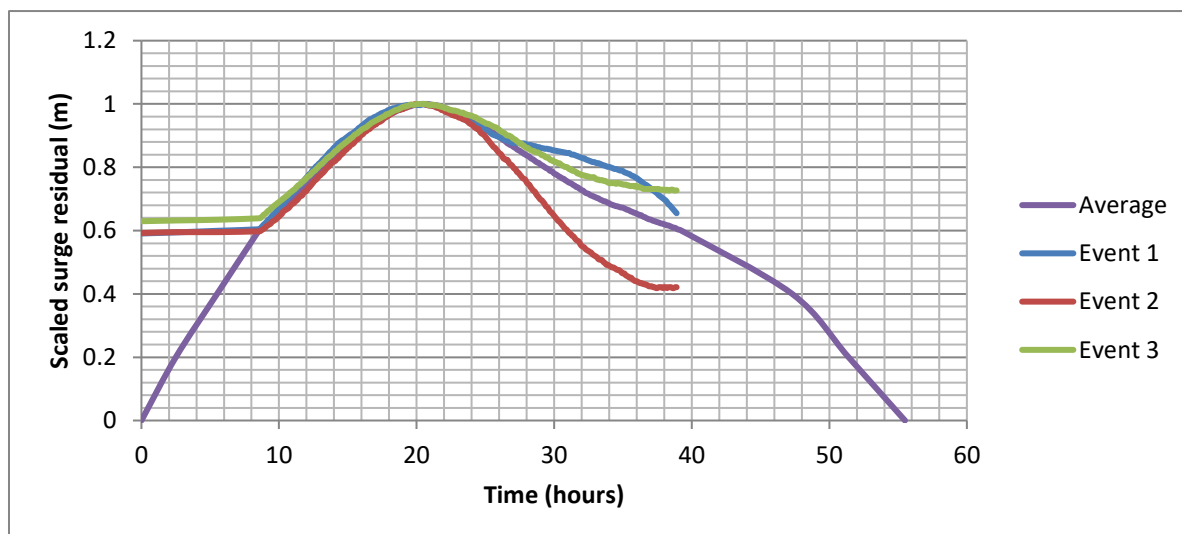
¹⁵ OPW, 2011, Irish Coastal Protection Strategy Study Phase 3 – West Coast

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; which 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, 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.

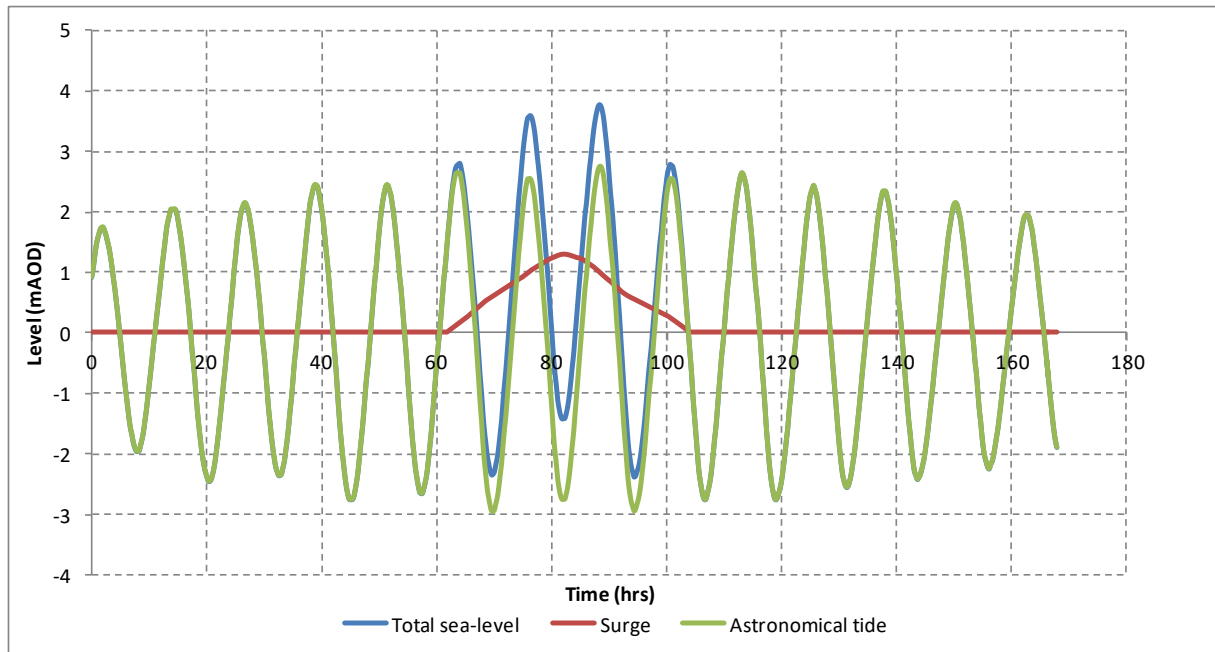
The design tidal curves were then corrected from Chart Datum, through Ordnance Datum Poolbeg, to Ordnance Datum Malin Head. In recognition of the complexity of translating through three different datums, a secondary correction factor of -0.15m or -0.1m was calculated in the Irish Coastal Protection Strategy Study, and was applied to the design tide 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 a 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 guidance¹⁶, 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.

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)¹⁷. 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 34 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.
- The housing land targets span only the period to approximately 2020 (depending on the dates of the relevant Development Plan).

¹⁶ OPW Assessment of Potential Future Scenarios, Flood Risk Management Draft Guidance, 2009

¹⁷ National Spatial Strategy for Ireland 2002-2020. The National Stationary Office

- 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
Ballina	340534	Mayo	Hub
Castlebar	340538	Mayo	Hub
Foxford	340542	Mayo	Urban Centre
Swinford	340543	Mayo	Urban Centre
Charlestown	340539	Mayo	No classification

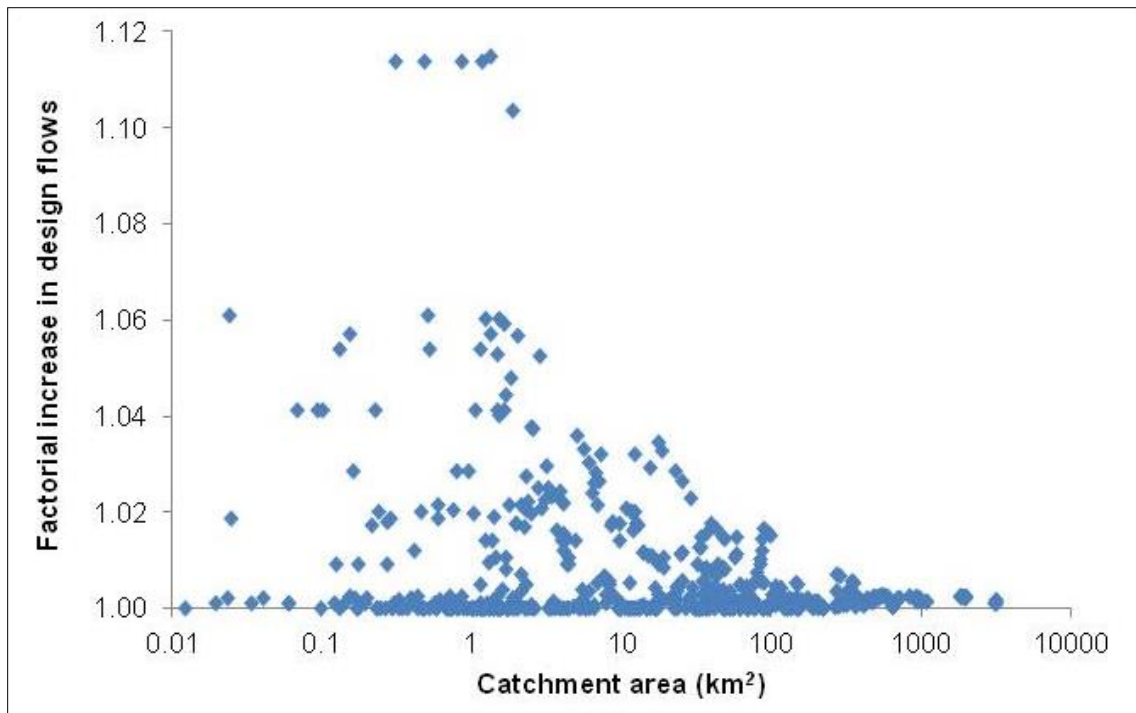
Future design flows have been tested using a future URBEXT value which is based on a percentage increase of the current URBEXT value, and then applying the urban adjustment formula developed in Flood Studies Update WP 2.3. The calculation involved first removing the effect of current urbanisation, converting 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 complex¹⁸. 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 estimation¹⁹. 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.

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

¹⁸ UNFAO Center for International Forestry Research (2005). Forests and Floods. UNFAO.

¹⁹ Institute of Hydrology (1991). Plynlion research: The first two decades. Report No. 109, Institute of Hydrology.

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 UoM34 (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 the future 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 34.

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 are 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. The Moy catchment is fascinating for hydrologists. Its diverse topography and geology and the presence of Loughs Conn and Cullin combine to produce floods that are unlike those seen on most other rivers, apart from those in other parts of the world where catchments are several orders of magnitude larger. Floods on the lower Moy tend to last for entire seasons rather than the hours or days that are more typical of many Irish rivers.
2. At Ballina and Foxford it has been possible to derive design flows, at least for the River Moy, with reasonable confidence thanks to the presence of gauging stations for which records of peak flow and flood hydrographs are available.
3. From the subsequent analysis of the December 2015 the following conclusions were drawn:
 - a. At Ballina, the December 2015 peak flood flow does not alter the extreme flow frequency produced from the analysis of the full historic record.
 - b. At Foxford, the update to the latest EPA rating curve (March 2010) and extension of the gauge record using the relationship between the Foxford and Rahans gauges results in a 2% increase in QMED and a slight reduction in the flood growth curve. The hydrological analysis remains uncertain.
4. At Castlebar, there is reasonable confidence in design flows for high AEPs given the presence of a downstream gauging station. However, karst features in the catchment introduce some uncertainty, in particular for more extreme floods.
5. There are some locations where the flows are less certain than elsewhere. In general, these are on small ungauged catchments, where QMED has been estimated from catchment descriptors and no suitable donor catchments have been identified for adjustment of QMED. Flows are also uncertain at Swinford and Charlestown, where there is no high quality flow data available.
6. 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.
7. 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.

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, with the exception of Castlebar, for which the hydrograph shape may require adjustment during the hydraulic modelling.
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. At Swinford it is recommended that the scope for developing a rating for the river level gauge is explored.
4. At Charlestown it is recommended that a review of the rating is carried out, and check gaugings are obtained for higher flows.
5. On the Castlebar River it is recommended that the unusually prolonged hydrographs are investigated within the Western CFRAM study, once the hydraulic model of the river is complete.

6. The Foxford gauge previously operated by the EPA became inactive in 2013. This gauge has the potential to operate as part of a flood warning system for the Blackrock Nursing Home, which was evacuated in the December 2015 floods. Re-activating the gauge without delay would enable telemetered alarms with appropriate trigger levels to be enacted at a low cost.

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

7. 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.
8. 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

Rahans

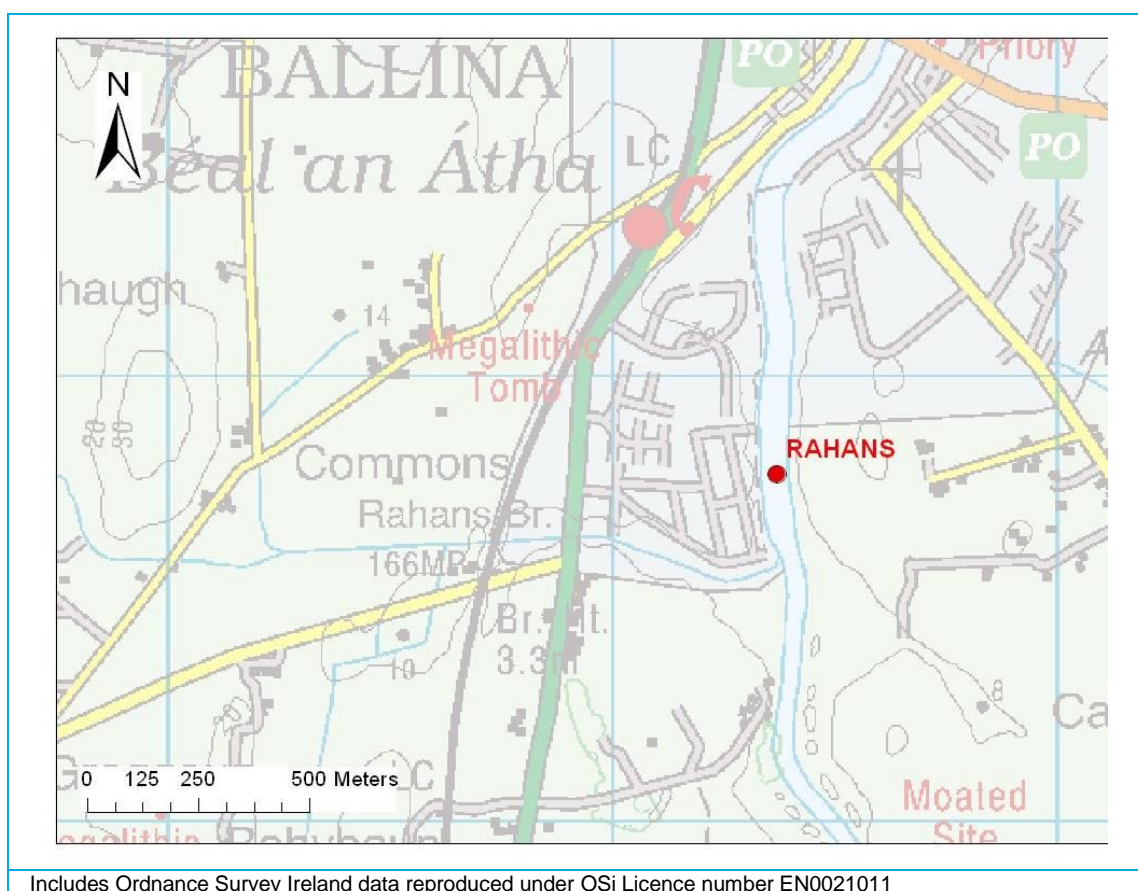
1.1 Station description - from Inception Report

1.1.1 Gauge summary

Station name	Rahans	Site type	Velocity-area
Station number	34001	Watercourse	River Moy
Grid reference	124367 317782	Operator	OPW

1.1.2 Location

The gauge is located on the left bank of the watercourse at the provided grid reference.



1.1.3 Gauge Datum


Gauge datum (mAOD)	5.78m (valid from 29/10/1997)
Means of confirmation (e.g. survey)	Supplied by OPW
Other comments (e.g. gauge boards)	No gauge board was visible.

1.1.4 Description/ other comments

The gauge is located on an open channel section of watercourse; however, there is a large weir downstream that will provide the dominant hydraulic control at the site. It is believed that flow over this weir is partially controlled by sluice gates. The implication of this is that there may not be a stable relationship between flow in the watercourse and stage recorded at the upstream gauge. OPW have indicated that they are not aware that the operation of the sluices has any effect on the rating. There was a partial blockage of the channel, owing to a

weir collapse, between 2000 and 2008; it is possible that this will have had an impact on the rating during this period.

1.1.5 Control on stage discharge relationship

Type of section	Open channel section.
Low flow control(s)	<p>Approximately 650m downstream of the gauge there is a large salmon weir; it is likely that this structure will be the dominant hydraulic control at this site.</p> 
High flow control(s)	At much higher flows it is possible that the weir downstream may become non-modular and no longer exert a hydraulic control on upstream water levels. If this is the case then the controls on the stage discharge relationship at the gauge will become much more complex with downstream bridges possibly playing a role.
Bed slope	Given the large weir downstream it is not possible to accurately estimate the bed slope at the gauge location from OS mapping. This will have to be measured following survey of the site.
Roughness	In channel roughness along the gauged reach is low.

1.1.6 Bypass routes

Significant bypassing of the gauge location is not considered possible even during extreme floods.

1.1.7 Additional photographs

Looking upstream towards the gauge hut	
	
Looking upstream from the gauge	Looking downstream towards the footbridge and the weir beyond.
	

1.2 Rating details

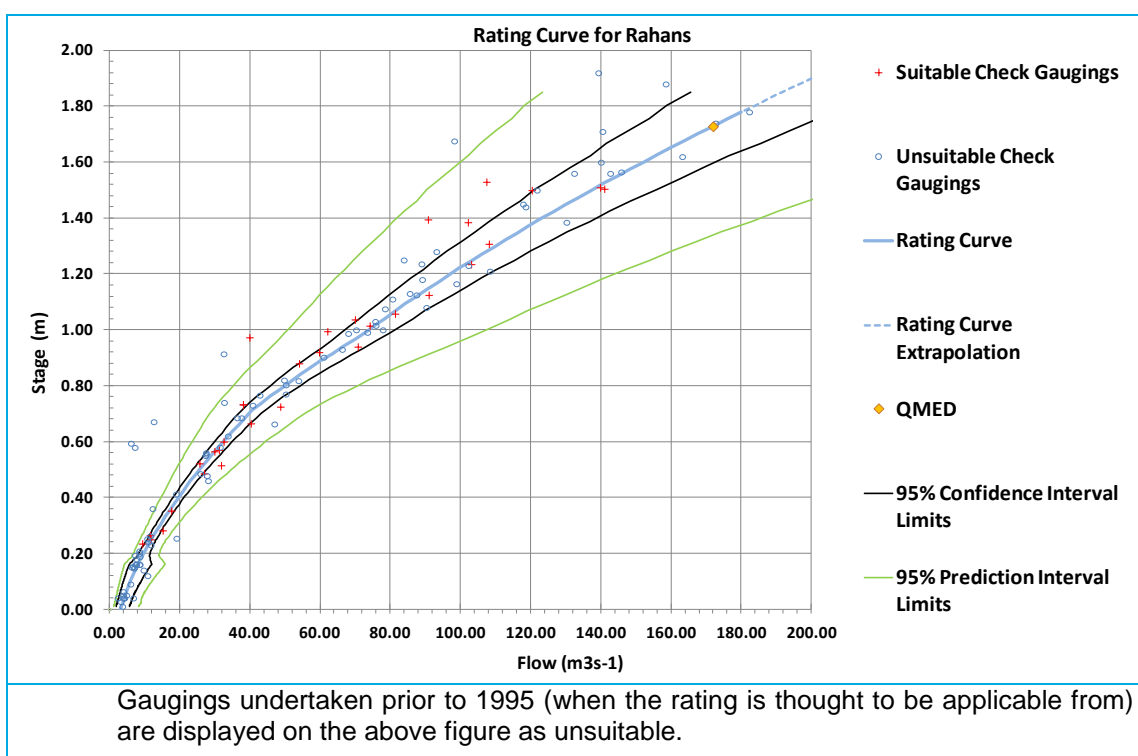
1.2.1 Check gaugings summary

No. of gaugings	126 (34 from 1995 onwards)	Date range	1995 - 2011
Maximum gauged stage (m)	1.66 (Since 1995)		
Approximate stage corresponding to QMED (m)	1.79	Extrapolation of rating to QMED (m)	0.13
Maximum observed stage (m)	2.08 (26/11/2009)	Extrapolation to highest flow (m)	0.42
Other comments	None		

1.2.2 Details of existing rating

The details of eight ratings for this gauge have been supplied by OPW. One of them (ID 8) has been supplied twice, once with a valid from date of 2004 and the other from 1995. In addition to these two, an additional rating (ID12) with a valid from date of 2001 has also been supplied with a comment indicating that it is an improved high flow gauging. This rating however captures the partial collapse of an eel fishing structure between 2000 and 2008, including partial blockage in the channel. This rating is therefore not representative of the flows for the entire gauged record. Given the uncertainty regarding which rating to use, both were compared to the check gaugings. Rating ID 8 gave the best fit and is plotted below.

Limb No.	C	A	b	Min stage (m)	Max stage (m)
1	46.29	0.22	1.787	0.000	0.712
2	50.00	0.20	2.203	0.712	0.973
3	53.50	0.20	1.778	0.973	1.778



1.2.3 Evaluation of existing rating

Overall agreement with check gaugings	There is a significant amount of scatter within the check gaugings particularly for those taken above a stage of approximately 1.2m.
Range of applicability	At present the existing rating is considered applicable for stages up to 1.778m and for data that was recorded after 1995. Given the scatter within the high flow gaugings further extrapolation of this rating should only be considered with caution.
Stability of rating	As already discussed there is a significant amount of scatter within the check gaugings supplied for this site. The partial collapse of an eel fishing structure between 2000 and 2008 explains some of the higher data points observed, however build up of debris on the gates of the Salmon Weir is a known issue. During higher flows, when backing up from the Salmon Weir extends far enough upstream to affect the gauge, it is possible that different observed water levels could result at the gauge in different events for the same flows. Further investigation of this blockage, which would require data on the level of blockage at the time of each gauging, is not realistic.
Uncertainty	The 95% confidence interval at QMED is estimated from the supplied data to be approximately 23m ³ /s, this represents nearly 27% of QMED.

1.2.4 Recommendations for rating improvement

In order to further improve the existing rating we recommend developing a hydraulic model of the gauged reach. This model will enable more reliable extrapolation of the rating to high flows and simultaneously provide increased confidence in the rating at moderate flows. Given the lack of bypassing at this structure and the nature of the downstream hydraulic controls (weir and possibly bridges) we recommend developing a 1D hydraulic model. The gauged reach falls within a designated HPW and will therefore already form part of a hydraulic model. To ensure this model is suitable for improving the existing rating it should be extended sufficiently far downstream that an accurate tidal boundary can be applied as the boundary condition. It should also extend at least 200m upstream of the gauge.

1.3 Rating improvements

With the aim of improving 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 Choice of modelling method

The Rahans Gauge is located on the Moy River within a wide channel with well maintained high banks that prevent bypassing of the gauge. The raised banks extend all the way down to the Ballina Salmon Weir, 900m in total, preventing out of bank flow for the full reach. The Ballina Salmon weir is the main control at the gauge site with the intervening channel between the gauge and the weir providing a further control, most likely at low flows.

Because of the constrained nature of the channel, all features can be modelled appropriately in a 1D hydraulic model and this is the preferred approach for the rating review assessment. The model has been built using ISIS v3.6.

1.3.2 Summary of hydraulic modelling

Overview of model and location

The Rahans gauge is located within the Ballina 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 2km upstream of the gauge site to downstream of Ballina.

There are a number of small tributaries discharging into the Moy River through this reach and these will be omitted for the purpose of the rating review model.

The Ballina River is represented in 1D to bank top. The model does not extend beyond this level into the floodplain as this is considered to provide a suitable coverage for the rating review.

Representation of channel controls

The main channel control at the site is the Ballina Salmon Weir. This is a complex structure designed to monitor the number of fish passing the site. The structure was refurbished in 2011 and for the purpose of the rating review the model has been constructed to represent the structure prior to 2011, since that is the period spanned by the river level record which will be included in the hydrological analysis.

The structure splits the channel in two, with the majority of flows passing down the right hand channel.

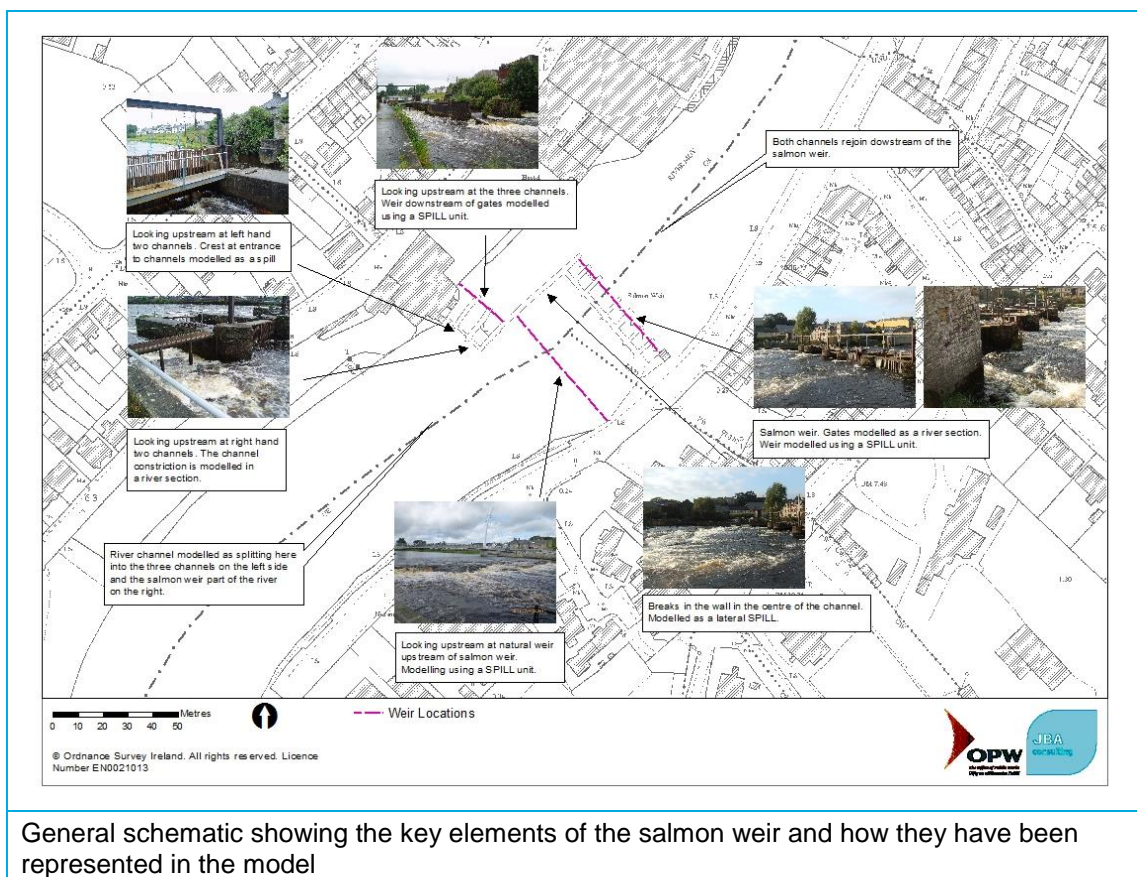
The left channel contains three gates which further split the channel for a short length. There is a screen on the central gate and drops in bed levels at the upstream and downstream of each gate.

The right channel contains a significant natural weir immediately upstream of the salmon gates which controls the upstream water levels. No survey is available of this weir because of the danger of surveying the channel during the prolonged high flows that were experienced during the survey period. Downstream of this weir there are seven gates, a number of which have screens to prevent fish passing. The fifth gate from the right bank is larger than the remaining gates and has no controls.

The pier that runs down the channel splitting the channel in two also contains two openings linking the left and right channels. These are both located between the natural weir and control gates on the right hand channel and these link into the left hand channel downstream of the three control gates.

This structure has been represented in the model as a series of parallel channels and weirs (included as SPILL units) representing the drops in bed levels at the upstream and downstream of the gates and the natural channel itself. The link between the two channels has been represented as a side spill.

There is a large bridge immediately upstream of the Salmon Weir that has a single pier. This is represented in the model using a USBPR unit but its effect on water levels is negligible compared to the Salmon Weir.



Comparison and use of gauge datum

The datum provided with the gaugings is 3.07m OD. The 0m level on the gauge board as surveyed was 3.03m OD. There is a close agreement between the datums and the datum provided with the gaugings has been used for the analysis.

Roughness values used

A channel Manning's n value of 0.030 has been applied for the channel bed of the Moy River in the vicinity of the gauge site where the bed is mud and stone. The left bank and right banks are long grass and Manning's n has been set as 0.04.

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 $15\text{m}^3/\text{s}$ and peaking at $350\text{m}^3/\text{s}$. The maximum AMAX flow at the gauge, from the existing rating, is $287\text{m}^3/\text{s}$. There is little attenuation between the top of the model and the gauge site and the peak flow at the gauge site is also $350\text{m}^3/\text{s}$.

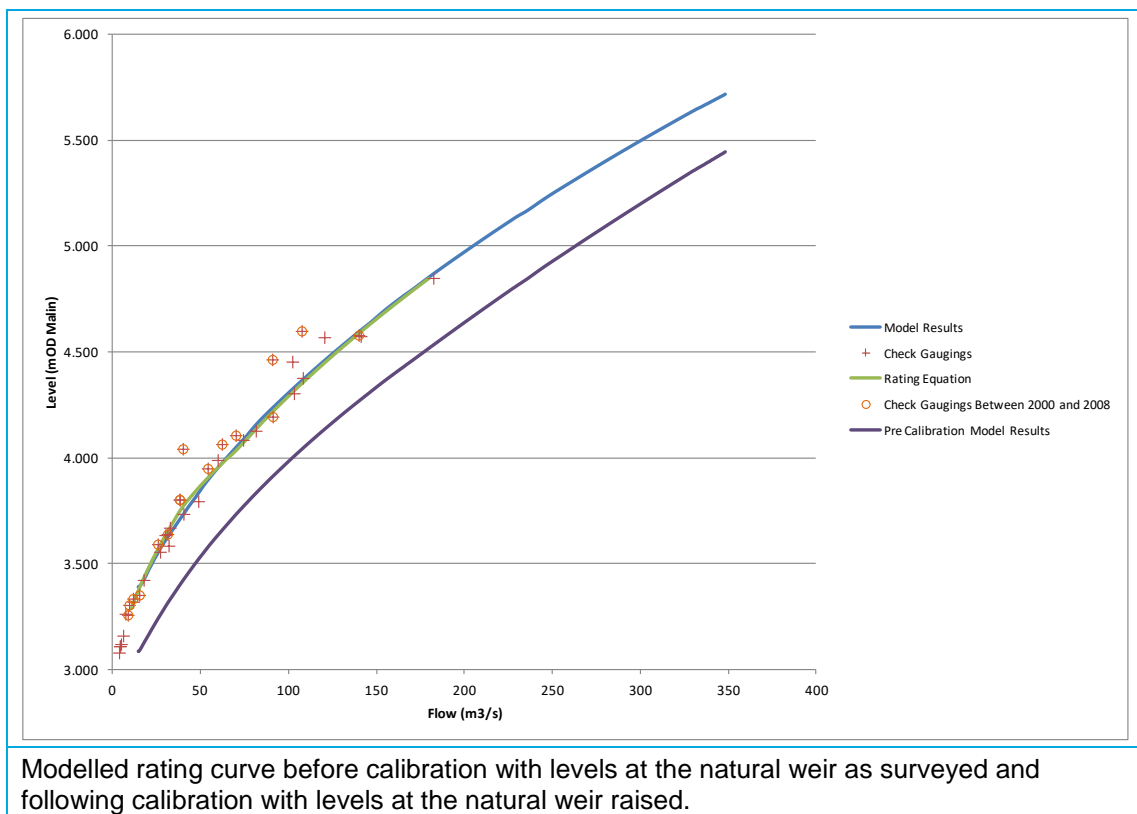
1.4 Further discussion

There were significant difficulties in achieving a suitable rating curve at the gauge site using the model. This was obviously complicated by the fact that no survey was available of the large natural weir and the survey data collected as part of the WCFRAM study was not applicable for the rating review given the refurbishment in 2011. File note 44 (issued on 23 April 2012) explains the problems in further detail.

A poor calibration has been achieved using the available survey and because of this it has been necessary to adjust the model geometry to achieve a fit to the gauging data. This has required an increase in the level of the natural weir above what would be expected based on surveyed levels at the channel sides and a higher hydraulic efficiency than would be expected

for the same structure. The need to do this suggests there may be additional local controls that have not been picked up in the survey due to high flows, but this is unconfirmed.

The result is a model that matches the check gaugings well but not necessarily for the right reasons. For this reason it is not advised to extrapolate the curve significantly beyond the gauged data.



Conclusions

The hydraulic model provides a reasonable fit to the gaugings, however it was not possible to achieve this fit using the available survey data without some significant assumptions being required. The model has therefore been calibrated to the existing rating and will provide little additional benefit to improve confidence above the range of check gaugings.

The variation between observed levels at the gauge and those predicted by the model using survey data suggests there may be an additional unknown control in the river bed between the gauge and the weir, but this has not been confirmed.

It is recommended that the existing rating ID8 is used to calculate flows for the purposes of the CFRAM study.

2 Ballycarroon

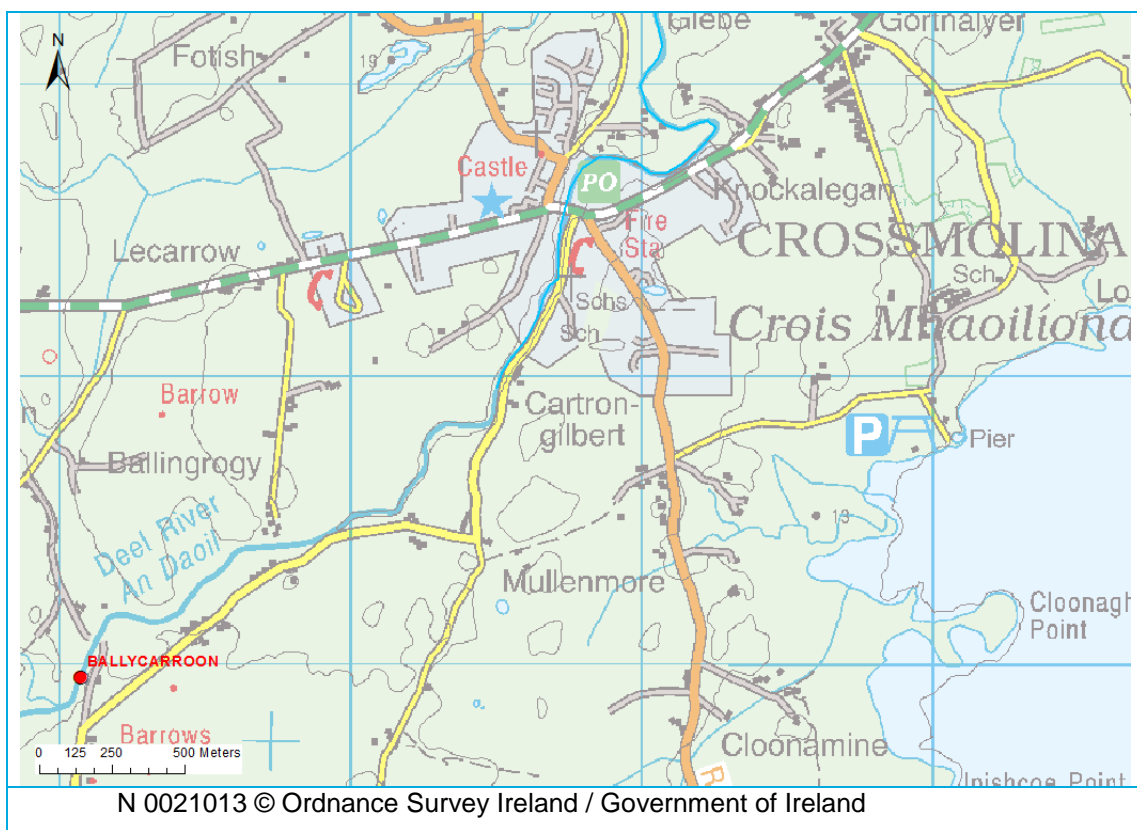
2.1 Station description

2.1.1 Gauge summary

Station name	Ballycarroon	Site type	Autographic recorder
Station number	34007	Watercourse	River Deel
Grid reference	112073 315967	Operator	OPW

2.1.2 Location

The gauge is located on the left bank of the river, approximately 50m upstream of a weir/ ford.




2.1.3 Gauge Datum

There are two gauge boards at the station location, as shown in the photograph below. The OPW reports measurements from the left (lower) of the two gauges. During the topographic survey (May 2013), the gauge to the right was surveyed, thus identifying a discrepancy between the two gauges. This is noted here so as to highlight the gauge board that is used for measurement of stage by the OPW.

Gauge datum (mOD) - left hand gauge	20.65m - OPW records
Gauge datum (mOD) - right hand gauge	20.434m - as surveyed
Other comments (e.g. gauge boards)	Gauge boards located 50m upstream of weir/ ford

2.1.4 Control on stage discharge relationship

Type of section	Open channel with gauge hut
Low flow control(s)	At low flows the dominant hydraulic control will be the channel geometry both at the gauge location and downstream. There is a weir/ ford approximately 50m downstream of the gauge, which may exert some hydraulic control at the gauge site.
High flow control(s)	<p>At higher flows the control is likely to be less well defined but will probably be dominated by the geometry and resistance of the natural channel downstream of the gauge.</p>  <p>Note: The OPW records data from the left gauge shown in the above photograph.</p>
Roughness	In-channel hydraulic roughness is moderate.
Bypass routes	The gauge will not be bypassed until the banks are overtopped. Overtopping of the bank level is considered possible during large flood events.

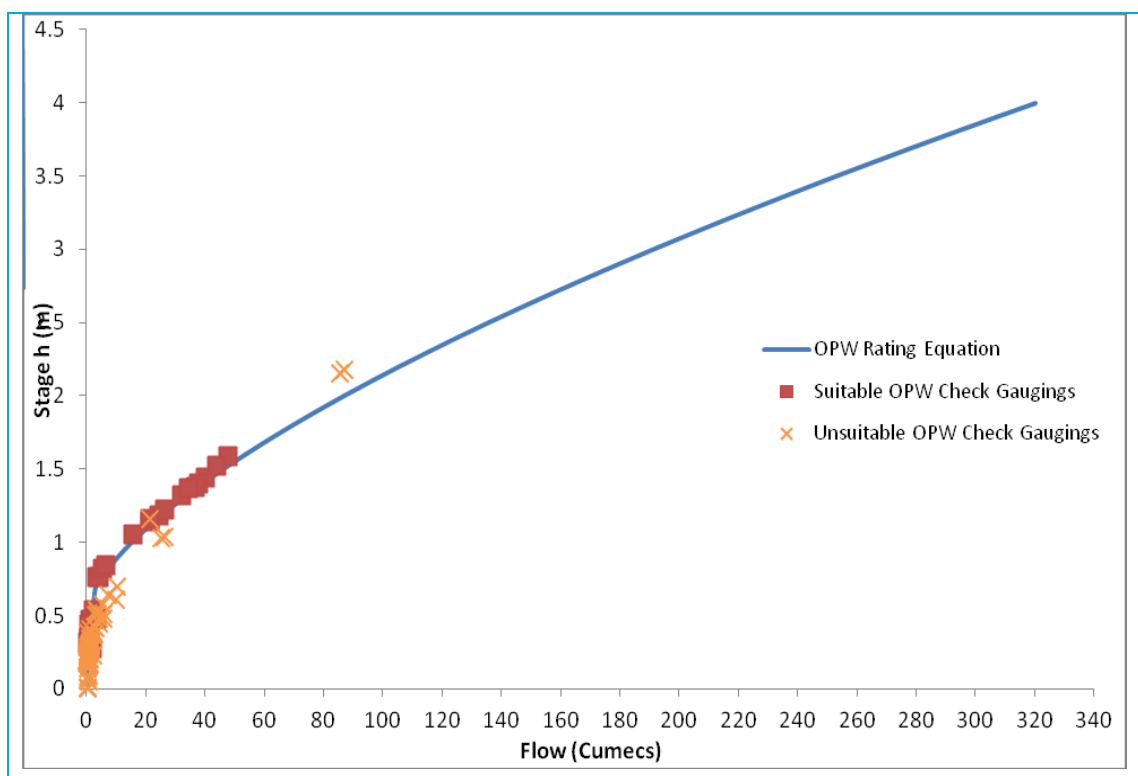
2.1.5 Additional photographs

Looking upstream from ford	Culvert inlet at weir/ ford
	
Looking at gauge hut from left bank	Gauge boards
	
View onto left bank away from channel	View across channel from left to right bank downstream of gauge
	

2.2 Rating details

2.2.1 Check gaugings summary

No. of gaugings	114 (26 suitable)	Date range	1940 - 2014
Maximum useable gauged stage (m)	1.38		
Approximate stage corresponding to QMED (m)	2.01	Extrapolation of rating to QMED (m)	n/a
Maximum observed stage (m)	1.16	Extrapolation to highest flow (m)	n/a
Other comments	<p>There is 22 suitable gaugings recorded since this rating equation was established in 2006.</p> <p>In order to improve confidence in this rating, further flow gaugings should be sought, particularly at higher flows when measurements at the gauge would be recommended. The figure below shows historic gaugings for this site, which match their respective rating curves. The plots in red are related to the rating curve shown on the plot.</p> <p>Although the maximum usable gauged flow was recorded at a stage of 1.38, a higher gauged stage has been recorded. This was at 2.18m (highest stage shown in the figure below). It, and the second highest stage (also shown in plot below), have been classed as unsuitable for use as they were measured in Crossmolina town and not at the gauge.</p>		



2.2.2 Details of existing rating

Many ratings have been developed for this site and the current one provided has an ID of rating curve number 125. The current rating was last updated in 2006. The rating is provided as having 4 limbs, with the parameters of the 3 upper limbs being the same; a rating with 2 limbs to the data has therefore been applied.

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	6.3000	-0.0300	2.0800	0.0000	0.6920
2	48.7	0.535	1.51527	0.692	1.5

2.2.3 Evaluation of existing rating

Overall agreement with check gaugings	There is currently a good correlation between existing rating and the suitable check gaugings.
Range of applicability	The existing rating is currently considered applicable to 1.5m, with extrapolation of the existing rating much beyond this resulting in greater uncertainty.
Stability of rating	The current rating at Ballycarroon gauge is considered to be relatively stable, due to the ford/ weir crest providing a hydraulic control. Scatter around the existing rating is most likely to represent errors in the gauging process.
Uncertainty	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. The latter option has been applied for this study.

2.3 Rating improvements

In order to improve confidence in the high flow 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.

2.3.1 Summary of hydraulic modelling

The Ballycarroon gauge is located on the River Deel, 50m upstream of a weir/ ford and 2km upstream of the town of Crossmolina. As such, the gauge is included in the 1D (ISIS) hydraulic model upstream of Crossmolina town. The floodplains are represented in the 1D model by extended cross sections, which enable representation of the floodplain flow. Crossmolina town is included in the 1D-2D (ISIS-TUFLOW) linked hydraulic model.

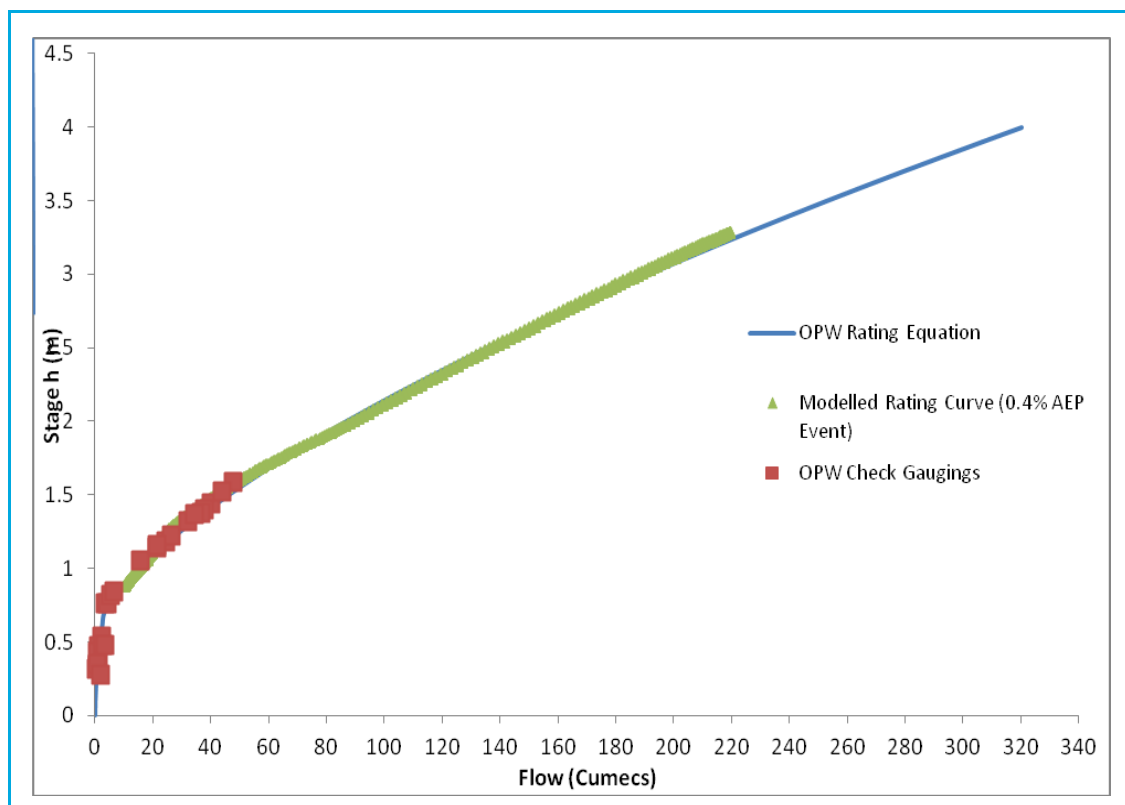
At low flows, the weir/ ford acts as the hydraulic control and this appears to be adequately represented in the survey data. Gaugings extend to approximately 25m³/s which is less than QMED; higher flows are represented by the model.

A channel Manning's n value of 0.04 has been adopted for the reach through Ballycarroon gauge in order to provide a realistic representation of the channel roughness, while providing a rating curve with appropriate curvature to fit the range of check gaugings. Channel bank and floodplain roughness is set to 0.065, given the moderate to dense vegetation cover, which includes a mixture of tree, bushes and scrub.

The hydraulic model run used to derive the rating curve was a single unsteady run with the 0.4% AEP flood event hydrograph, derived from previous OPW design flows. The peak flow used gives a level approximately 1.75m above the highest recorded level at this site.

The 0.4% AEP hydrograph applied in the modelled rating curve does not take into account very low flows, due to instabilities in the model at such flows. Therefore, the modelled rating

curve cannot compare to the stage-discharge of low flow OPW check gaugings. However, it is clear that the model replicates the hydraulic conditions at the gauge from 10 cumecs upwards when comparing to the OPW rating equation.



2.3.2 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 respective stage discharge pairs available in Section 1.4. The proposed rating form is shown in the graph below.

Limb	C	e	β	SG (min)	SG (max)
1	6.3000	-0.0300	2.0800	0.0000	0.6920
2	48.7	0.535	1.51527	0.692	1.5

The proposed rating consists of 2 limbs to describe the hydraulic relationship at Ballycarroon gauge up to a water depth of 1.5m. The proposed rating adopts the same parameters as the existing rating.

2.3.3 Comparison with existing rating

The graph above compares the existing rating with the newly proposed modelled rating. The stage discharge relationship extracted from the model is very similar to the existing rating equations. The relevant check gaugings fit well to both existing and modelled rating curve.

2.3.4 Overall agreement with check gaugings

The proposed rating fits relatively well to the available flow gaugings. It should be noted that these are all well within bank and there are no check gaugings available beyond 2.18m with which to calibrate the model or provide any validation during high flow conditions. The modelled rating curve is thought to provide a reasonable estimate in the absence of any additional gaugings.

2.3.5 Range of applicability

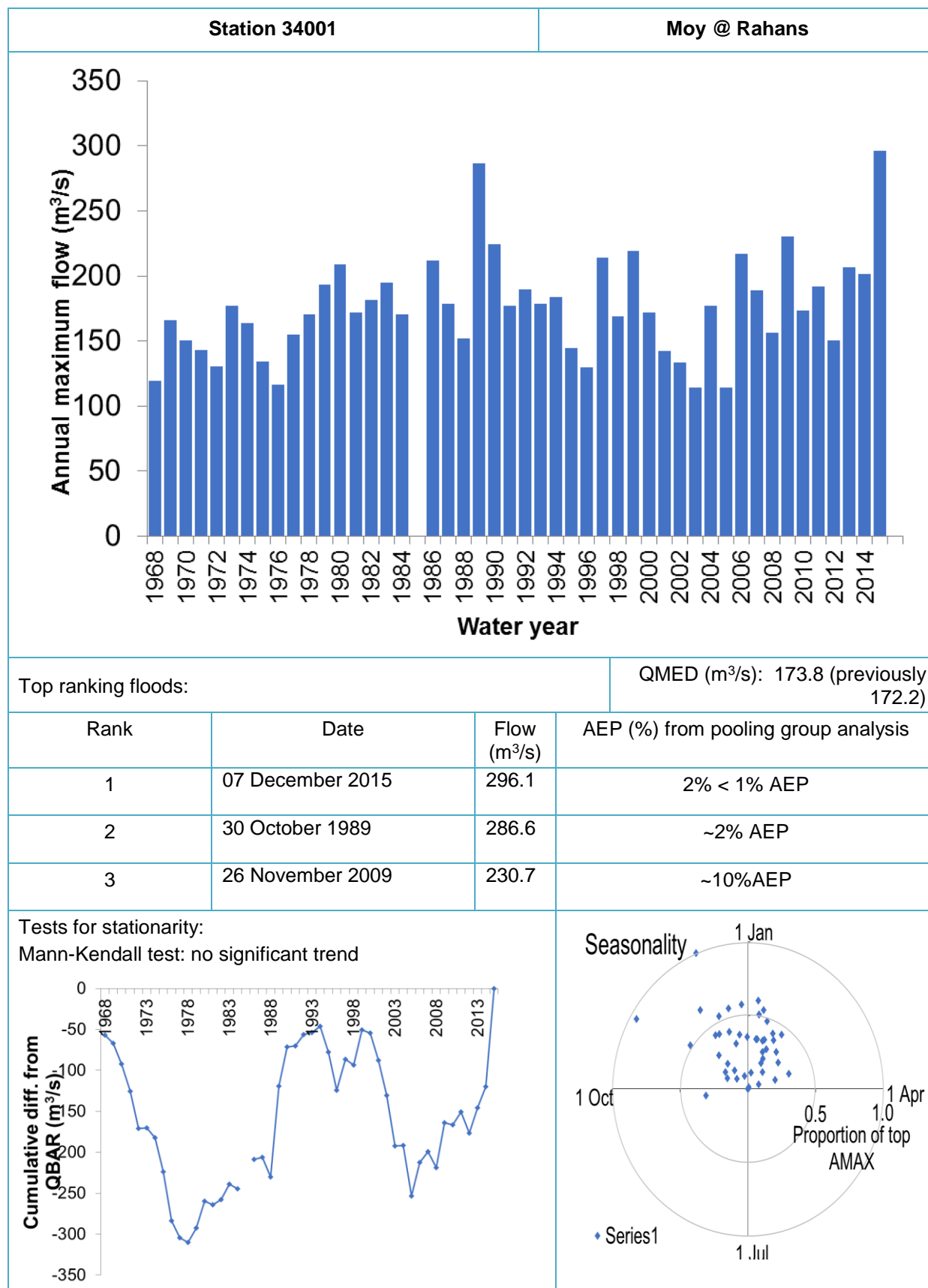
The upper limit of the rating is specified at 3.25m, which is the peak stage value attained during the hydrodynamic simulation. For stage values beyond this there is less certainty on what form and gradient the rating will assume, although it is likely that the upper limb will continue as the floodplain is already inundated by this point and the floodplain appears relatively well contained.

2.4 Proposed rating stage discharge pairs

Stage [h]	Discharge [Q]
4.5	392.6711
4.4	377.7626
4.3	363.0516
4.2	348.5406
4.1	334.2321
4	320.1291
3.9	306.2342
3.8	292.5504
3.7	279.081
3.6	265.8291
3.5	252.7981
3.4	239.9917
3.3	227.4136
3.2	215.0677
3.1	202.9583
3	191.0897
2.9	179.4668
2.8	168.0944
2.7	156.9778
2.6	146.1228
2.5	135.5353
2.4	125.222
2.3	115.1897
2.2	105.4463
2.1	95.99988
2	86.85958
1.9	78.03536
1.8	69.53824
1.7	61.38053
1.6	53.57609
1.5	46.14065
1.4	39.09231
1.3	32.45224
1.2	26.24556
1.1	20.50291
1	15.26267
0.9	10.5751
0.8	6.510146
0.7	3.175454
0.6	0.004283
0.5	0.004283
0.4	0.004283
0.3	0.004283
0.2	0.004283
0.1	0.004283
0	0.004283

B Flood peak analysis

Flood peak series summary sheet



The vast majority of flood events at this site occur between October and April. There is a fairly typical range of AMAX magnitudes with the largest flood on record having a growth factor of approximately 1.7. There appears to be no significant long term trend within this data set.

The AMAX series has been extended with the latest flow data and contains the highest recorded flow at Rahans on 07 December 2015. There has been no update to the stage-discharge rating equation. QMED has increased marginally by 1.6 m³/s.

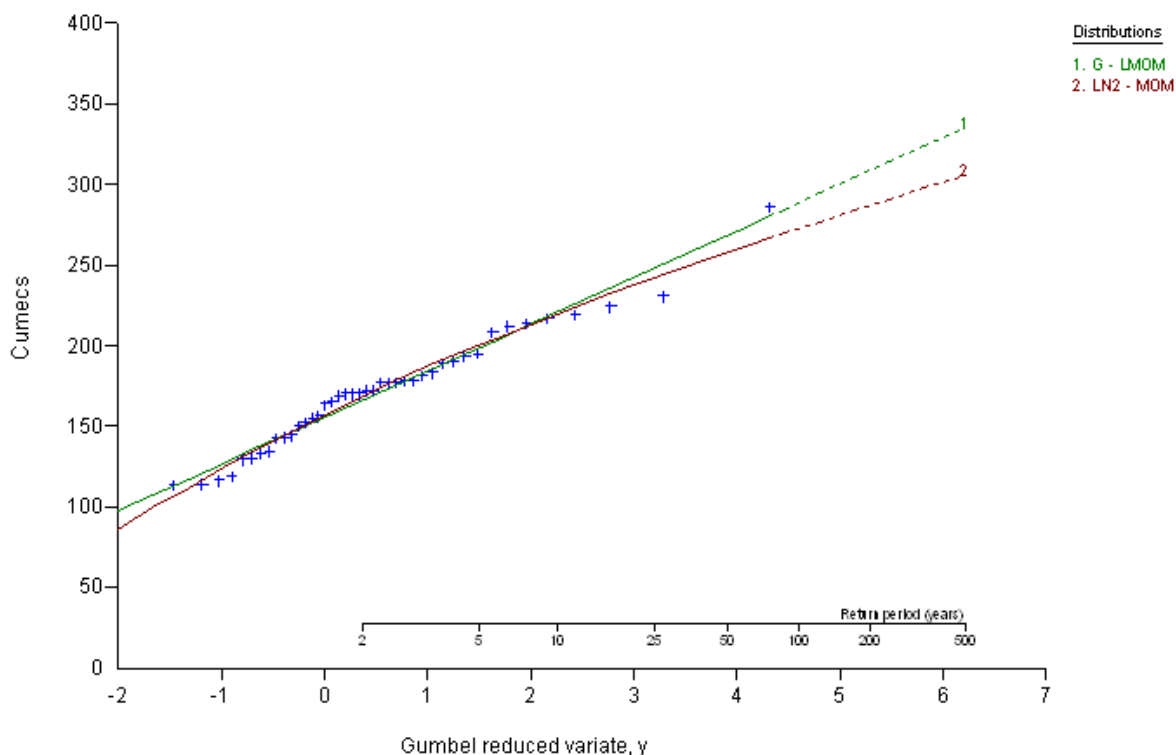
Statistical QMED estimates at Rahans (34001)

Statistical QMED at gauge (m ³ /s)	Rahans (34001)
Previous approved (Western CFRAM 2012)	172.2
Updated (April 2017)	175.4

Notes: Annual maxima have been sourced directly from the Office of Public Works and extended with latest flow data.

Flood frequency analysis

Western CFRAM Hydrology Report (2014)



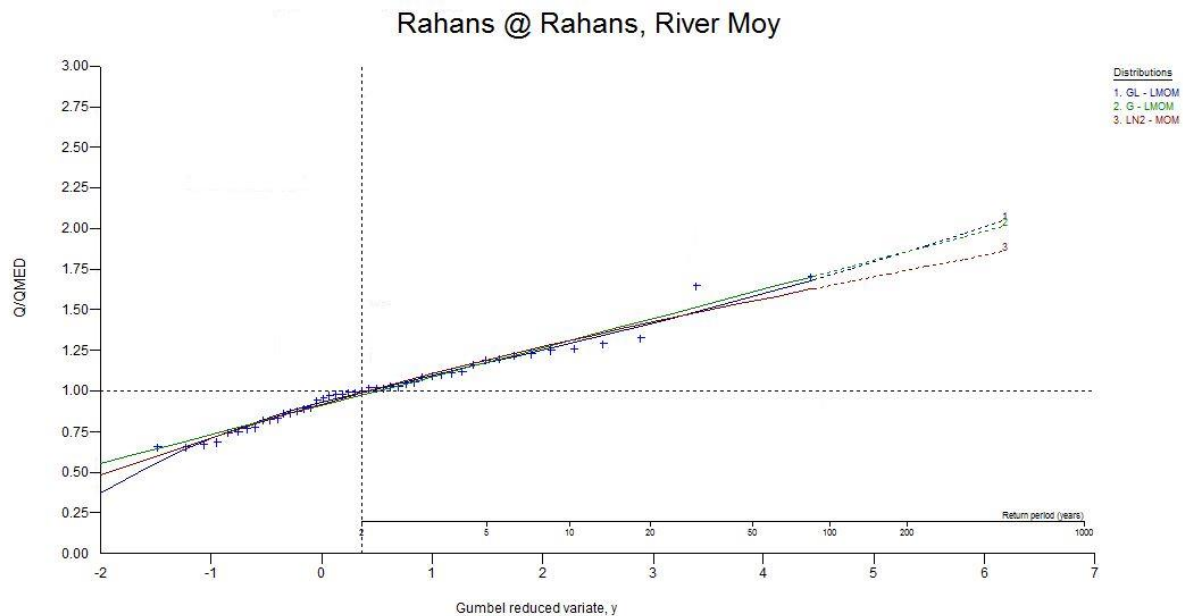
The Gumbel (G) distribution has been fitted using L-moments and the 2-parameter log-normal (LN2) distribution using moments.

Without the presence of the October 1989 event, these annual maxima would have a probability distribution of a strongly concave downwards shape. Although this shape could be better fitted by a 3-parameter distribution, introducing a third parameter increases the standard error. However this high magnitude event in 1989 has forced a more linear distribution fit, which may be erroneous if this flow were overestimated. This should be taken into consideration when looking at the flood frequency of particularly high magnitude, long return period events.

With this in mind, the 2-parameter distributions: the Gumbel distribution and the 2-parameter lognormal have been plotted. The LN2 has been selected as it provides a more realistic fit to this

generally concave distribution.

Updated with inclusion of latest data (April 2017)



The inclusion of recent peak flows adds confidence in the growth curve for flows with a %AEP of between 10% and 20%. The October 1989 and December 2015 peak flows appear as visual outliers. With the December 2015 peak flow there appears to be a slight upwards shift to the flood frequency curve, principally due to the plotting positions of the AMAX series distribution.

A pooled growth curve is used to determine hydraulic model inflows at Hydrological Estimation Points (HEPs). This is compared to the latest single site growth curves for Rahans in the table below. There is insufficient difference between the pooling group growth curve and any of the single site growth curves to justify any update to the Ballina AFA model inflows.

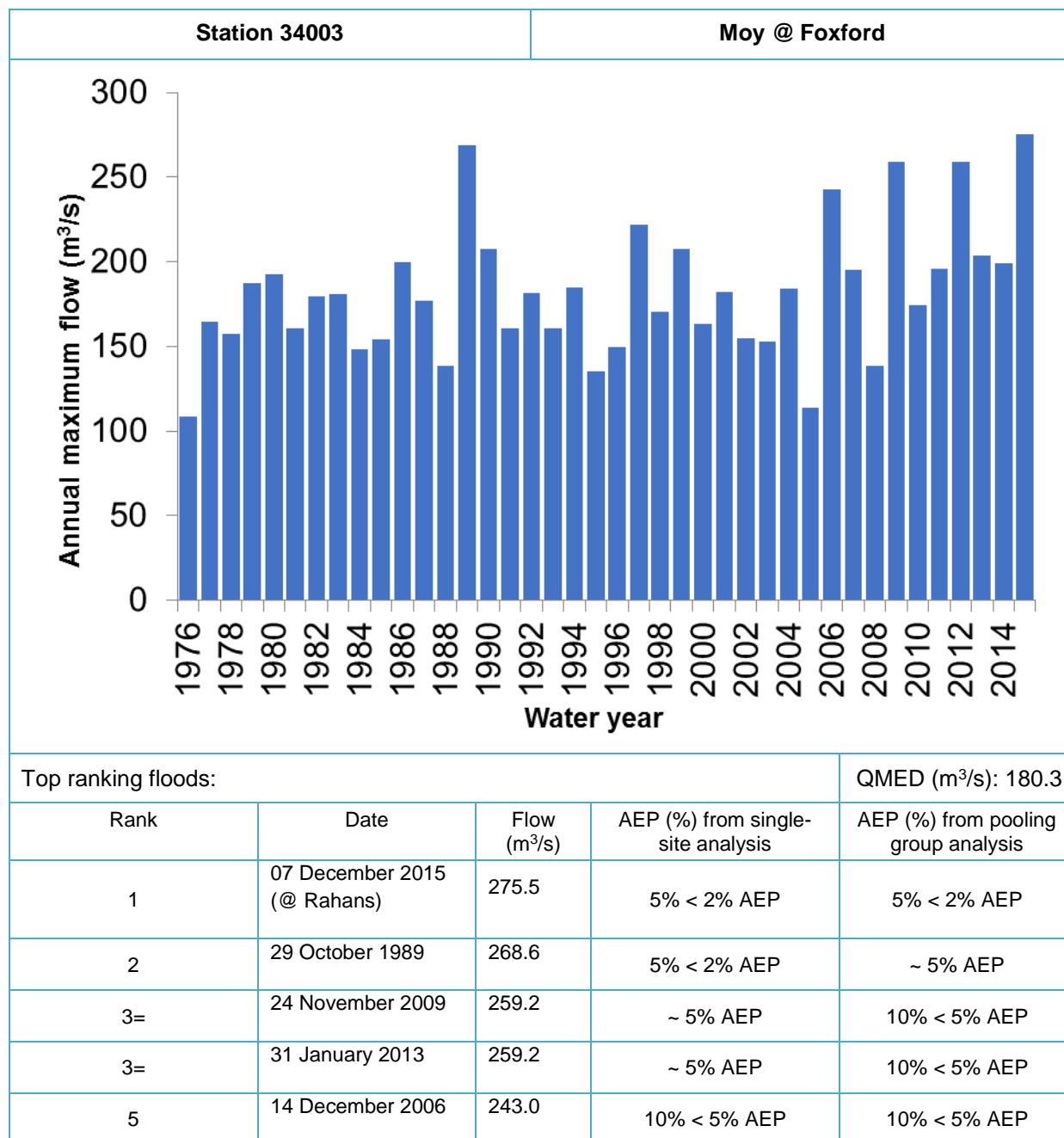
The updated QMED and inclusion of the December 2015 peak flood, give sensible return period estimates for the December 2015 and October 1989 flood events.

Growth curve comparisons at Rahans

	% AEP (Q/QMED)							
	50%	20%	10%	5%	2%	1%	0.5%	0.1%
HEP inflows pooling group (2014)	1.00	1.20	1.33	1.46	1.64	1.78	1.94*	2.35*
Updated AMAX single site GL-LMOM (2017)	0.992	1.173	1.293	1.413	1.578	1.713	1.857	2.233
Updated AMAX (without Oct 89) single site G-LMOM (2017)	0.975	1.178	1.313	1.441	1.608	1.733	1.858	2.146
Updated AMAX (with Oct 89) single site LN2-MOM (2017)	0.989	1.190	1.311	1.420	1.554	1.650	1.743	1.952

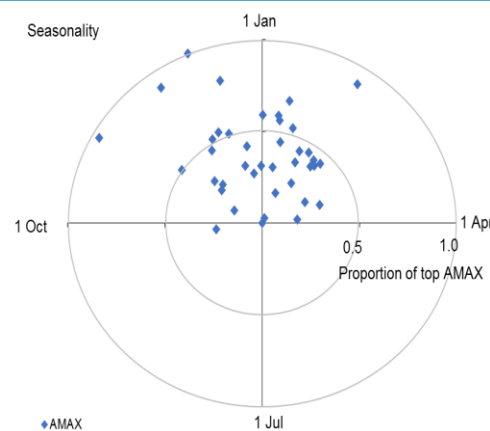
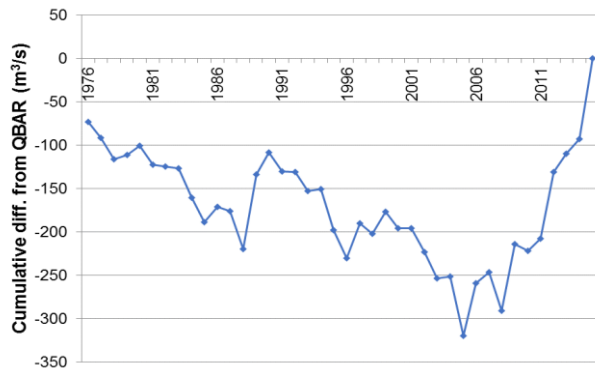
*for the HEP inflows used in the hydraulic modelling the 0.5% and 0.1% AEP peak flows were based upon FSR unit hydrograph methods and based upon a ratio of the 1% AEP flow of 1.112 for the 0.5% AEP and 1.467 for the 0.1% AEP.

Flood peak series summary sheet



Tests for stationarity:

Mann-Kendall test: no significant trend



AMAX Data Series

The AMAX data series has been updated in April 2017 to take account of recent flood events, the latest EPA stage-discharge rating equation and analysis of the relationship between this gauge (Foxford 34003) and Rahans (34001) in Ballina. The Foxford gauge was operated as part of the EPA hydrometric network and has been inactive since 2013.

As part of the EPA hydrometric network, the Foxford gauge (34003) was not shortlisted to be subject to a rating review during the early stages of the Western CFRAM in 2011. The previously approved Western CFRAM hydrology is based upon an AMAX series taken direct from FSU data, which is based upon the previous EPA rating version C2. The EPA Rating was revised on 31 March 2010 and takes account of a spot flow gauging on 20 November 2009. The latest version C3 supersedes versions C1 and C2 of the rating.

The updated Foxford AMAX series in April 2017 is based upon:

1. Application of version C3 of the rating to the full AMAX data series. This is appropriate because the last major change to the catchment and channel through Foxford was during the Moy Arterial Drainage Scheme works which were completed before the installation of the Foxford gauge. As version C3 of the rating takes account of the highest spot flow gauging available (20/11/2009) it is an appropriate update to all historic data at the gauge. The EPA have confirmed this is the correct rating for use. This latest rating is not currently applied to the FSU AMAX series and as the site was not selected for a rating review was not updated to reflect this latest rating.
2. Extension of the AMAX series with the latest available data from the end of the FSU gauge AMAX record in 2004 to the last full water year (2012) before the gauge became inactive. The AMAX data has been extracted from the full 15 minute interval flow record from the EPA hydronet site downloaded on 05 April 2017. The EPA have confirmed that the flow data on the hydronet site is based on the current rating (version C3).
3. Extension of the AMAX series for full water years after the gauge became inactive (2013 to 2015). The purpose of this is to take account of the recent Winter 2015/16 flooding that occurred in Foxford in the hydrological analysis. This extension is based upon a relationship between the Foxford (34003) and Rahans (34001) gauges during the water years when both gauges are operational (1976 to 2012, excluding 1985 where there is a missing AMAX series record for the Rahans gauge. Further details and validation of this relationship are outlined below.

The AMAX data for this site indicates that flood events are most likely to occur in the autumn and winter months, with a relatively even distribution of flood events throughout these seasons. The apparent step change evident in the plot of cumulative deviation from the mean should not be misinterpreted. This just reflects the occurrence of two large floods towards the end of the series (Nov

2009 and Dec 2015) - there is no clear change in gradient evident within this plot. Statistical tests support that no long term significant trend is present within this dataset.

Foxford-Rahans Relationship

As a means of extending the AMAX record series at Foxford, specifically to include an estimate of the December 2015 flow, a regression analysis between the Rahans (34001) and Foxford (34003) gauges has been developed. A regression equation has been applied to the Rahans AMAX data for water years since the Foxford gauge became inactive. The resulting estimates are then added to the Foxford gauge AMAX record series.

1. Catchment comparison

The Rahans gauge (34001) just upstream of Ballina is classified with an FSU A2 quality class, to denote a high quality record series used in the development of the FSU methods. There is less than a 10% increase in catchment area with no significant tributaries joining the River Moy between the gauges.

Comparisons with other gauges upstream on the Moy catchment are less appropriate. The Ballylahan gauge is frequently bypassed during high flows and is upstream of the confluence with the River Deel and Lough Conn & Cullin systems which can store flood flows. The QMED estimates from both catchment descriptors and statistical AMAX series at Ballylahan are significantly higher than the QMED estimates at Foxford and Rahans. The higher catchment descriptors QMED estimate for Ballylahan is a function of the greater proportion of impermeable soils (lower BFISOIL), less attenuation (higher FARL) and steeper slopes (S1085), despite the significantly smaller catchment area.

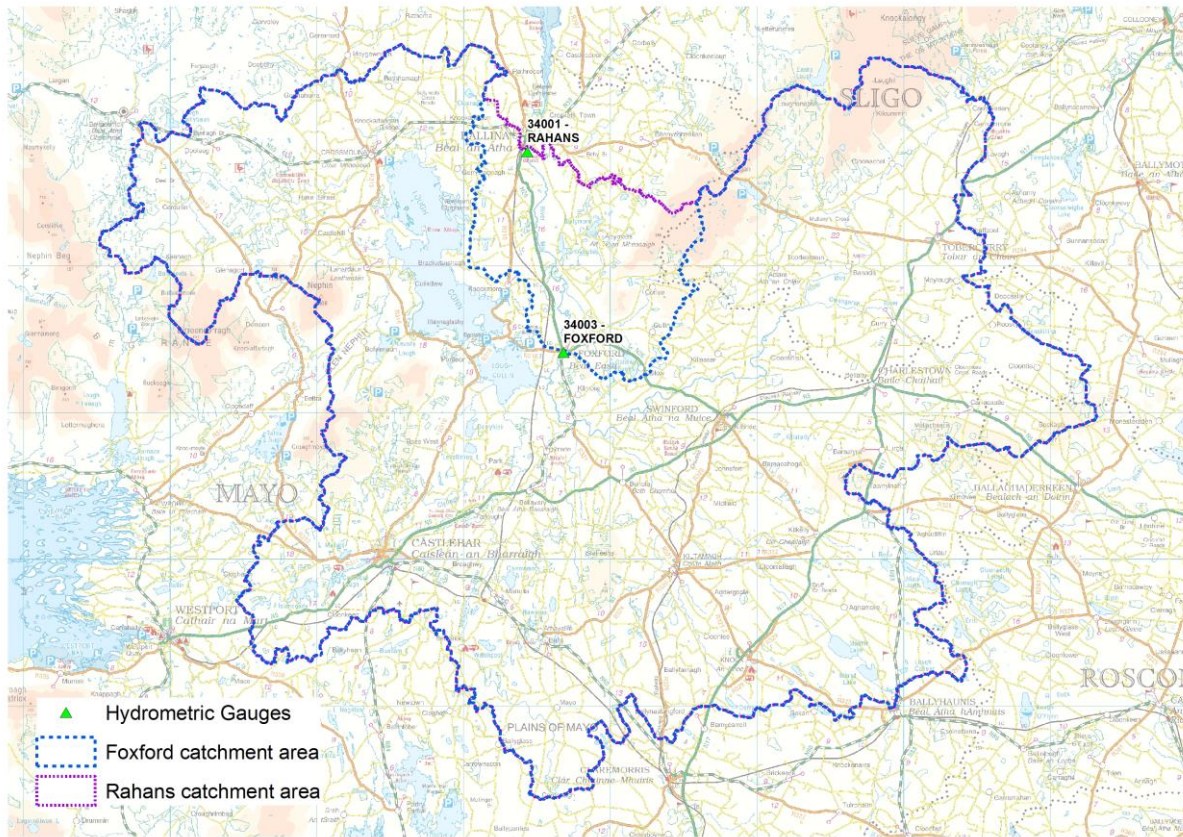
The main catchment descriptors for the gauges are listed below and show that the main differences in the catchments is the gradient and storage.

Selected Catchment Descriptors (Western CFRAM hydrological assessment 2012)

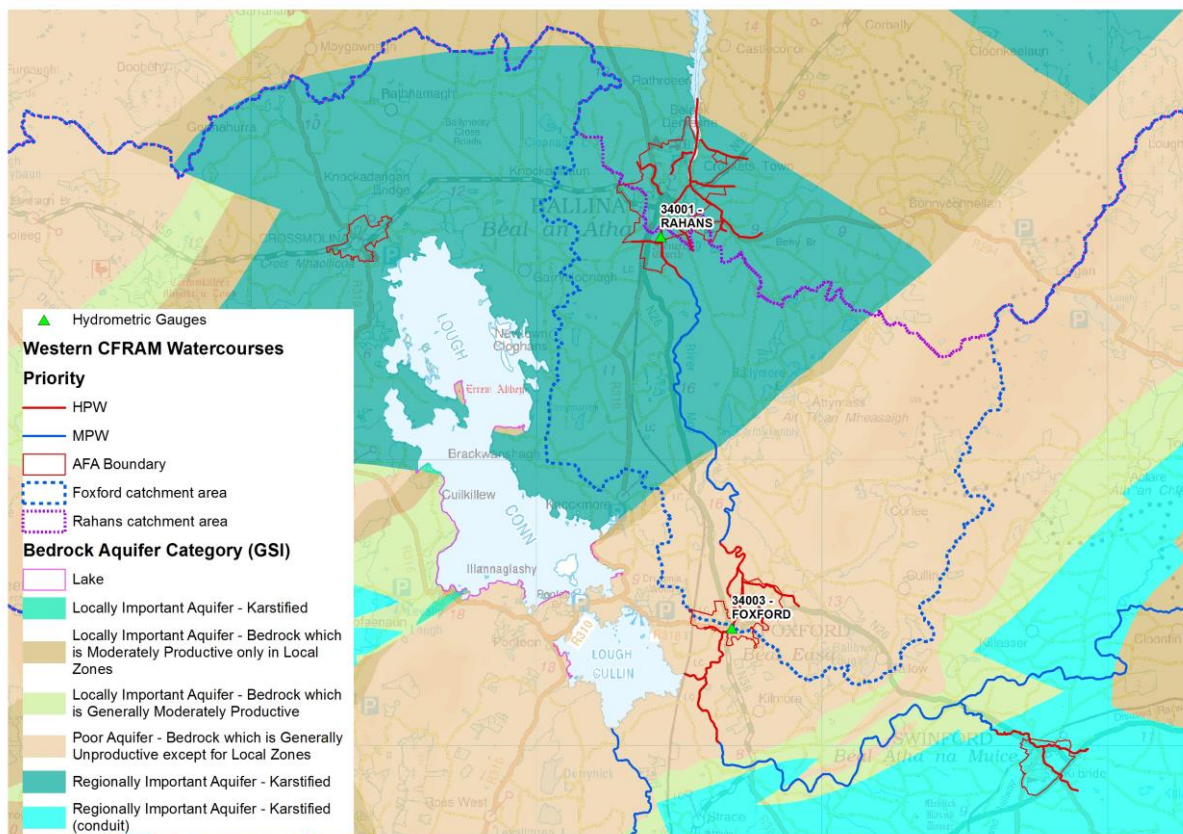
Catchment Descriptor	Rahans (34001)	Foxford (34003)	Ballylahan (34004)
AREA km ²	1974.7	1802.38	935.42
MSL	89.15	69.47	59.45
S1085	0.73	0.96	1.12
DRAIN	1.35	1.37	1.34
ARTDRAIN2	33.59	33.72	38.53
FARL	0.82	0.82	0.96
SAAR	1323	1339	1292
URBEXT	0.0082	0.0082	0.0081
BFISOIL	0.78	0.79	0.48
QMED _{CD}	174.24	167.65	201.48

There are two geology and topography types within the increase in contributing catchment between Foxford and Ballina. There is a band of intrusive granite, quartzite, gneiss and schist formations upon which Foxford is located. This band continues to the East and North East of Foxford and this impermeable bedrock results in poorly drained soils with the upland areas being covered by blanket bog peatland. To the north of this band is an area of limestone bedrock which is classified as a karstified regionally important aquifer. The path of the River Moy through Foxford overlays the impermeable bedrock. To the north of Foxford (from 6km north of Foxford Bridge to downstream of Ballina) the path of the River Moy overlays the karstic groundwater body. The karstic groundwaterbody also underlays Lough Conn suggesting the potential for groundwater interactions between Lough Conn and the River Moy between Foxford and Ballina. To the East of the River Moy there are a number of lakes at the base of the slope at the boundary between the impermeable granites and permeable limestone bedrock.

Map showing respective contributing catchment areas



Map showing groundwater aquifer types



2. Comparison of AMAX record series

The gauged AMAX record series at Rahans and Foxford have been compared. For 17 years the peak flow at the gauges occurs on the same day, with a total of 28 years where the peak flow occurs within two days of each other. Water years 1987 and 1988 have also been excluded from the analysis as the AMAX peak flow at the two gauges occurs 10 or more days apart, suggesting separate flood events. These peak flows are ranked the 19th and 34th (at Rahans) and 18th and 32nd (at Foxford) highest AMAX peak flows. At neither gauge are these critical in the flood history.

This similarity between the Rahans and Foxford AMAX peak flows for each water year suggests a close relationship between the two gauges which is expected given Rahans is downstream of Foxford with less than 10% increase in catchment area and no significant tributaries joining the River Moy between the towns.

The table below presents this comparison, with all flows at Foxford based upon the latest EPA rating.

Recorded AMAX peak flow series at Rahans (34001) and Foxford (34003) gauges

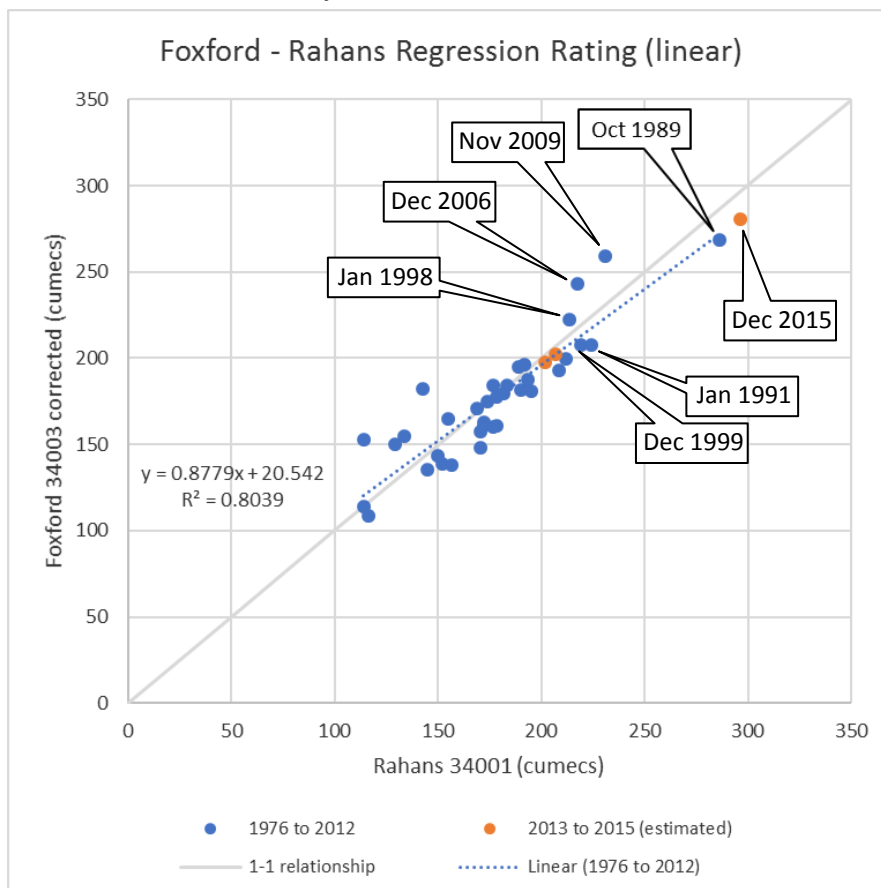
Rahans (34001)				Foxford (34003)				Notes
Date	Year	Flow	Rank	Date	Year	Flow	Rank	
11/02/69 (1)	1968	119.31	44					
20/02/1970	1969	165.92	30					
04/11/1970	1970	150.64	35					
21/11/1971	1971	143.24	38					
12/12/1972	1972	130.33	42					
17/01/1974	1973	177.00	21					
22/01/1975	1974	163.77	31					
11/01/1976	1975	134.57	40					
21/01/1977	1976	116.62	45	21/01/1977	1976	113.00	37	
18/11/1977	1977	155.15	33	12/11/1977	1977	171.00	21	
16/11/1978	1978	170.63	27	15/11/1978	1978	164.00	26	
26/11/1979	1979	193.38	13	26/11/1979	1979	196.00	8	
03/11/1980	1980	208.66	9	03/11/1980	1980	201.00	7	
12/03/1982	1981	172.22	25	12/03/1982	1981	168.00	23	
20/12/1982	1982	181.85	18	19/12/1982	1982	187.00	17	
02/02/1984	1983	195.05	12	06/02/1984	1983	189.00	16	
21/09/1985	1984	170.63	27	21/09/1985	1984	154.00	31	
				21/12/1985	1985	161.00	27	
06/12/1986	1986	212.12	8	05/12/1986	1986	209.00	6	
19/01/1988	1987	178.61	19	09/02/1988	1987	185.00	18	Greater than 10 days between peak flows
13/03/1989	1988	152.13	34	24/03/1989	1988	144.00	32	
30/10/1989	1989	286.56	2	29/10/1989	1989	282.00	1	
06/01/1991	1990	224.44	4	07/01/1991	1990	217.00	5	
09/01/1992	1991	177.00	21	09/01/1992	1991	167.00	25	
24/01/1993	1992	190.05	15	24/01/1993	1992	190.00	14	
29/12/1993	1993	178.61	19	29/12/1993	1993	168.00	23	
29/01/1995	1994	183.48	17	31/01/1995	1994	193.00	12	
26/10/1995	1995	144.70	37	26/10/1995	1995	140.00	34	
08/03/1997	1996	129.52	43	02/03/1997	1996	156.00	30	
11/01/1998	1997	213.86	7	11/01/1998	1997	232.00	4	
26/01/1999	1998	169.06	29	26/01/1999	1998	178.00	19	
25/12/1999	1999	219.12	5	30/12/1999	1999	195.00	10	
13/12/2000	2000	172.22	25	13/12/2000	2000	170.00	22	
11/02/2002	2001	142.72	39	11/02/2002	2001	190.00	14	

10/11/2002	2002	133.61	41	10/11/2002	2002	161.00	27	
02/02/2004	2003	114.16	46	03/02/2004	2003	159.00	29	
09/01/2005	2004	177.00	21	10/01/2005	2004	192.04	13	
20/01/2006	2005	113.96	47	20/01/2006	2005	114.00	36	Extended since FSU
15/12/2006	2006	217.36	6	14/12/2006	2006	243.00	3	
10/12/2007	2007	188.90	16	10/12/2007	2007	195.00	10	
26/01/2009	2008	156.67	32	21/01/2009	2008	138.00	35	
26/11/2009	2009	230.71	3	24/11/2009	2009	259.00	2	
08/02/2011	2010	173.81	24	07/02/2011	2010	175.00	20	
30/11/2011	2011	191.88	14	30/11/2011	2011	196.00	8	
27/01/2013	2012	150.19	36	31/01/2013	2012	143.00	33	
08/01/2014	2013	207.00	10					End of Foxford gauge record.
16/01/2015	2014	201.80	11					
07/12/2015	2015	296.15	1					

3. Regression Analysis

The AMAX peak flow record series at Foxford and Rahans gauges have been plotted to ascertain whether regression analysis can be used to derive a rating relationship between the flows at the two gauges. The plot below shows the coincident AMAX record series as blue points with Foxford on the y-axis and Rahans on the x-axis. A linear regression fits the scatter of points fit better than two or three order polynomial equations. Orange dots represent the estimated Foxford AMAX series based upon a linear regression relationship of the co-incident gauged AMAX peak flows at the two gauges.

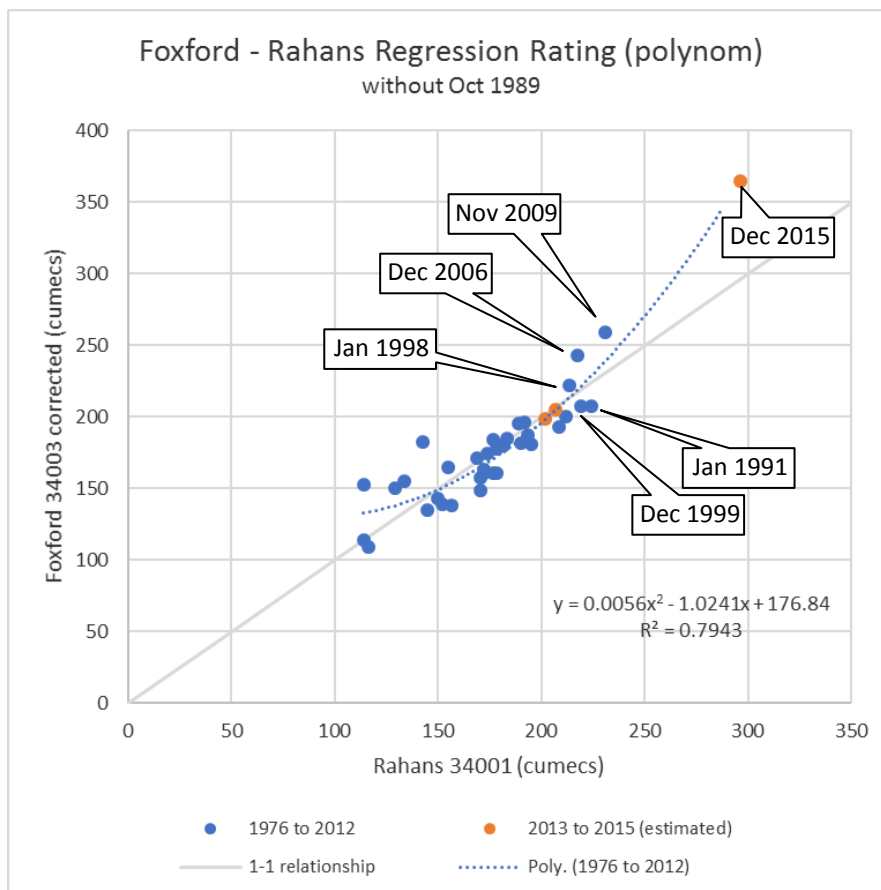
Linear regression rating between Rahans (34001) and Foxford (34003), with estimated AMAX peak flows for Foxford for water years 2013, 2014 and 2015.



A faint diagonal line has been included to indicate whether peak flows for each AMAX record are greater at Foxford or Rahans. Points to the top-left or above this line represent AMAX records where the peak flow at Foxford is greater than the peak flow at Rahans.

The December 2015 peak flow is an extension of the linear relationship and is potentially strongly influenced by the October 1989 peak flow recorded at Rahans and Foxford. The October 1989 flood appears as a visual outlier on the relationship between the two gauges, when compared to the November 2009, December 2006 and January 1998 floods. To understand the uncertainty a sensitivity test has been carried out on the regression analysis to exclude the October 1989 peak flood. A two-order polynomial relationship provides a good visual fit to this sensitivity test scatter plot.

Two order polynomial regression rating between Rahans (34001) and Foxford (34003) – excluding the October 1989 event, with estimated AMAX peak flows for Foxford for water years 2013, 2014 and 2015.



4. Resulting QMED Estimation

The December 2015 peak flow (as estimated by the relevant regression equation) is estimated to be the greatest flow at Foxford from both regression equations. As such it does not have any influence over the QMED estimate (the October 1989 AMAX series is still included for the QMED estimate as it is a valid AMAX series – it is just an outlier for the Foxford-Rahans relationship).

Understanding the relationship between these gauges for the largest flood events on record is important in the determination of flood growth curves and is explored further below, through analysis of the Foxford to Ballina MPW (Medium Priority Watercourse) model. The sensitivity test makes no difference to QMED estimates, however provides a significantly higher estimate for the December 2015 peak flow in Foxford of 364.69 m³/s. This is considerably greater than the recorded peak flow at Rahans of 296.15 m³/s and suggests this grossly overestimates the peak flow.

Statistical QMED estimates at Rahans (34001) and Foxford (34003)

Statistical QMED at gauge (m ³ /s)	Rahans (34001)	Foxford (34003)	Ballylahan (34004)
(Western CFRAM 2012 Hydrology Report)	172.2	176.3	231.0
Updated (April 2017)	175.4	180.3	n/a

It should be noted that the previously hydrological assessment, following the Western CFRAM methodology, estimates QMED at Hydrological Estimation Points (HEPs) using QMED from catchment descriptors adjusted by the statistical QMED estimate at the nearest of the Foxford or Rahans gauges and by the difference in catchment area between the HEP and the gauge location. The adjustment results in lower QMED estimates for HEPs in Ballina than in Foxford. This reflects the statistical QMED estimates and peak flows for the January 1998, December 2006 and November 2009 floods where flows at Foxford are greater than the flow at Rahans, despite the larger catchment area. This could be explained by the increase in floodplain storage attenuating the peak flow between Foxford and Ballina.

During the FSU research the influence of Arterial Drainage Schemes on QMED was investigated. It was found that on 11 of the 15 gauging stations with a pre- and post- drainage scheme record QMED was substantially higher for the post-drainage record¹. It was also found that Arterial Drainage Schemes had a greater impact on QMED for catchments with more permeable soils (lower BFISOIL value). The FSU catchment descriptor formula takes account of arterial drainage schemes through the ARTDRAIN2 catchment descriptor, however local catchment specifics, such as this secondary impact of BFISOIL. In the case of Foxford, Rahans and Ballylahan the long record series reduce the uncertainties associated with using catchment descriptors for QMED estimation. It may be appropriate to use an alternative method, based solely on catchment area, of data transfer for QMED estimation at ungauged HEPs in Foxford and Ballina.

5. Validation of Foxford – Rahans regression analysis

Three pathways to validating the estimate of the December 2015 peak flow derived from the statistical regression relationship have been followed.

- Validation 1 – analysis of flood event gauge data
- Validation 2 – comparison of routing model to gauged flow records at Foxford and Rahans
- Validation 3 – calibration of hydraulic model to the EPA rating and comparison with observed spill levels during December 2015 flooding

Validation 1 – analysis of flood event gauge data

In the November 2009, December 2006 and January 1998 flood events peak flows at Foxford are higher than the downstream gauged flows at Rahans. This pattern is not replicated in the October 1989, December 1999 and January 1991 flood events. The linear regression equation estimates the December 2015 peak flow at Foxford to follow the same pattern as the October 1989 flood, principally due to the October 1989 flood being a key point on the regression line. The table below presents a comparison of the preceding rainfall for the December 2015 and October 1989 flood events at Rahans and Foxford. Preceding rainfall and flood response times to the October 1989 and December 2015 are similar, which suggests that these two events have similar flood mechanisms. This comparison gives no reason to exclude the October 1989 from the development of the regression relationship.

¹ Flood Studies Update Technical Research Report Volume II – Flood Frequency Estimation <http://opw.hydronet.com/default.aspx?page=11>

Comparison of rainfall data preceding the seven largest flood events at Rahans and Foxford.

	Moy catchment 2 day rainfall		Moy catchment 8 day rainfall		Lag time	Notes
	Depth (mm)	AEP (%)	Depth (mm)	AEP (%)		
Oct 1989	108.6	1.0	173.3	1.0	5 days	Significant daily peak of rainfall on 26 October 1989. 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. At Belmullet (NW corner of County Mayo) it was the wettest October since records began, with 129mm recorded in a 36 hour period.
Dec 2015	Not evaluated as part of Western CFRAM inception report flood event analysis. Comparable 2 day and 8 day rainfall depths to the October 1989 event.				1<2 days (estimated)	Storm Desmond was the most significant of a number of Atlantic depressions that affected Ireland in Winter 2015/16. At Knock Airport continuous rainfall from 16:00 on 04 December until 04:00 on 6 December with a total of 96.5mm over the 34 hour period. 95.2mm rainfall in the 11 preceding days from 23/11/2015 to 03/12/2015 with an 8 day rainfall depth of 161 mm.

Validation 2 – comparison of routing model to gauged flow records at Foxford and Rahans

As documented in the updates to the Hydraulic Modelling Report² and Model Check Files³ for Foxford, the detailed Foxford River Moy HPW (High Priority Watercourse) has been extended to include the Foxford to Ballina MPW model, and also subject to detailed model review in response to the December 2015 flooding. The schematisation of the model has been refined and the extension of the downstream boundary to 2.5km upstream of the Rahans gauge (the location of the MPW downstream boundary) allows for comparison of the model outputs to gauged records of flow.

The hydraulic model reports include documentation of the calibration of the refined model to the gauged November 2009 and October 1989 flood levels at Foxford.

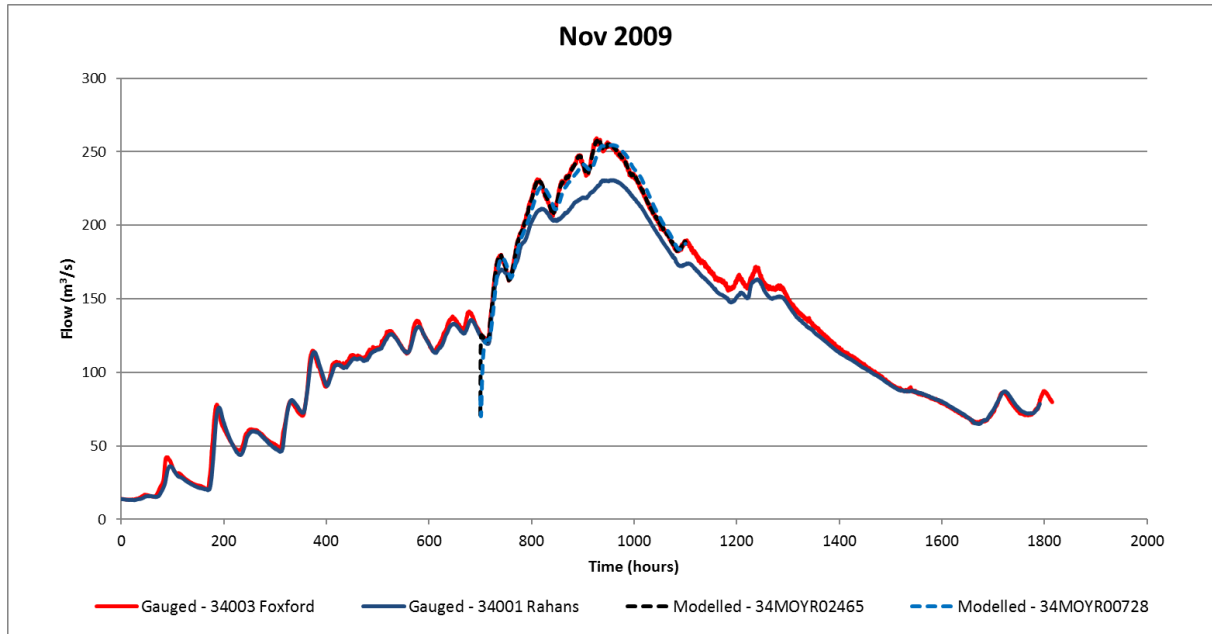
The recorded 15 minute gauged flows at the Foxford gauge have been used as upstream model inflows to simulate the November 2009 and October 1989 flood events. The December 2015 flood event has been simulated by transferring the recorded 15 minute gauged flows at Rahans, adjusting each flow observation by the linear Foxford-Rahans regression equation. The downstream boundary of the model is based upon a stage discharge relationship from the Ballina River Moy HPW model. For both the November 2009 and October 1989 the peak flow hydrograph input into the flood model matches the model output flow hydrograph at the location of the Foxford gauge.

At downstream end of the extended model the modelled November 2009 flow hydrograph has higher flow at and around the peak of the event than the gauged Rahans flow records (modelled flow is 23.8 m³/s higher than the gauged peak flow). The modelled hydrograph near Rahans was a close match to the inflow hydrograph at Foxford with a smoothing out of some minor peaks and troughs in the hydrograph shape. The gauged Rahans hydrograph shape was also smoother than the shape at Foxford.

² WCFRAM UoM34 Moy Killala Bay Hydraulic Modelling Report Vol 2d Foxford

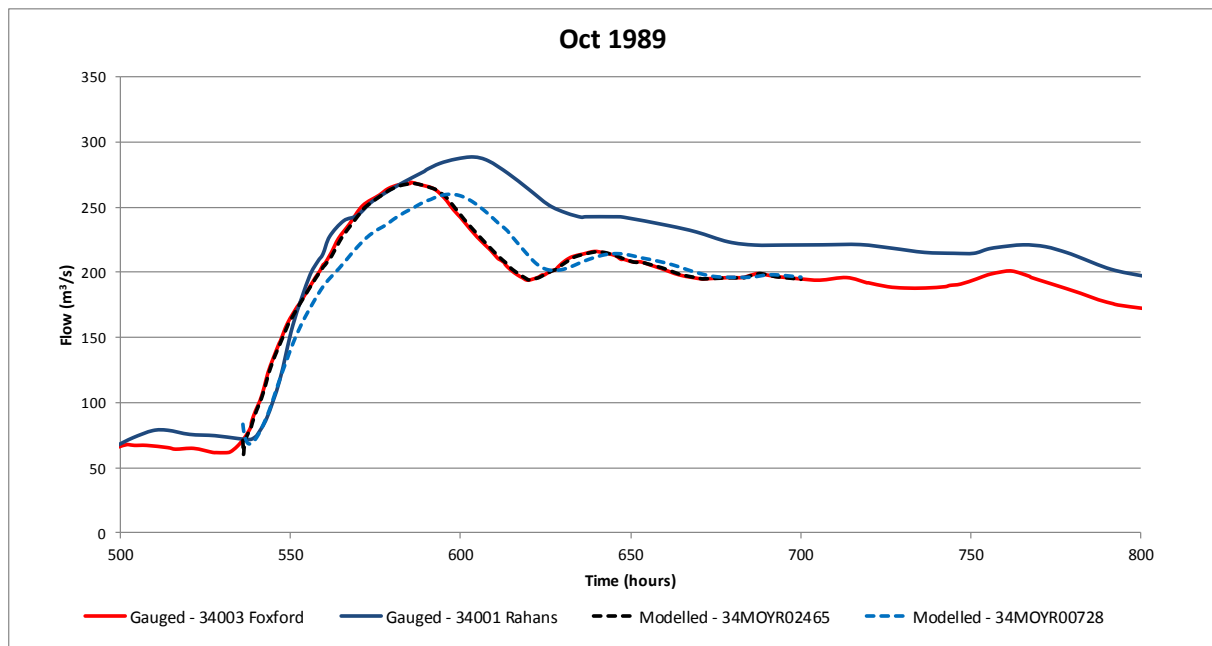
³ 2012s6485 River Moy Foxford Hydraulic Model Check File

November 2009 modelled and gauged flood hydrographs



In the October 1989 simulation, the downstream end of the extended model the modelled flow hydrograph has a lower flow at and around the peak of the event than the gauged Rahans flow records (modelled flow is 28.6 m³/s lower than the gauged peak flow). This is the opposite of the pattern in November 2009. The flood hydrograph shape in the October 1989 is far smoother with a shorter time to peak than the observed November 2009 event.

October 1989 modelled and gauged flood hydrographs



The main observations from these model simulations are:

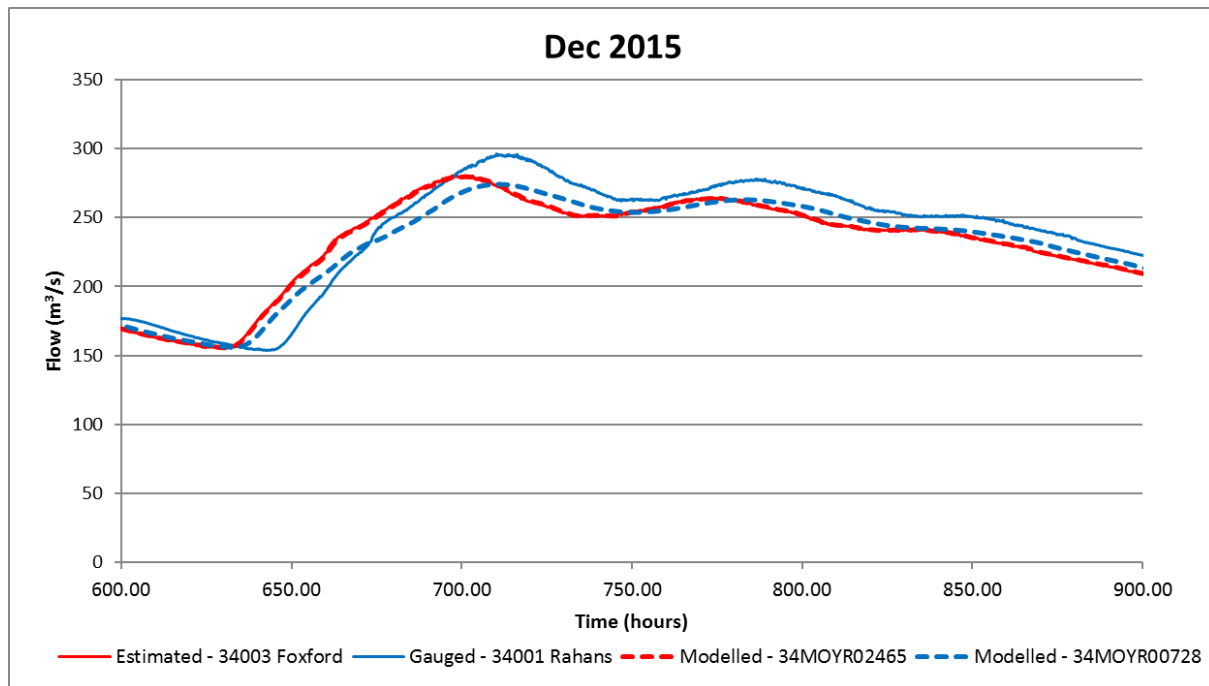
- The downstream model hydrograph shape (resulting from the gauged Foxford flows used as model inflows) reflect the gauged hydrograph shape at Rahans in both events. In November

2009, the smoothing of distinct peaks is represented in the flood model.

- In October 1989, between Foxford and Ballina, there are some extra inflows, less attenuation or channel conveyance is greater than in November 2009.
- The duration and time to peak in November 2009 is longer than in October 1989.

The simulation of the December 2015 flood event follows a similar pattern to the October 1989 comparison. The shape of the flood hydrographs at Rahans and Foxford gauges are similar to the October 1989 flood. The November 2009 flood has more individual peaks in the Foxford hydrograph. The modelled flow is 22.1 m³/s lower than the gauged peak flow at Rahans. The similarity between the October 1989 and December 2015 gauged and modelled hydrographs give weight to validate the Foxford-Rahans regression analysis.

December 2015 modelled and gauged flood hydrographs



This analysis suggests that the River Moy between Foxford and Ballina responds differently, depending upon groundwater and sub-catchment inflows and conditions. With the longer flood duration and time to peak, it is possible that the sub-catchment inflow in Nov 2009 came and went before the peak on the River Moy passed Rahans, and that in Oct 1989 and Dec 2015 sub-catchment inflows coincided closer to the peak flow at Ballina. Groundwater contributions and the discharge of flood water stored in Lough Conn and Cullin are likely to be the mechanism that keeps the flow higher at Ballina over a prolonged period of time after the peak.

These assumptions are extremely uncertain which is highlighted by the preceding flows on the River Moy remaining lower in November 2009 for longer than in December 2015. This counters the groundwater contribution explanation, which would be expected to be higher given the high rainfall in the preceding days and weeks before the November 2009 flood.

Attempts to model these individual flood event variations would be extremely uncertain and would not give any extra useful insight. It is possible that any matches would be entirely likely to be co-incidence. Further detailed analysis of the catchment conditions, spatial rainfall patterns, groundwater and lake level monitoring is required to validate this pattern. Secondly, forestry

operations could potentially have a significant impact as notable areas will have been harvested between the flood events. Arterial Drainage maintenance is unlikely to make a difference due to the cyclical nature of work on different sections or tributaries each year, unless conveyance at key pinch points has been improved (e.g. Ballylahan and other bridges upstream on the Moy, the link channel between Lough Cullin and the River Moy, Foxford bridge, works to the Salmon weir and footbridge in Ballina). There is also the possibility that there could be cumulative impacts resulting from undocumented works by landowners to improve the performance of field/peatland drains during flooding.

Validation 3 – calibration of hydraulic model to the EPA rating and comparison with observed spill levels during December 2015 flooding

Calibration to the latest EPA stage discharge rating curve was undertaken during the detailed review of the Foxford River Moy HPW model in response to the December 2015 flooding. Further details are contained in the Hydraulic Modelling Report⁴ and Model Check Files⁵ for Foxford.

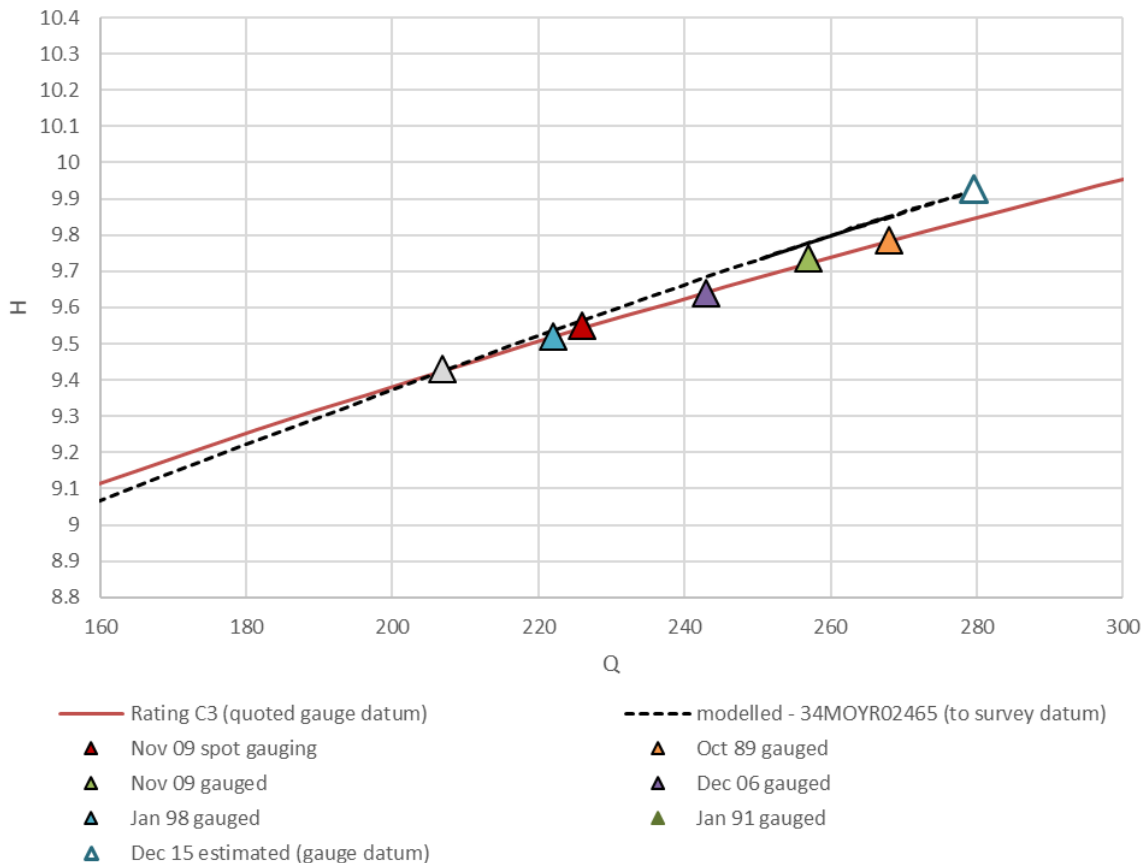
Comparison of model and EPA stage discharge rating at the location of the Foxford gauge (model outputs adjusted to the gauge datum⁶)

⁴ WCFRAM UoM34 Moy Killala Bay Hydraulic Modelling Report Vol 2d Foxford

⁵ 2012s6485 River Moy Foxford Hydraulic Model Check File

⁶ Hydraulic model is based upon survey which quotes the gauge datum to be 102mm higher than the EPA quoted gauge datum. Hydraulic model outputs adjusted to be consistent.

Comparison of model and EPA stage discharge rating at Foxford (34003) - all gauged flows based on current rating and EPA gauge datum



The upper bound of the latest EPA rating is the spot flow gauging carried out in November 2009. This is lower than the peak flow estimate for November 2009. Above the upper bound the rating equation has been extrapolated and peak flow estimates as recorded at the gauge here are uncertain.

The hydraulic model stage discharge relationship at the location of the gauge is heavily influenced by the conveyance through Foxford Bridge. The level of afflux through this structure in the model has been compared to video footage taken during the December 2015 flood event showing both the upstream and downstream face of the bridge and the spill of water onto Green Lane further downstream. These three observations have been used to calibrate the model water level profile through Foxford, including the location of the gauge. The modelled peak water level of 9.90 mOD Malin (corrected to gauge datum) is a close match to the observed flood levels.

The resulting hydraulic model provides a very close match to the EPA rating at the November 2009 spot flow gauging. The hydraulic model rating diverges from the extrapolation of the EPA rating as stage increases. Based on the above estimate of peak flood level at Foxford, the extrapolated EPA rating estimates the peak flow for the December 2015 event to be 291 m³/s. This is 11.2 m³/s more than the estimated peak from of 279.8 m³/s from the Foxford-Rahans relationship. This is well within an acceptable tolerance given the uncertainty in both methods.

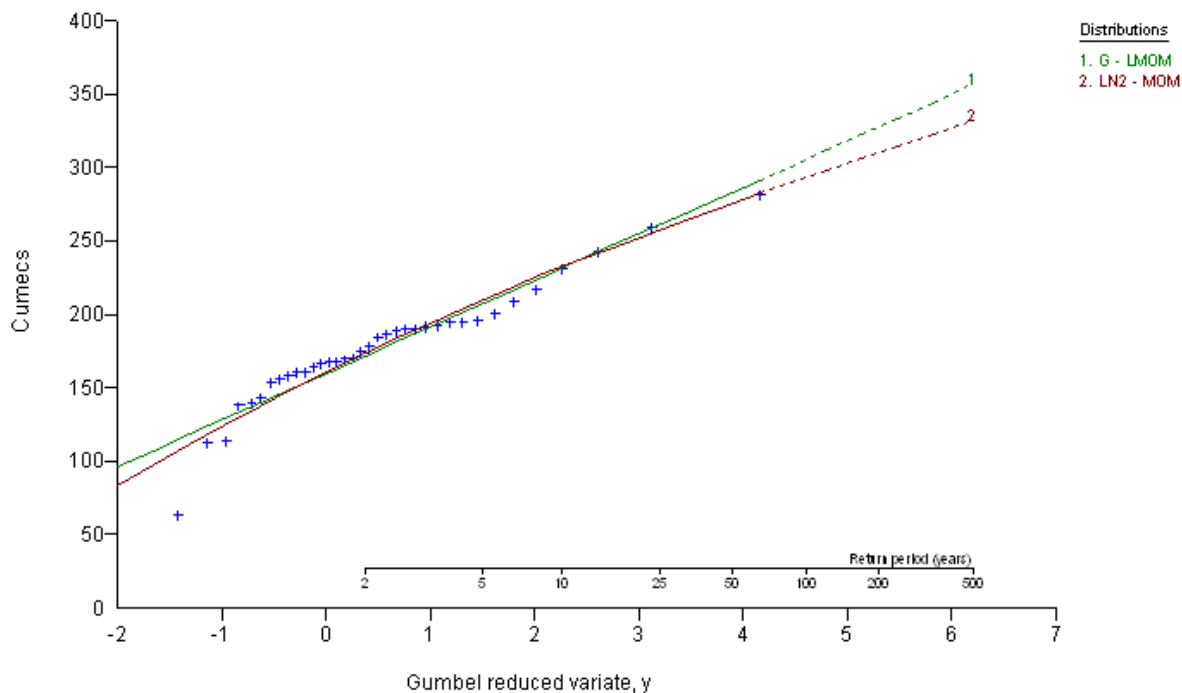
These considerations are considered further in the selection of flood growth curves, as documented below.

The validation can conclude that the Foxford-Rahans relationship is uncertain but generates a close match to the EPA rating curve extrapolated beyond its upper bound. There is too much uncertainty in

the routing model to be of use in replicating peak flows at Foxford and Rahans. The statistical Foxford-Rahans regression relationship can be used to extend the Foxford AMAX record series to include peak flow values for the 2013, 2014 and 2015 water years to include the December 2015 flood. This is preferred over the use of the EPA rating because there are no water level observations or records to estimate the peak flow for the 2013 or 2014 water years.

Flood frequency analysis

Previous Western CFRAM Hydrology Report (2012)



In the previous analysis, the Gumbel (G) distribution was fitted using L-moments and the 2-parameter log-normal (LN2) distribution using moments. The October 1989 flood caused the probability plot to have a slight concave upwards shape. Although this shape could be better fitted by a 3-parameter distribution, introducing a third parameter increases the standard error. In addition it is possible that the parent distribution is 2-parameter and that these two events were outliers with very long return periods. It is also possible that the flow was underestimated during October 1989. With these considerations in mind, and bearing in mind the recommendations from FSU work package 2.2, only 2-parameter distributions were fitted.

Either the Gumbel or log-normal distribution appears to be a reasonable fit to the sample of annual maximum flows. They give similar flood frequency curves. The LN2 distribution has been selected as it gives the best fit to flood peak data at both the low and high extreme peak flows.

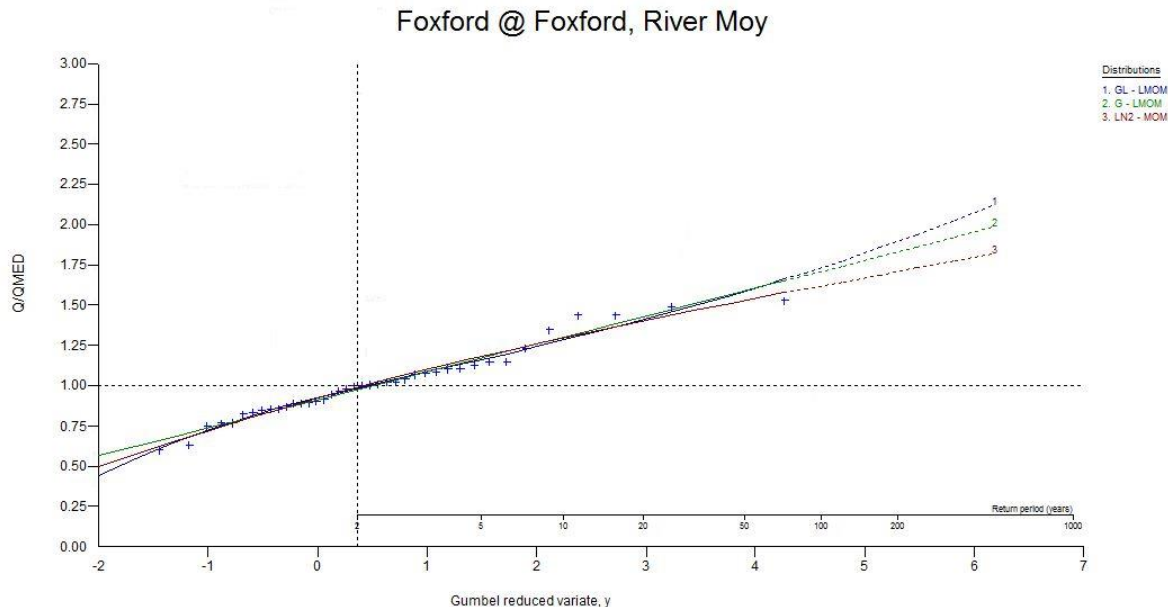
Parameters of the fitted LN2 distribution:

$$u = 5.16$$

$$\sigma = 0.23$$

This distribution was used to estimate the AEPs quoted in the appendix to the Hydrological Report. In the main stage of the study a pooled flood growth curve is used to estimate inflows at HEPs to accommodate a longer-term flood history.

April 2017 update (with full Foxford-Rahans Relationship)



In addition to the Gumbel and 2-parameter log-normal distributions the Generalised Logistic (GL) fitted with L-moments has also been assessed to consider the December 2015 flow based upon the Foxford-Rahans relationship which excludes October 1989. The inclusion of the October 1989 flood in the Foxford-Rahans relationship makes a significant difference to the extreme of the growth curves. The position of the December 2006 and November 2009 points on the y-axis suggest there could be a step change in the plotted AMAX series between the 5% and 10% AEP. The addition of these events change the October 1989 flood event from influencing a concave to a convex curve. The inclusion of an estimated December 2015 flow results in a convex curve. The distribution fitting will take account of this variance at the extreme end of the distribution. These flood flows have been investigated in the review of the hydraulic model and there is no evidence to suggest there is a threshold for a step change in the AMAX distribution (such as bank-full stage, floodplain storage or structure afflux).

Selection of any of the growth curve distributions plotted above (GL-LMOM, G-LMOM and LN2-MOM) account for the alternative estimate of the December 2015 peak flow using the extrapolated EPA stage discharge rating. The 11 m³/s increase in the estimate of the December 2015 peak flow remains below all three growth curves. As such the uncertainty and variation in peak flow estimate is addressed through the growth curve analysis.

The key questions are whether a single site growth curve is less uncertain than a pooling group and if it is appropriate to exclude the October 1989 flow from the Foxford-Rahans relationship. The October 1989 flood appears to be the sole factor for the significant difference in the fitting of single site growth curves to represent the extreme flows. There is some doubt over whether the Foxford gauge recorded the full peak of the October 1989 flood as documented in the Western CFRAM hydrology reports for Unit of Management 34. The EPA rating curve includes a flow gauging in late November 2009 which suggests the peak flow for this event was correctly recorded by the gauge.

Further the use of single site growth curves reduce the propagation of uncertainties in catchment descriptors in their use in identifying potential pooling group members. The 5 to 10 year cycles of arterial drainage scheme maintenance are unlikely to have a significant influence on AMAX record series.

There is insufficient evidence to justify exclusion of the October 1989 flood event from the Foxford-Rahans relationship and so the linear regression is appropriate. The polynomial fitting excluding the October 1989 event is useful as an indicator of possible uncertainty in single site flood estimation.

Growth Curves

The table below presents previous and updates to the flood growth curves. The latest analysis has only considered a single site growth curve to reduce the uncertainty in identifying similar catchments based upon catchment descriptors. The latest rating at Foxford (version C3) which takes account of the November 2009 flow further reduces uncertainty in the single site analysis. This uncertainty will need to be offset against fewer AMAX peak flows in the single site than contained within the pooling group. The table includes the latest G-LMOM as a direct comparison based upon both the with and without October 1989 Foxford-Rahans relationship used to estimate the December 2015 flow at Foxford. The LN2-LMOM growth curve is a close fit to the AMAX series with October 1989 included in the Foxford-Rahans relationship and the GL-LMOM distribution for the AMAX series without October included in the Foxford-Rahans relationship. These provide the best visual fitting to the extreme events for their respective AMAX series as highlighted below.

Growth curve comparisons at Foxford (preferred growth curve highlighted)

	% AEP (Q/QMED)							
	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Hydrology report single site G-LMOM (2012)	0.962	1.185	1.333	1.475	1.659	1.797	1.934	2.252
HEP inflows pooling group (2014)	1.00	1.20	1.33	1.46	1.64	1.78	1.94*	2.35*
Updated AMAX (with Oct 89) single site G-LMOM (2017)	0.975	1.172	1.302	1.427	1.589	1.710	1.830	2.110
Updated AMAX (without Oct 89) single site G-LMOM (2017)	0.984	1.201	1.344	1.482	1.660	1.794	1.927	2.236
Updated AMAX (with Oct 89) single site LN2-MOM (2017)	0.989	1.182	1.298	1.402	1.529	1.620	1.708	1.905
Updated AMAX (without Oct 89) single site GL-LMOM (2017)	0.977	1.176	1.323	1.484	1.729	1.947	2.199	2.952
*for the HEP inflows used in the hydraulic modelling the 0.5% and 0.1% AEP peak flows were based upon FSR unit hydrograph methods and based upon a ratio of the 1% AEP flow of 1.112 for the 0.5% AEP and 1.467 for the 0.1% AEP. The pooling group growth curve was not used.								

The analysis of the different growth curves generated show that the 1% AEP growth factor could vary from 1.620 to 1.947 depending upon whether it is deemed appropriate to include the October 1989 peak flow in the calculations for the Foxford-Rahans relationship.

Using the previous growth curve and the new QMED estimate increases peak flows for all design events by 2%. This is a small increase, but could potentially result in greater flooding if this flow increase results in flood levels that exceed bankfull or road crest levels in Foxford.

Using the G-LMOM distribution from the updated AMAX series and QMED estimate results in marginally lower peak flow estimates (3% lower for the 1% AEP).

With the GL-LMOM distribution peak flows in excess of the 5% AEP are significantly higher (an increase of 11% for the 1% AEP peak flow) than the inflows used in the 2014 hydraulic modelling. There is no requirement to use the FSR ratios for the 0.5% and 0.1% AEP inflows as this distribution already contains such an uplift in the growth curves. Peak flows less than or equal to the 5% AEP are

very similar to the previous HEP inflows.

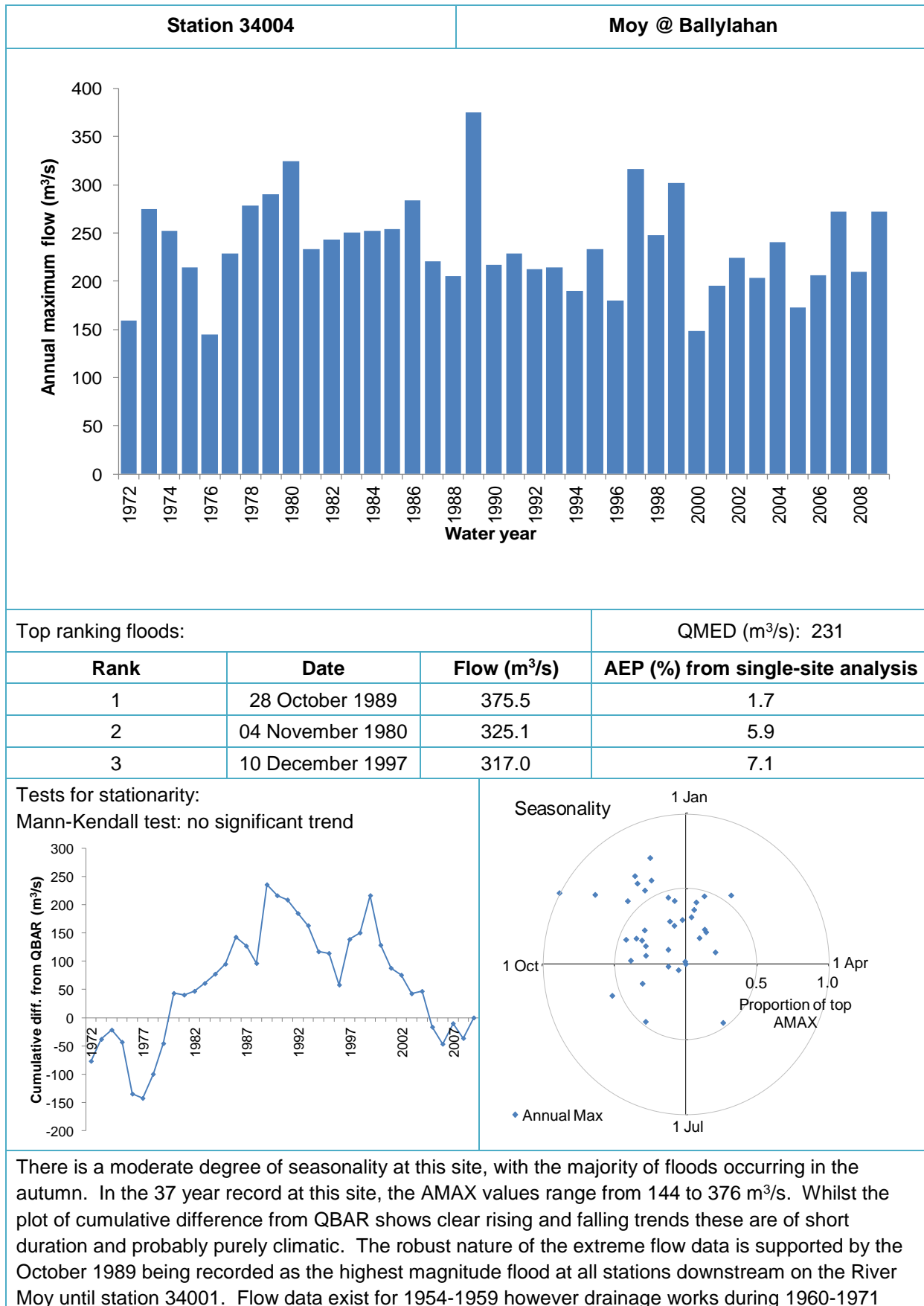
With the LN2-MOM growth curve peak flows are significantly lower than those input into the hydraulic model.

Peak flow estimates (preferred peak flow estimates highlighted)

	% AEP (m ³ /s)							
	50%	20%	10%	5%	2%	1%	0.5%	0.1%
HEP inflows pooling group (2014)	177.51	212.48	235.46	258.46	290.39	316.27	351.58*	464.06*
HEP inflows pooling group (2014) with updated QMED (2017)	180.30	215.82	239.16	262.52	294.95	321.24	357.10*	471.35*
Updated AMAX (with Oct 89) single site G-LMOM (2017)	175.79	211.31	234.75	257.29	286.50	308.31	345.14*	463.39*
Updated AMAX (with Oct 89) single site LN2-MOM (2017)	178.32	213.11	234.03	252.78	275.68	292.09	307.95	343.47
	"	"	"	"	"	"	324.69*	428.57*
Updated AMAX (without Oct 89) single site GL-LMOM (2017)	176.15	212.03	238.54	267.57	311.74	351.04	396.48	532.25
	"	"	"	"	"	"	390.23*	515.08*
*as discussed above the 0.5% and 0.1% AEP flows based on FSR ratios and not the growth factors.								

The review of the hydraulic model shows that water levels are unlikely to exceed the maximum level which the gauge can record and so further enhance the validity of the October 1989 recorded level. The observations of flood levels and bridge afflux during the December 2015 flood event have validated the December 2015 peak flow estimates.

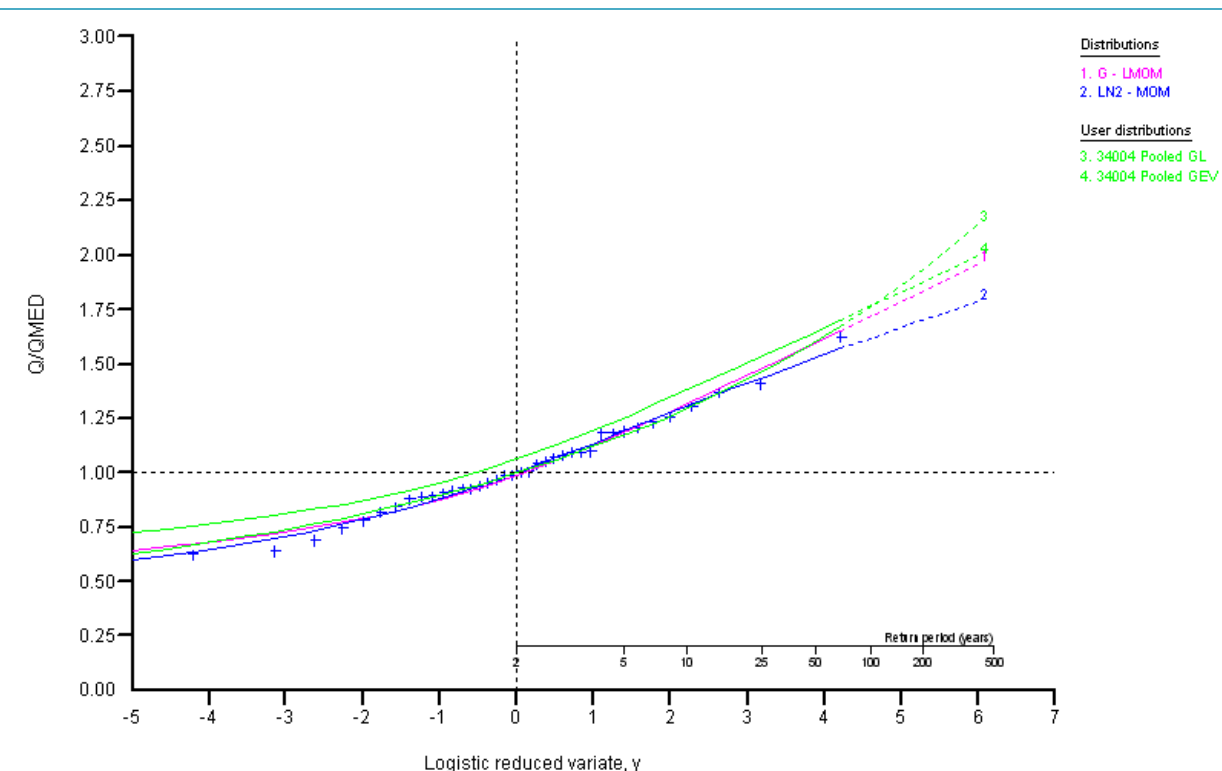
Flood frequency analysis summary sheet



prevent this data from being relevant to the present day hydrological situation. Statistical testing supports the assertion that there is no long term trend evident within this dataset; however caution must be exerted when using these AMAX flows given the classification as a FSU grade C gauge. This implies that further improvements to the rating are necessary before extrapolation to QMED. This assertion is supported by comparison with downstream gauges which suggest flows at this site are being overestimated.

Notes: Annual maxima have been sourced directly from OPW. A rating review was not undertaken at 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)	0.923	0.173	n/a	1.718
Single-site LN2 (moments)	0.001	0.206	n/a	1.618
Pooled GL (L-moments)	1	0.1103	-0.1677	1.760
Pooled GEV (L-moments)	1	0.1682	0.0035	1.710

Comments on growth curves

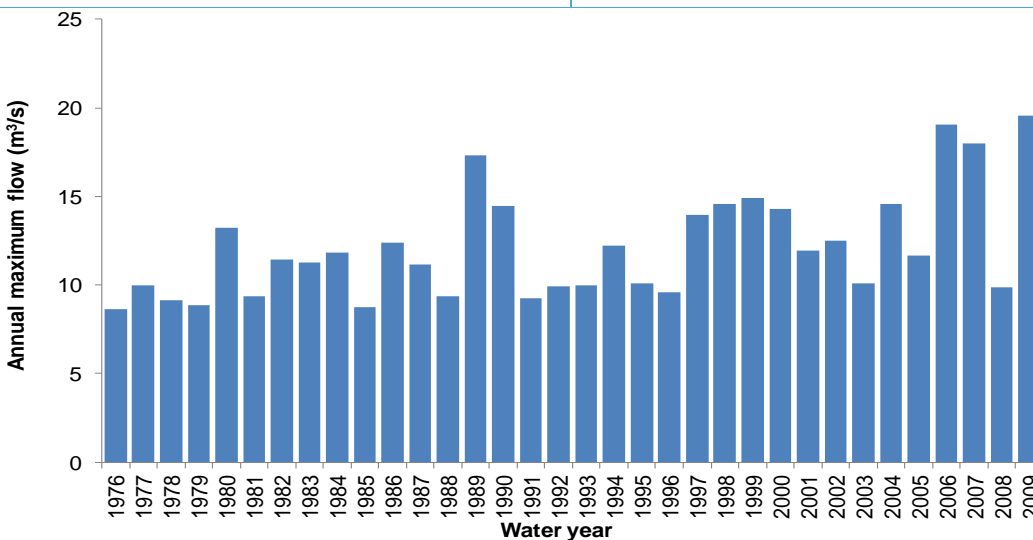
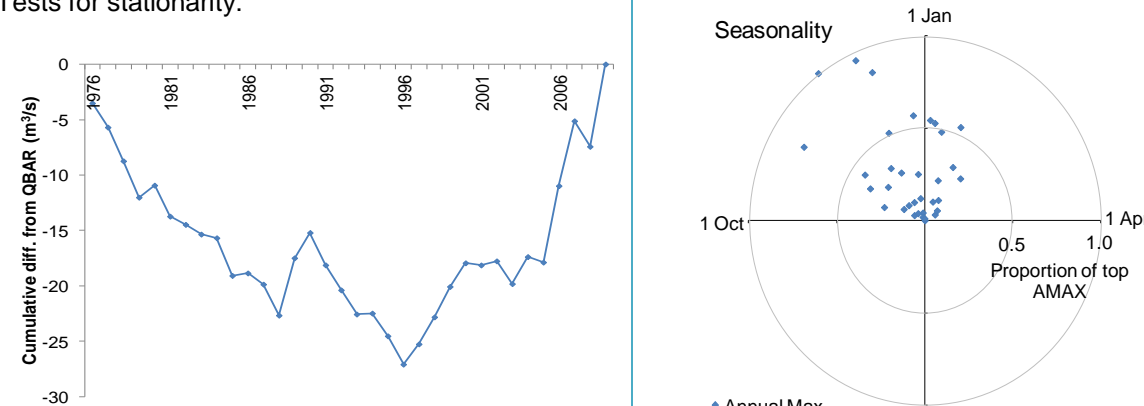
There is little difference between the growth curves, especially both single-site curves and pooled GL at lower return periods. Over the 10-year return period, the LN2 curve becomes shallower than the other three curves whilst the Gumbel and GL curves have a similar shape for the whole curve. The LN2 curve is likely flattened by the November 1980 event which has a similar growth factor to the December 1997 flood.

Recommended growth curve

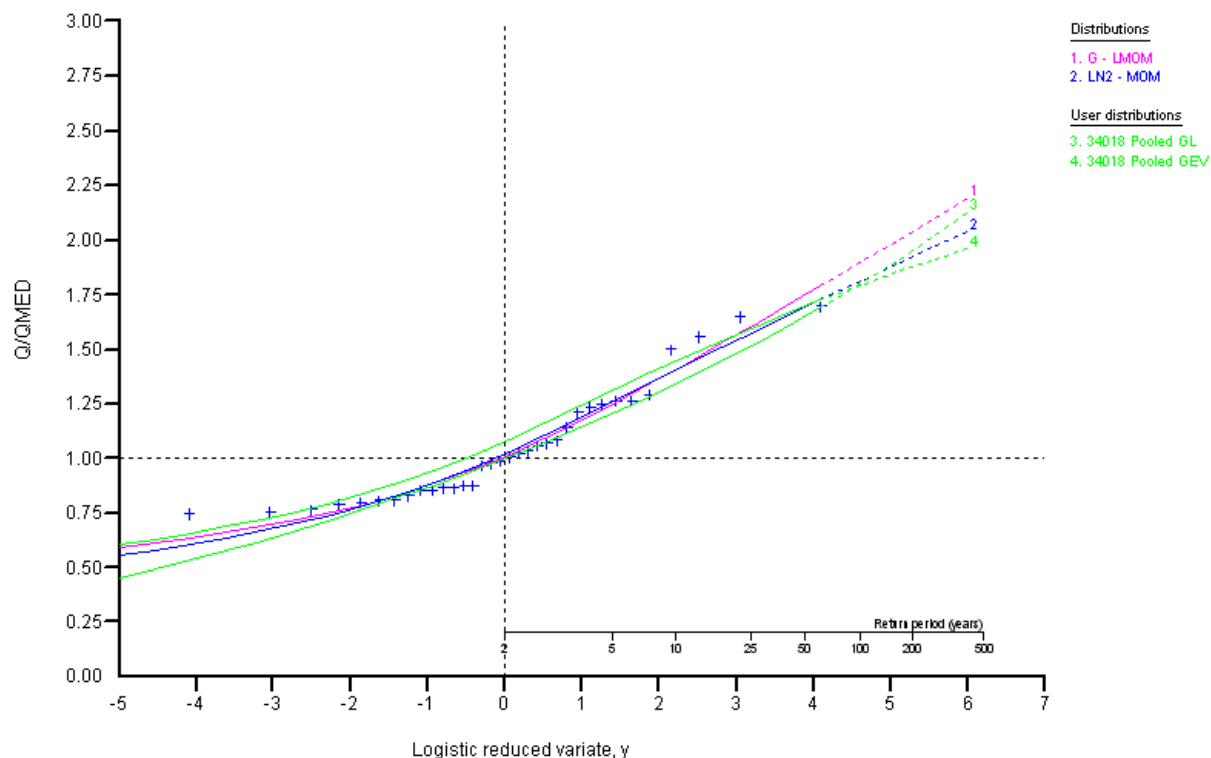
The single-site Gumbel curve is recommended as the preferred growth curve for design flood estimation as it fits the recorded flood peaks and there are 37 years of data.

Recommended design flows (Single-site Gumbel)								
AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m ³ /s)	227.8	273.0	303.1	331.7	368.9	396.9	461.1	488.8
Growth factor	0.986	1.182	1.312	1.436	1.597	1.718	1.996	2.116
Composition of pooling group <p>The stations in the pooling group have been selected as the most similar catchments in Ireland according to three descriptors at the subject site: AREA (935km²), SAAR (1292mm) and BFIsoil (0.485).</p> <p>The Ballylahan gauge does not appear in the pooling group as it is not classed as A1 or A2 in the FSU quality classification and its rating has not been reviewed within the CFRAM.</p>								
ID	Rank	Watercourse	Location		Years			
18003	1	Munster Blackwater	Killavullen		55			
201010	2	Mourne	Drumnabuoy House		27			
203093	3	Maine	Shane's Viaduct		26			
203020	4	Moyola	Moyola New Bridge		37			
201006	5	Drumragh	Campsie Bridge		37			
203011	6	Maine	Dromona		19			
24013	7	Deel	Rathkeale		41			
34010	8	Moy	Cloonacannana		11			
203012	9	Ballinderry	Ballinderry Bridge		40			
201005	10	Camowen	Camowen Terrace		37			
202002	11	Faughan	Drumahoe		33			
202001	12	Roe	Ardnargle		34			
236005	13	Colebrooke	Ballindarragh Bridge		27			
201008	14	Derg	Castlederg		34			
30012	15	Clare	Claregalway		15			
35005	16	Ballysadare	Ballysadare		62			

Flood frequency analysis summary sheet

Station 34018		Castlebar @ Turlough	
			
Top ranking floods:			QMED (m³/s): 11.5
Rank	Date	Flow (m³/s)	AEP (%) from single-site analysis
1	22 November 2009	19.6	2.54
2	05 December 2006	19.0	3.25
3	09 December 2007	18.0	4.87
Tests for stationarity:			
Mann-Kendall test: significant increasing trend		<p>All supplied AMAX events have occurred between October and April but there appears to be a bias for the largest events to occur in the Autumn. The largest flood on record has a growth factor of 1.7. The plot of cumulative deviation from QBAR at this site indicates that a change may have occurred around 1996. Prior to this date the majority of floods in the series were lower than the mean whereas after this date the majority were large. A similar change can be seen on the neighbouring catchment, the River Manulla at Gneeve Bridge. This similarity suggests that the change is likely to be genuine rather than an artefact of the way flows are measured. It may be a climatic effect or else could be due to a change within the catchments such as river maintenance work. It is also possible that this is the result of a long term trend of increasing peak flows, which was described as strongly significant in the report on FSU WP2.2.</p>	
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)	0.933	0.209	n/a	1.896
Single-site LN2 (moments)	0.022	0.246	n/a	1.811
Pooled GL (L-moments)	1	0.1375	-0.0973	1.800
Pooled GEV (L-moments)	1	0.2227	0.1170	1.710

Comments on growth curves

There is little difference between the single-site and pooled growth curves. The single-site Gumbel curve is steeper than the single-site LN2 curve and the two pooled curves. If the top four ranking floods were excluded, the single-site curves are likely to have a similar gradient to the pooled curves. All four curves fit the 33-year recorded AMAX series reasonably closely.

Recommended growth curve

The pooled GL curve is recommended as the preferred growth curve for design flood estimation. The single-site Gumbel curve fits the recorded AMAX series and the length of the gauge record is an appropriate length for estimating flows for short to moderate AEPs. However, it does not provide enough data to be confident in the design flows for longer AEPs, making the pooled analysis more appropriate.

Recommended design flows (Pooled GL)

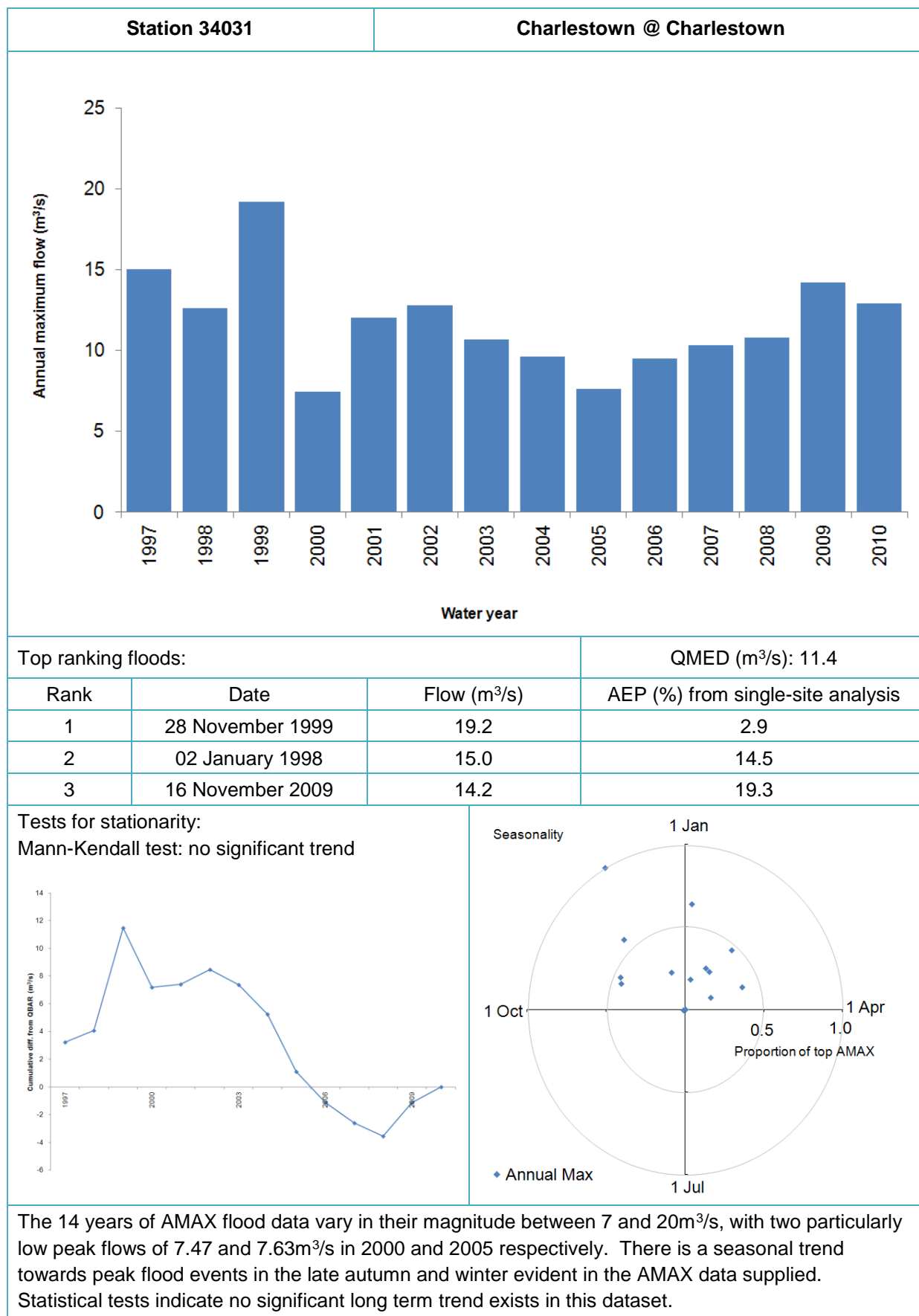
AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m ³ /s)	11.5	13.8	15.4	16.9	19.0	20.7	22.4	27.0
Growth factor	1.00	1.20	1.34	1.47	1.65	1.80	1.95	2.35

Composition of pooling group

The stations in the pooling group have been selected as the most similar catchments in Ireland according to three descriptors at the subject site: AREA (95.4km²), SAAR (1554mm) and BFIsoil (0.75).

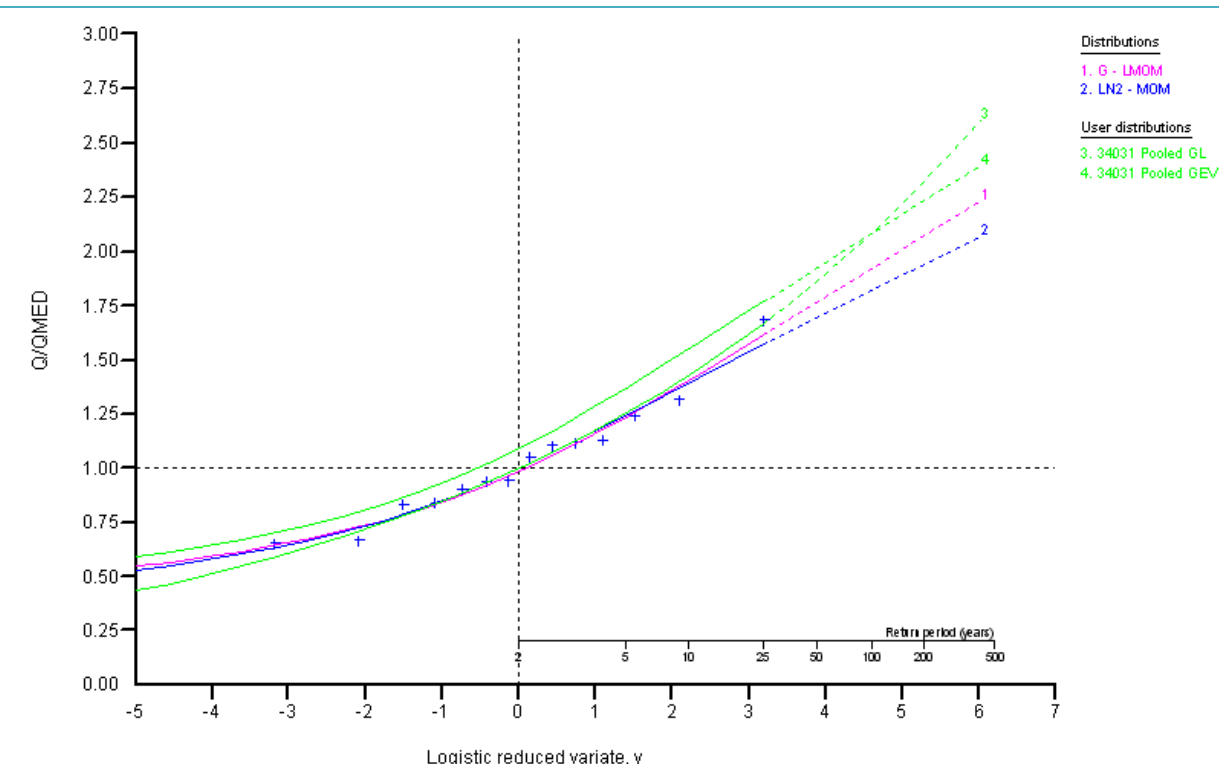
ID	Rank	Watercourse	Location	Years
34018	1	Castlebar	Turlough	34
27070	2	L. Inchiquin	Baunkyle	29
27003	3	Fergus	Corrofin	48
35073	4	Lough Gill	Lough Gill	30
32012	5	Newport	Newport Weir	31
35012	6	Garvogue	New Bridge	10
35071	7	L. Melvin	Lareen	35
19020	8	Owennacurra	Ballyedmond	28
35028	9	Bonet	New Bridge (Manorhamilton)	20
35003	10	Unshin	Ballygrania	45
27002	11	Fergus	Ballycorey	56
33070	12	Carrowmore L.	Carrowmore	28
25044	13	Kilmastulla	Coole	40
29004	14	Clarinbridge	Clarinbridge	37
29071	15	L. Cutra	Cutra	36

Flood frequency analysis summary sheet



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)	0.905	0.221	n/a	1.922
Single-site LN2 (moments)	-0.002	0.258	n/a	1.82
Pooled GL (L-moments)	1	0.1615	-0.1543	2.080
Pooled GEV (L-moments)	1	0.2492	0.0245	1.990

Comments on growth curves

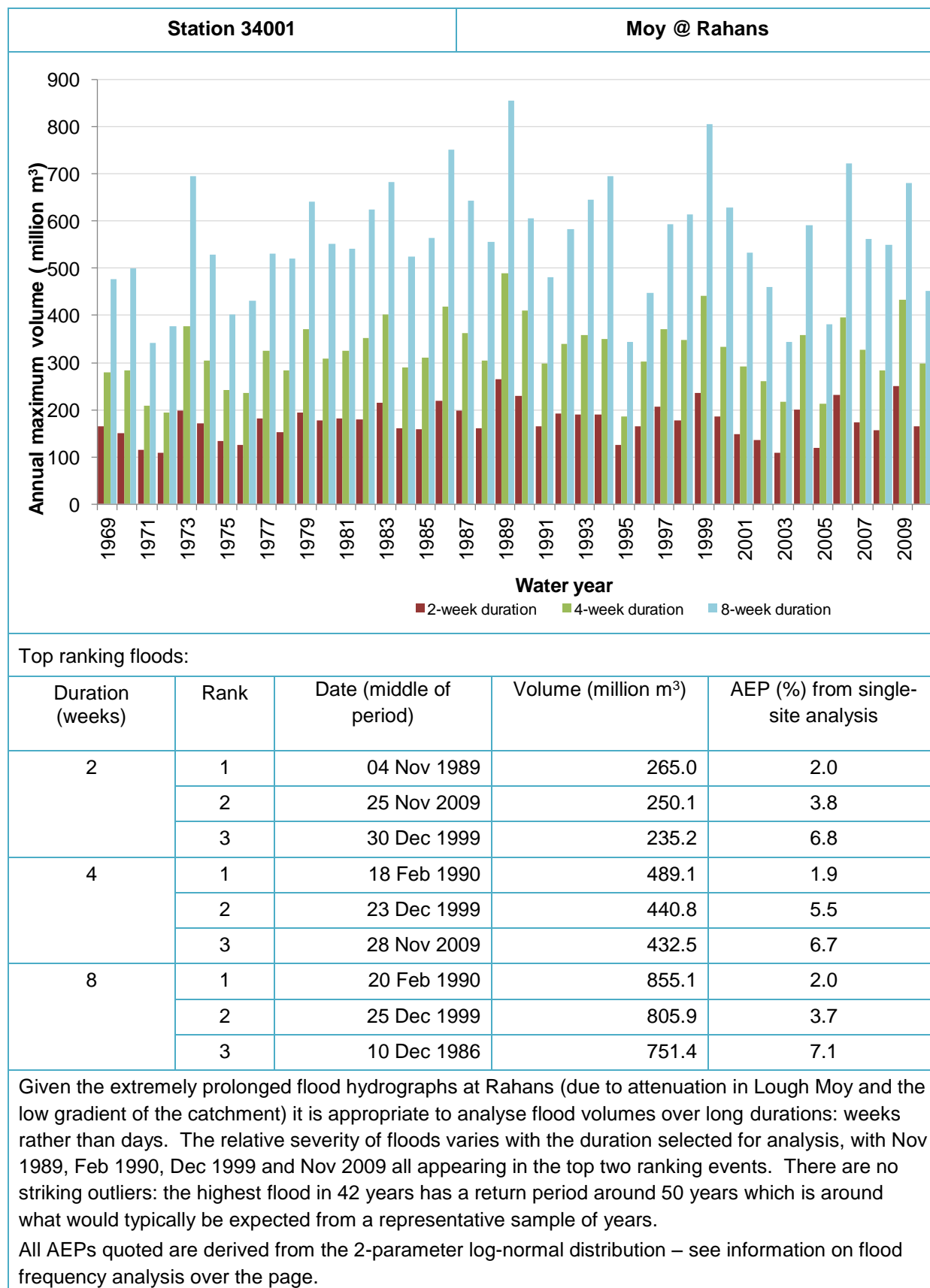
There is little difference between the single-site curves, although at higher return periods, the Gumbel growth curve steepens. The pooled growth curves are slightly steeper than the single-site curves. The AMAX series appears to fit the pooled GL curve the best.

Recommended growth curve

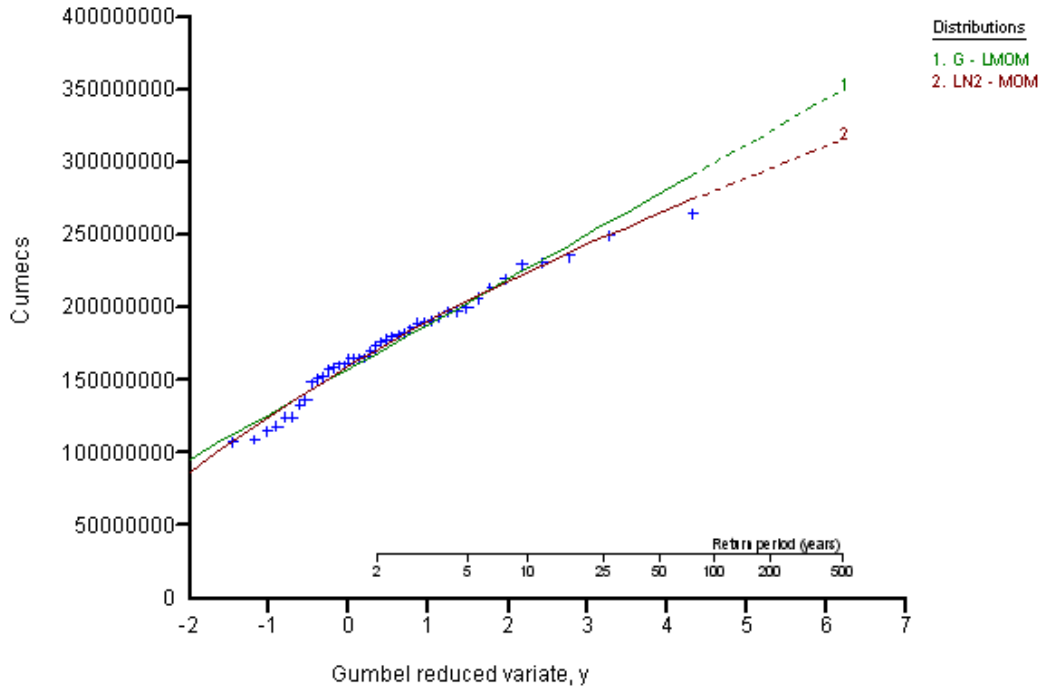
The pooled GL curve is recommended as the preferred growth curve for design flood estimation as it fits the recorded AMAX series well. Although the single-site Gumbel curve fits the recorded peak flows well, the record length is relatively short (14 years), meaning there may be more confidence in the pooled data.

Recommended design flows (Pooled GL)								
AEPs	50%	20%	10%	5%	2%	1%	0.5%	0.1%
Flow (m ³ /s)	11.4	14.3	16.2	18.2	21.2	23.7	26.4	34.1
Growth factor	1.00	1.25	1.42	1.60	1.86	2.08	2.32	2.99
Composition of pooling group <p>The stations in the pooling group have been selected as the most similar catchments in Ireland according to three descriptors at the subject site: AREA (21.1km²), SAAR (1275mm) and BFIsoil (0.33).</p> <p>The Charlestown gauge does not appear in the pooling group as it is not classed as A1 or A2 in the FSU quality classification and its rating has not been reviewed within the CFRAM. The small size of the catchment results in the inclusion of many gauges from Northern Ireland where there is a larger number of gauged small catchments.</p>								
ID	Rank	Watercourse	Location	Years				
203049	1	Clady	Clady Bridge	27				
203042	2	Crumlin	Cidercourt Bridge	30				
32011	3	Bunowen	Louisburgh Weir	27				
206006	4	Annalong	Recorder 1895	48				
23012	5	Lee	Ballymullen	18				
203028	6	Agivey	Whitehill	37				
203046	7	Rathmore Burn	Rathmore Bridge	27				
205101	8	Blackstaff	Eason's	22				
205008	9	Lagan	Drumiller	35				
203039	10	Clogh	Tullynewey	28				
23001	11	Galey	Inch Bridge	38				
203043	12	Oonawater	Shanmoy	23				
201002	13	Fairywater	Dudgeon Bridge	38				
34009	14	Owengarve	Curraughbonaun	29				
203024	15	Cusher	Gamble's Bridge	38				
205005	16	Ravernet	Ravernet	37				

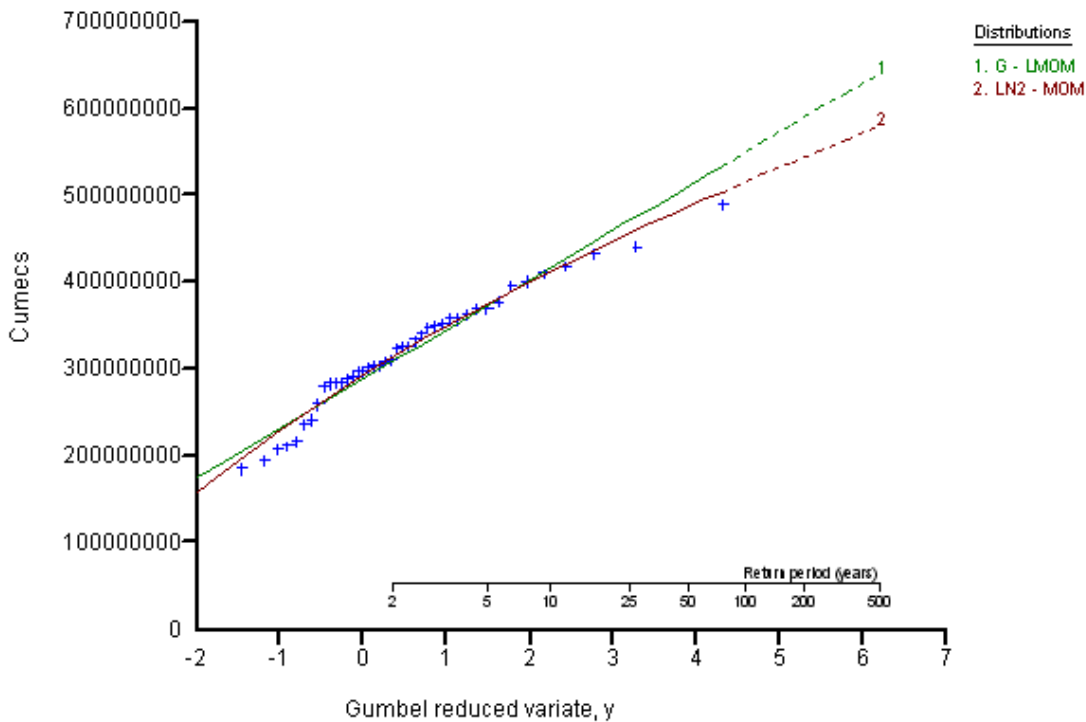
Flood volume analysis summary sheet



Flood frequency analysis: 2-week volumes (m³)



Flood frequency analysis: 4-week volumes (m³)



Flood frequency analysis: 8-week volumes (m³)

Graph unable to plot due to magnitude of volumes.

The Gumbel (G) distribution has been fitted using L-moments and the 2-parameter log-normal (LN2) distribution using moments. The LN2 distribution appears to give a good fit and has been used to estimate the AEPs for the top three floods.

C Historical flood chronology

C. Flood chronology

This appendix provides results from analysis of flood history for UoM34. 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 Reports¹.

The table below gives a chronology of flood events, including information on their impacts. All information on floods up to 1954 was obtained from the Irish Independent unless otherwise stated.

Date	Catchment/ river	Details
11 September 1908	Ballina	Extensive flooding of land
	Pontoon	Bridge at Pontoon [on the channel between Lough Conn and Lough Cullin] was swept away (this must be at least similar flooding to that in 1932)
4 September 1910	Montiagh	2 square miles up to 7 feet deep lake around the village as a result of flood. Heavy rainfall across the County, pluvial flooding. (Sunday Independent)
18 January 1932	North Mayo	Flooding worst in living memory.
	Foxford	Flooding created a lake with 1 mile diameter around Foxford.
	Ballylahan	Water was 4 feet on the road.
	Pontoon	The road suffered 2 feet of water [between Lough Conn and Lough Cullin].
20 January 1932	Ballina	Flooding up to bridge soffits.
11 December 1947	Ballina	Heavy rainfall in past few weeks led to flooding on south side of Ballina.
		River Moy burst its banks, highest flood in 4 years.
13 December 1948	Co. Mayo	Torrential rain followed by heavy flooding in Co Mayo.
	Ballina	Streets under 2 feet of water.
19 October 1954	Co. Mayo	Widespread flooding.
	Castlebar	Streets flooded to 1 foot.

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

Date	Catchment/ river	Details
1968	Ballina	Several houses and fire station flooded. This flooding presumably occurred before September 1968 and thus is not included in the annual maximum flow series for the gauge at Rahans, which starts in the water year 1968-69.
	Castlebar	Historical flooding noted.
1989	Crossmolina	Storms and floods engulfed the north and west Mayo regions in what were described as “the worst floods since 1969” [presumably 1968] (Mayo News). Extensive town flooding from the River Deel in Oct 1989, roads and properties were flooded.
	Ballina	River Moy flooded, boats used in a “dramatic rescue operation” (Mayo News).
December 1999	Swinford	Historical flooding associated with the unnamed tributary occurred along Park Road and Riverside due to a blocked culvert. Flood depth of 1m was reported.
2005	Foxford	Road and Land flooding in the callows near Derrygaury south of Foxford from the River Moy.
5 th December 2006	Crossmolina	A large section of the town was covered by 3 feet of water from the River Deel. Chapel and Church Streets were the first affected when the River Deel burst its banks. Contents of a hardware shop and the FAB Carpet and Furniture Store were damaged. The flood was described as not as severe as that of 1989 (Mayo News). No severe flooding between 1989 and 2006.
2 nd July 2009	Castlebar	Extensive flooding in Mayo after several hours of torrential rain. Worst hit areas were the Castlebar-Westport Road, Castlebar-Glenisland Road, Castlebar-Newport Road. Several houses and fire station flooded.
November 2009	Castlebar	Highest flow on record (1976 to date) for the Castlebar River at Turlough. No reports of flood damage found.
January 2014	Ballina	Flooding from high tide and storm surge affecting Clare Street, Bachelor's Walk areas.
November 2015	Crossmolina	Flooding of the town centre and Chapel Road from river and surface water.
December 2015	Foxford	Flooding following Storm Desmond. Evacuation of nursing home in Foxford, with B&B and GAA grounds also affected in Foxford.
	Ballina	Flooding of the N59 at Cathedral Road and Bachelor's Walk but no property flooding as water levels responded to storm event during preceding days.
	Crossmolina	Severe flooding of the town in response to intense storm rainfall.

Based on the outcomes of the analysis, a flood history time line was produced. The time line provides an 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 34

Artificial influence:

Drainage

Flood events:

Legend

Source of information

- History review
- Hydrometric data

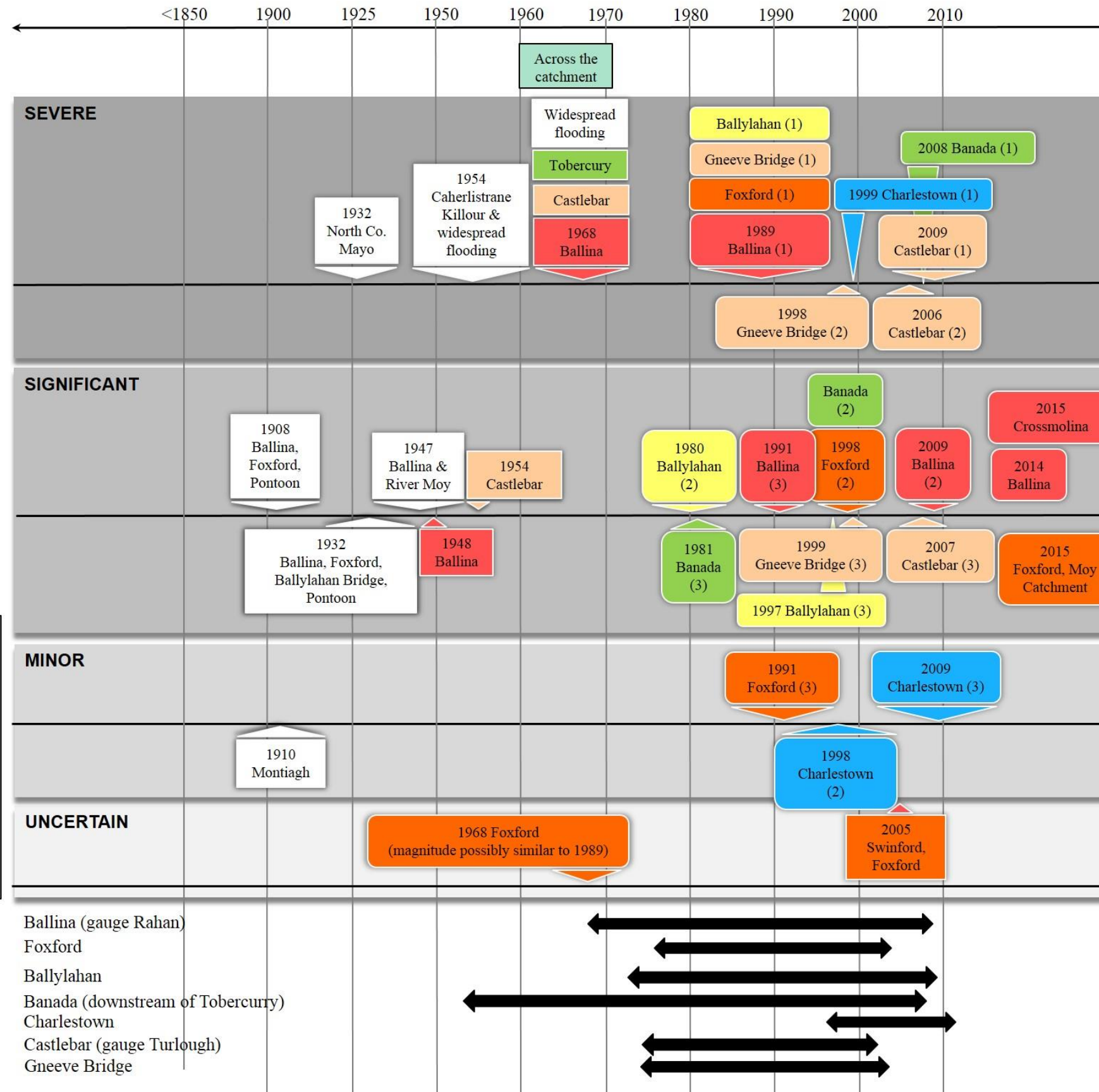
Spatial distribution of the locations

Downstream → Upstream

..... Widespread flooding

(1), (2), (3) Rank based on hydrometric data only

Available periods of hydrometric data:

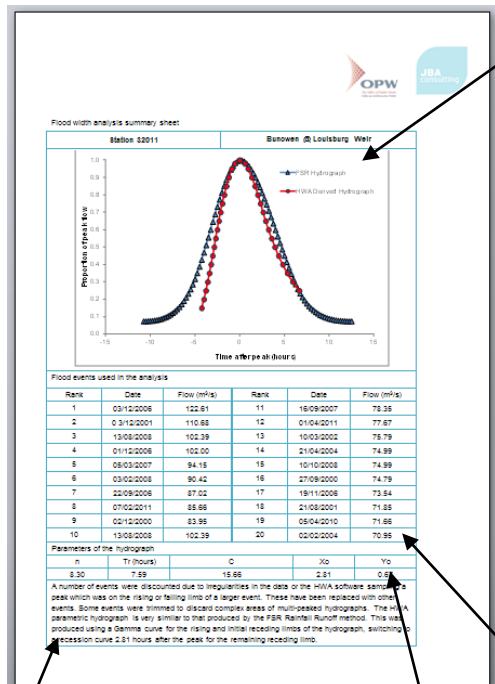


D Hydrograph width analysis

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 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, something which will be considered in the next stage of the study.

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

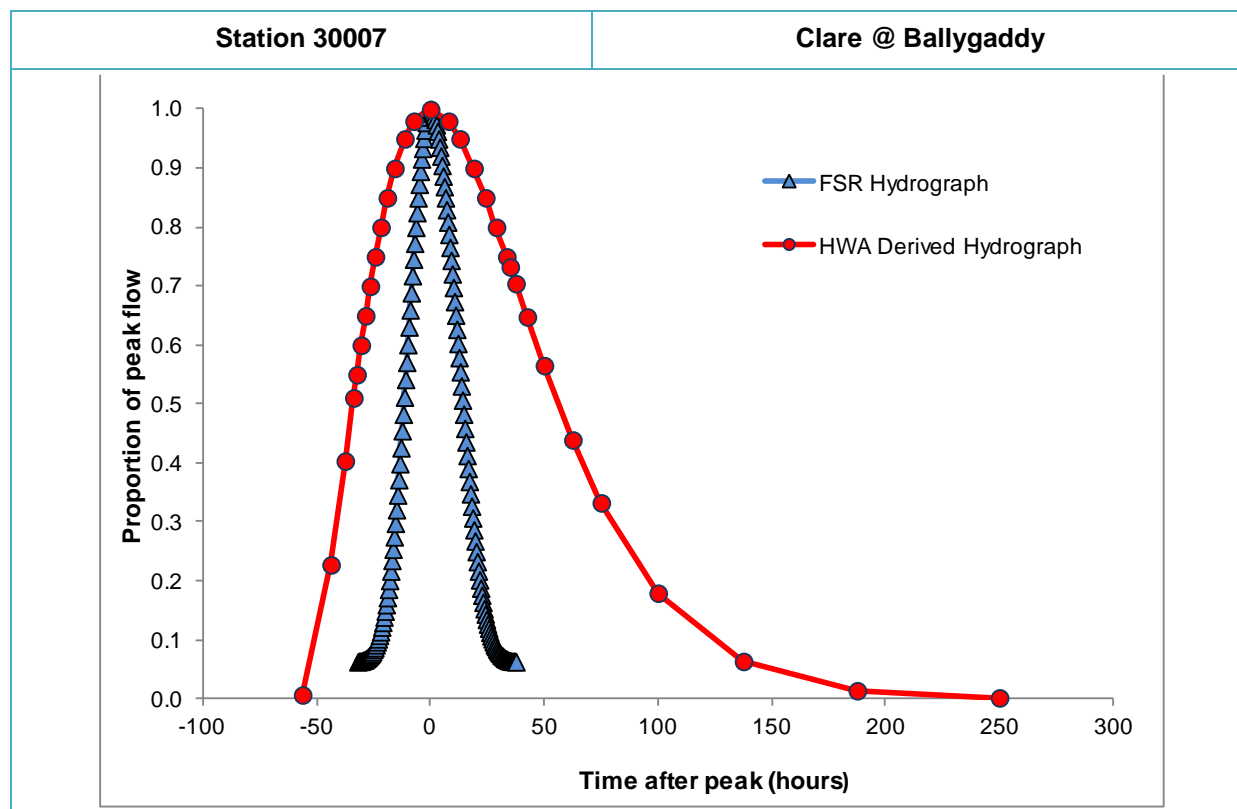
C: Parameter of the exponential function which is used to describe the recession part of the flood hydrograph

X₀, Y₀: Co-ordinates for the transition between the gamma and exponential functions. X₀ is the time after the peak (in hours) and Y₀ is the normalised flow at this time.

Commentary

Notes on the analysis.

Flood width analysis summary sheet



Flood events used in the analysis

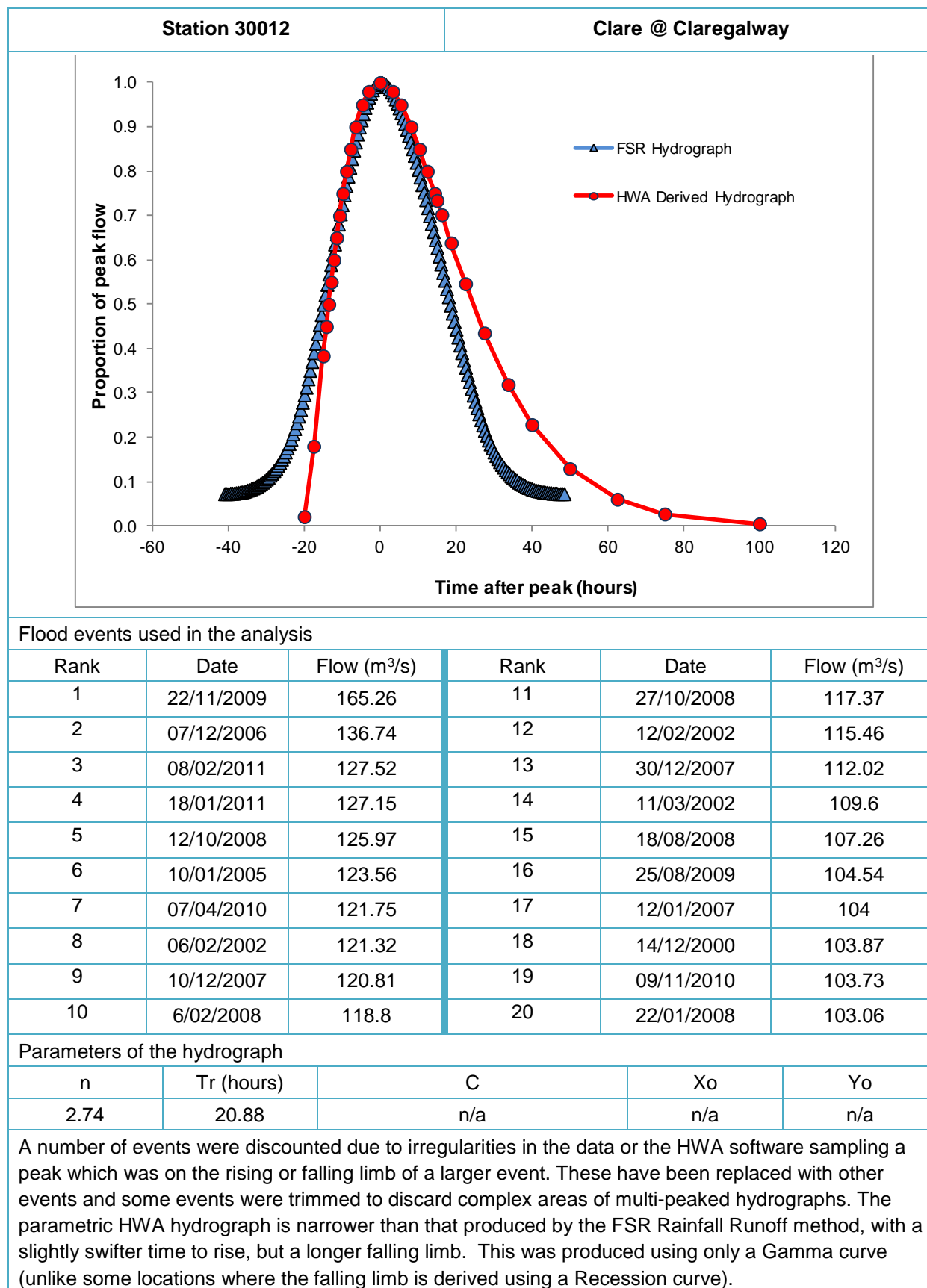
Rank	Date	Flow (m ³ /s)	Rank	Date	Flow (m ³ /s)
1	21/11/2009	108.81	11	08/11/1977	69.55
2	30/11/1999	93.38	12	03/12/1992	67.97
3	30/10/1989	92.08	13	27/10/1995	67.34
4	07/02/1990	89.41	14	20/12/1982	66.66
5	05/12/2006	85.11	15	2/01/1991	66.53
6	03/11/1980	80.88	16	11/03/2002	66.08
7	09/01/1992	74.98	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

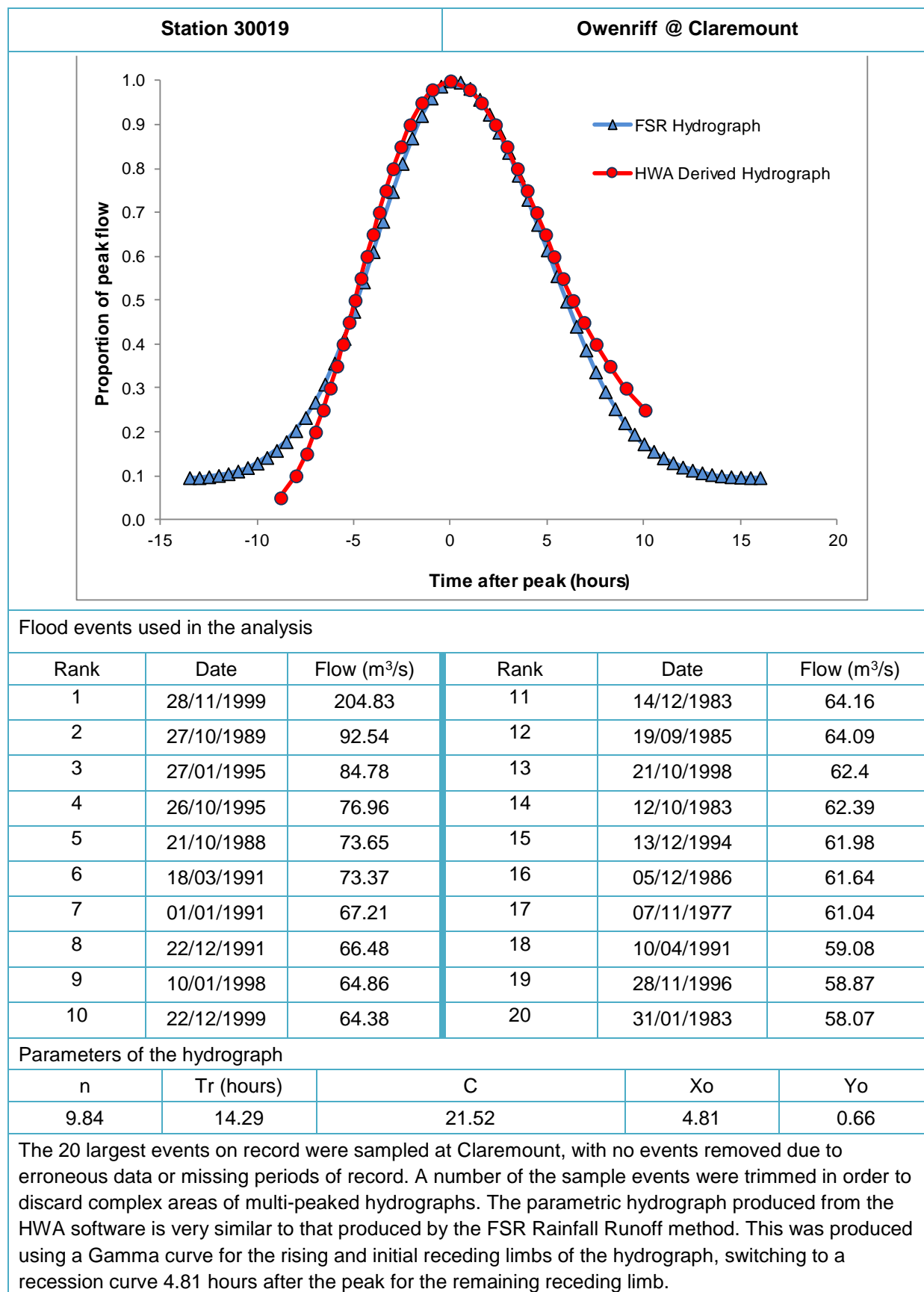
n	Tr (hours)	C	Xo	Yo
3.458	59.25	n/a	n/a	n/a

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).

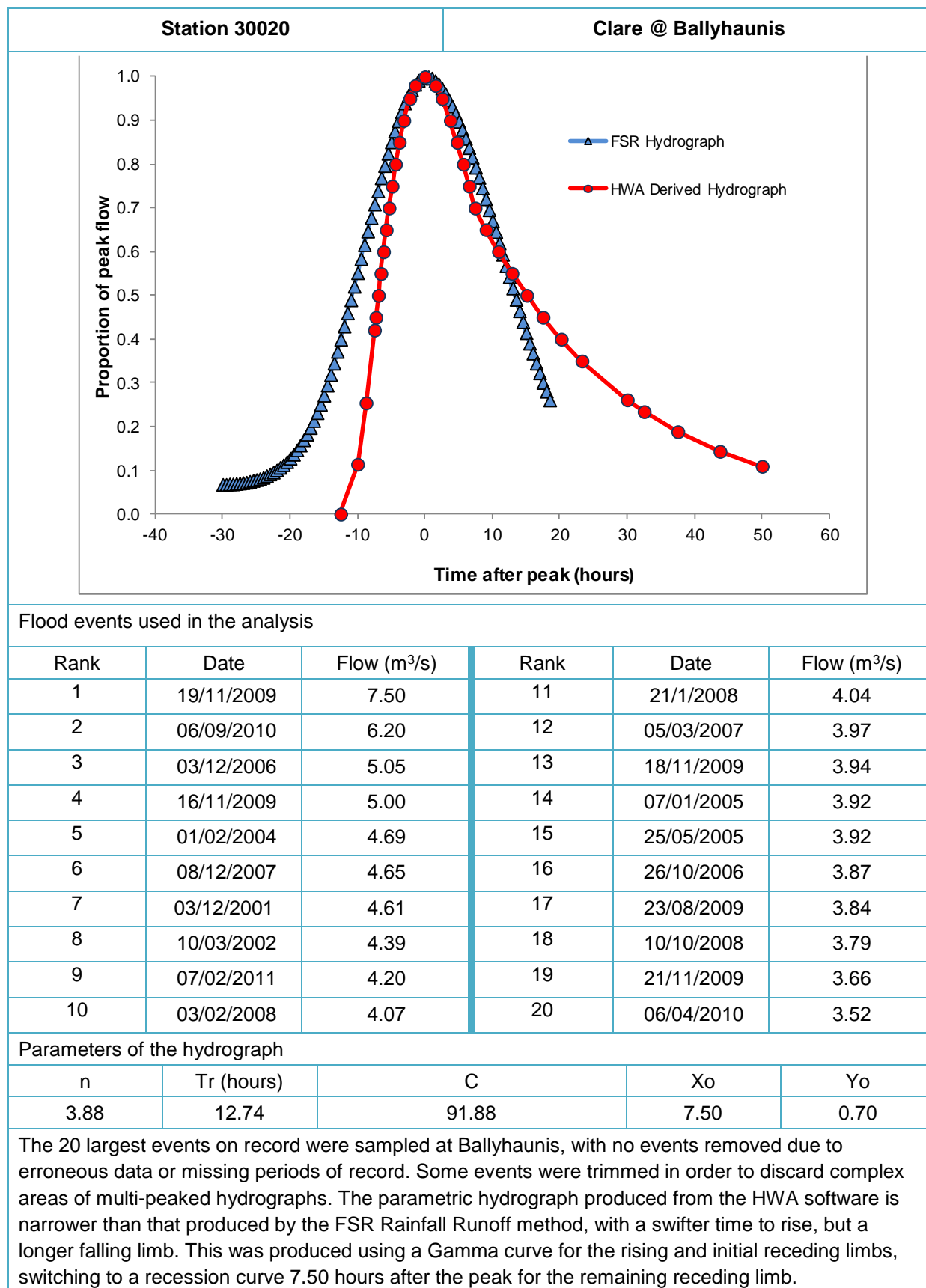
Flood width analysis summary sheet



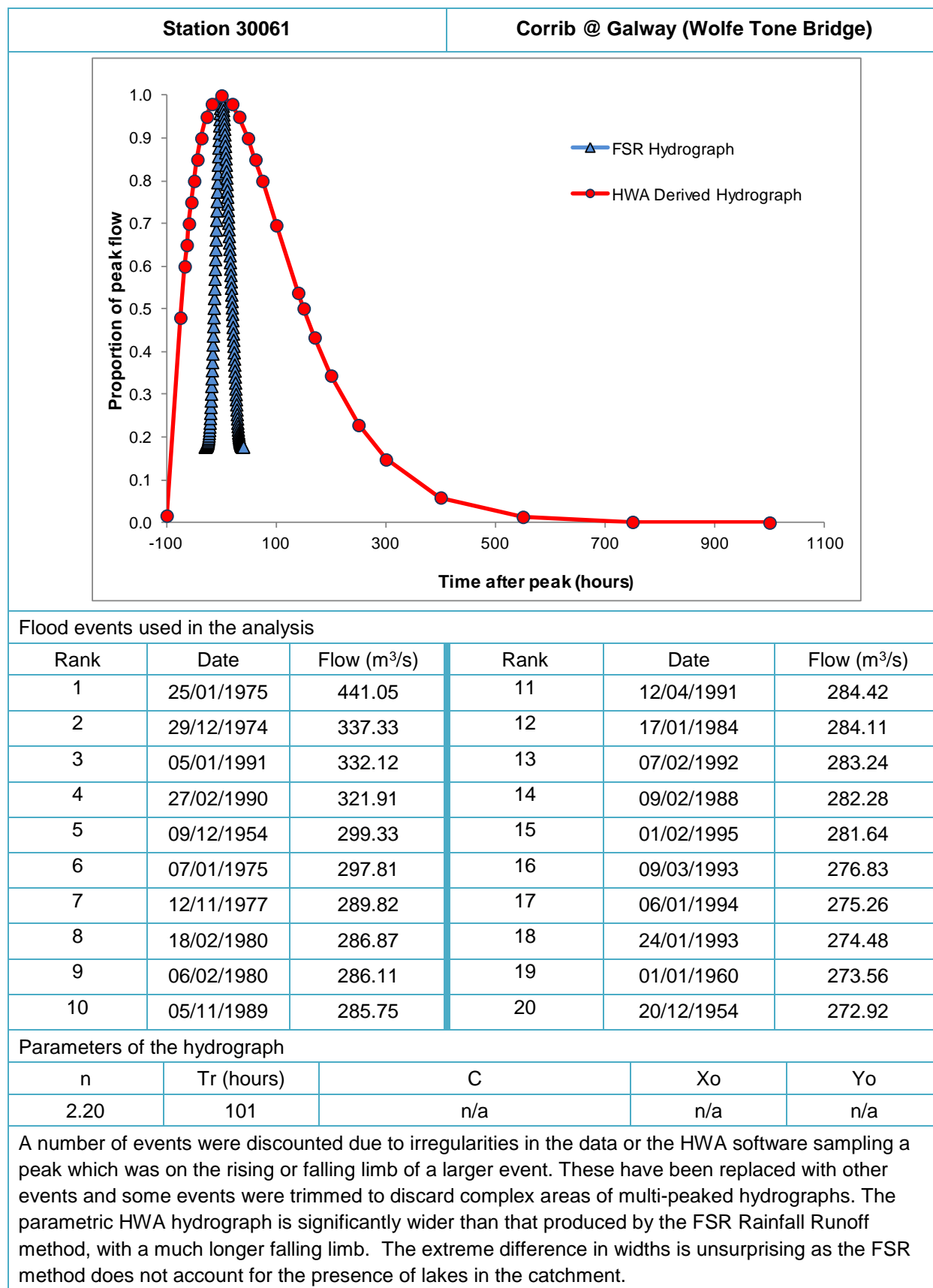
Flood width analysis summary sheet



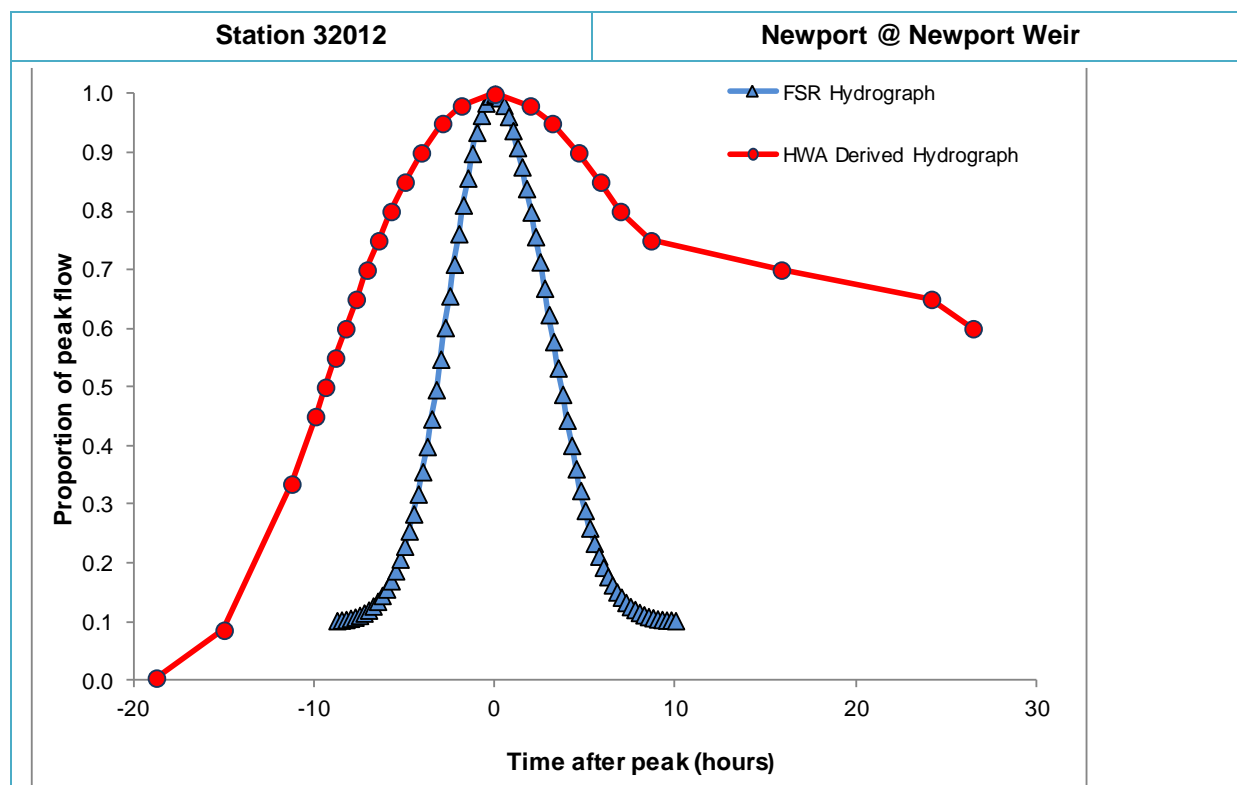
Flood width analysis summary sheet



Flood width analysis summary sheet



Flood width analysis summary sheet



Flood events used in the analysis

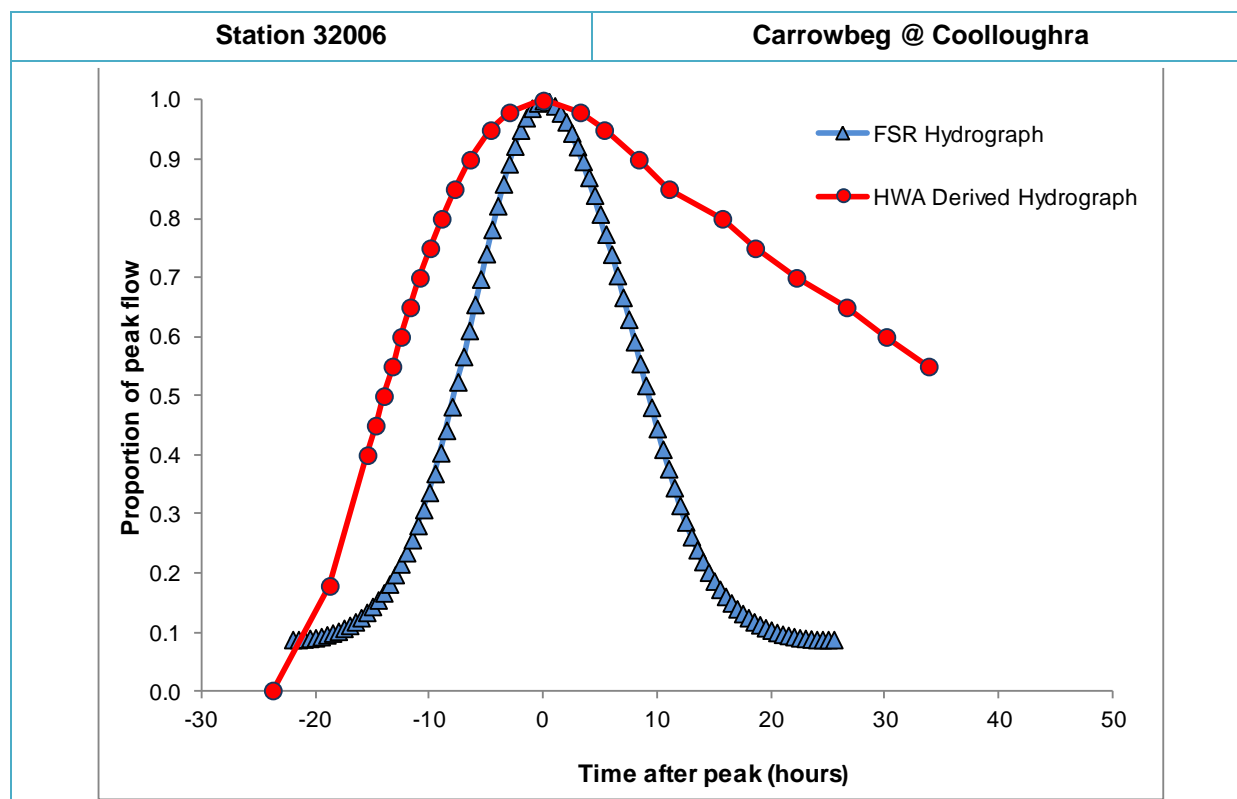
Rank	Date	Flow (m ³ /s)	Rank	Date	Flow (m ³ /s)
1	03/12/2006	37.60	11	03/07/2009	31.76
2	07/02/2011	36.30	12	08/11/2010	30.58
3	08/12/2007	36.04	13	05/12/2001	30.12
4	04/11/2010	35.72	14	22/12/2004	29.71
5	13/08/2008	35.66	15	11/12/2006	29.60
6	10/10/2008	35.34	16	27/10/2000	29.37
7	15/01/2005	34.02	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



Flood events used in the analysis

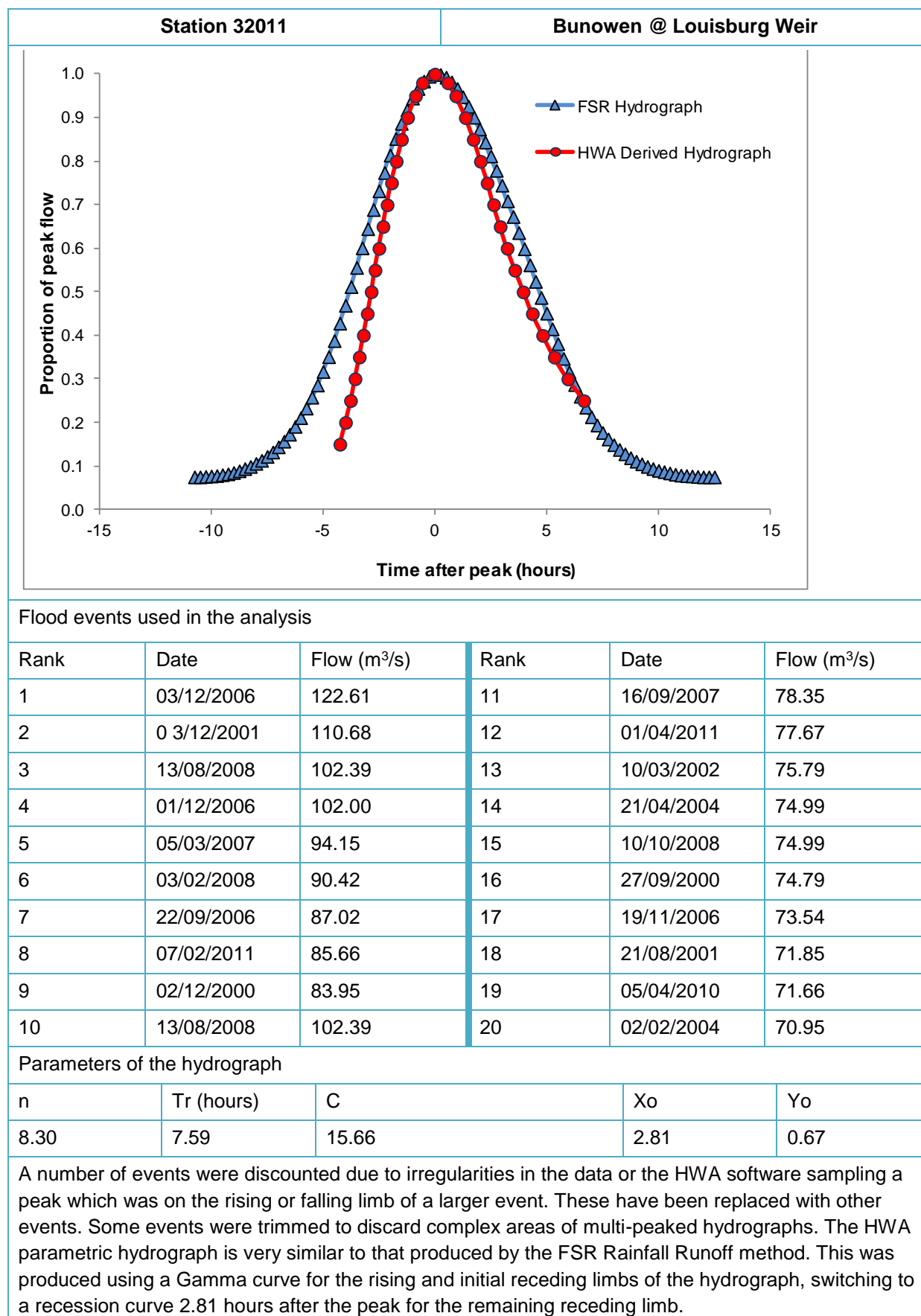
Rank	Date	Flow (m ³ /s)	Rank	Date	Flow (m ³ /s)
1	19/11/2009	15.54	11	06/03/2007	9.35
2	03/12/2006	13.53	12	30/11/2006	9.26
3	24/11/2009	12.52	13	20/11/2006	9.07
4	16/08/2008	11.89	14	01/11/2009	8.59
5	14/12/2006	10.45	15	04/11/2009	8.50
6	22/11/2009	10.20	16	02/12/2007	8.20
7	09/12/2007	10.12	17	18/01/2009	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

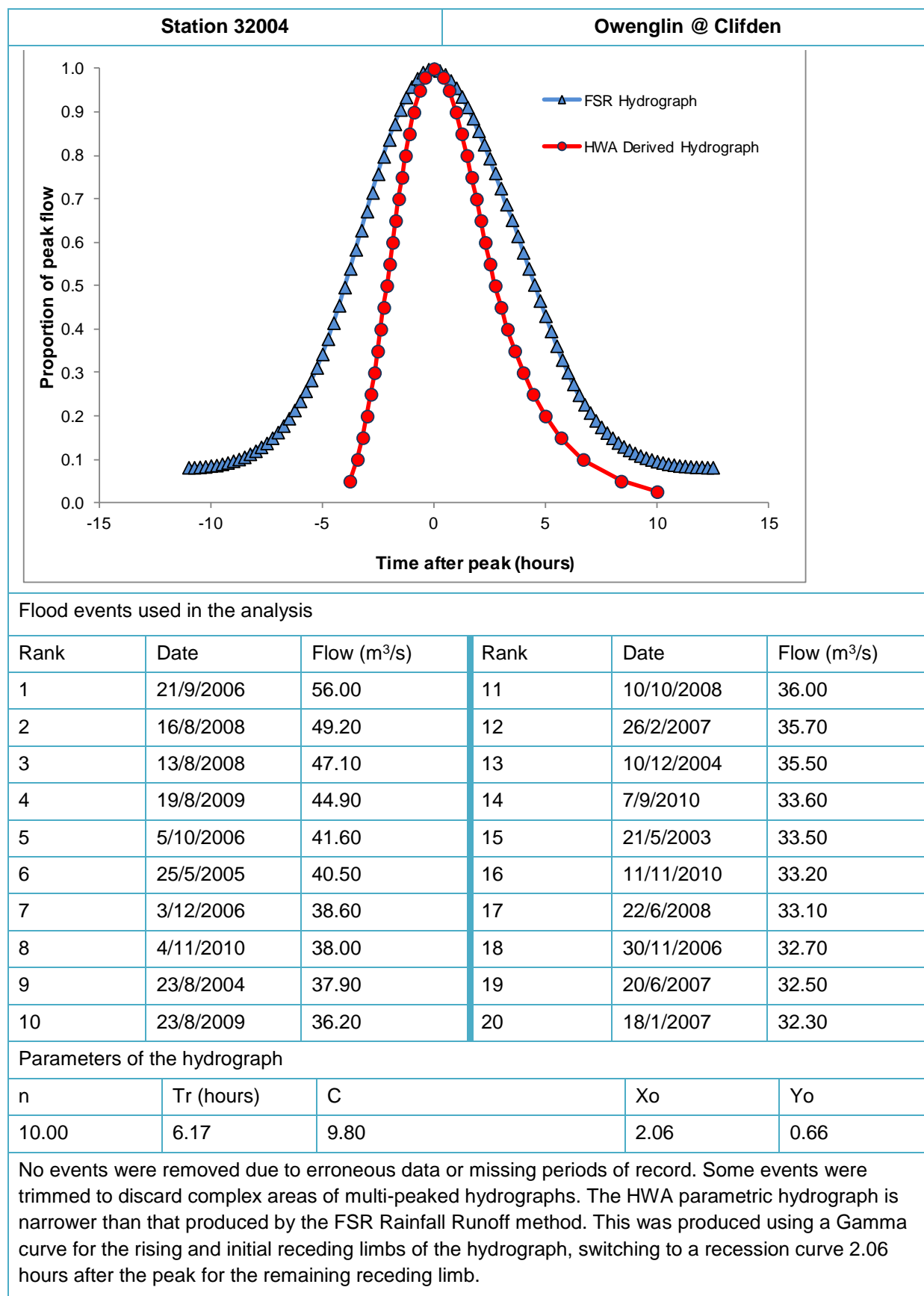
n	Tr (hours)	C	Xo	Yo
3.52	98.02	n/a	n/a	n/a

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).

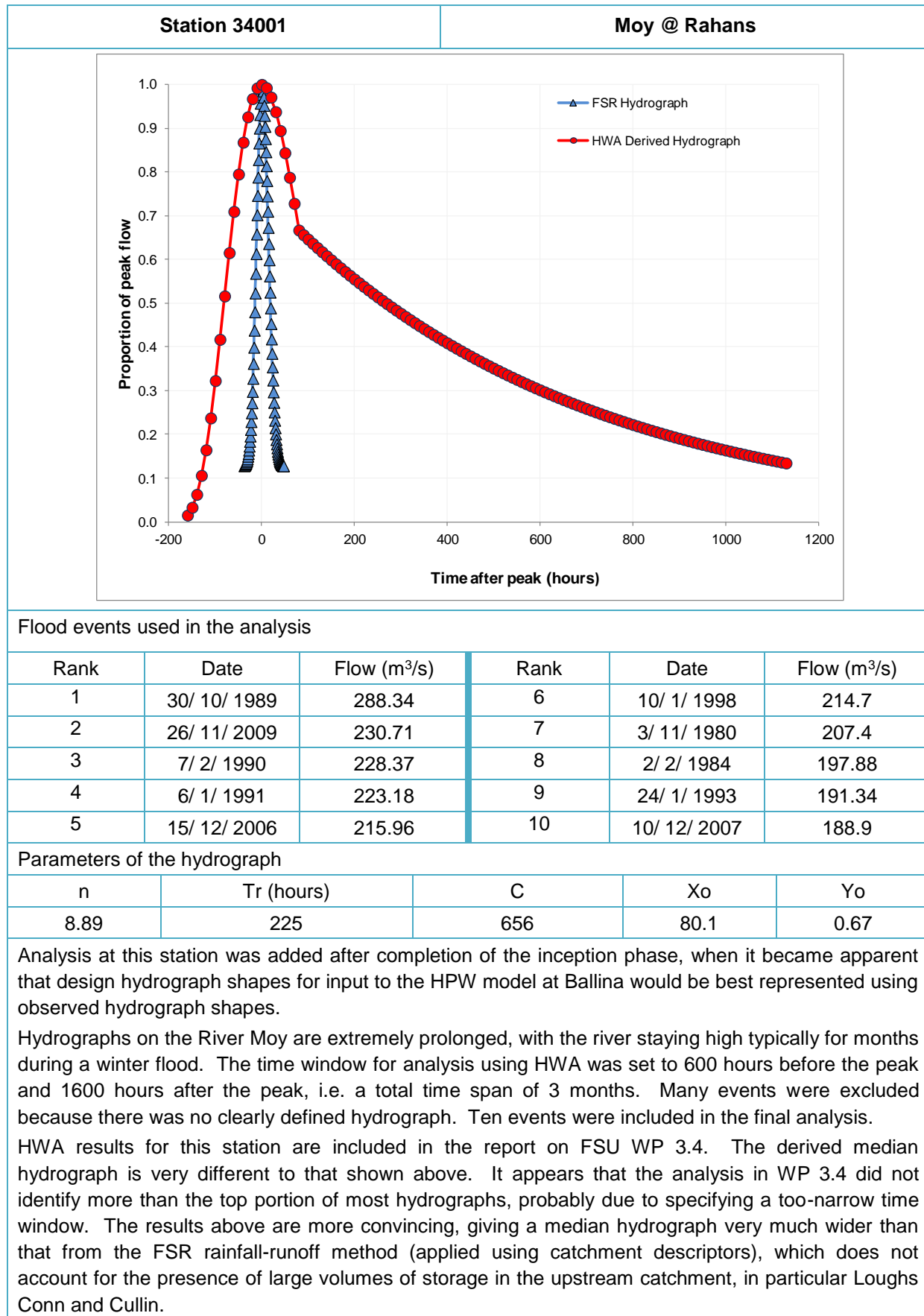
Flood width analysis summary sheet



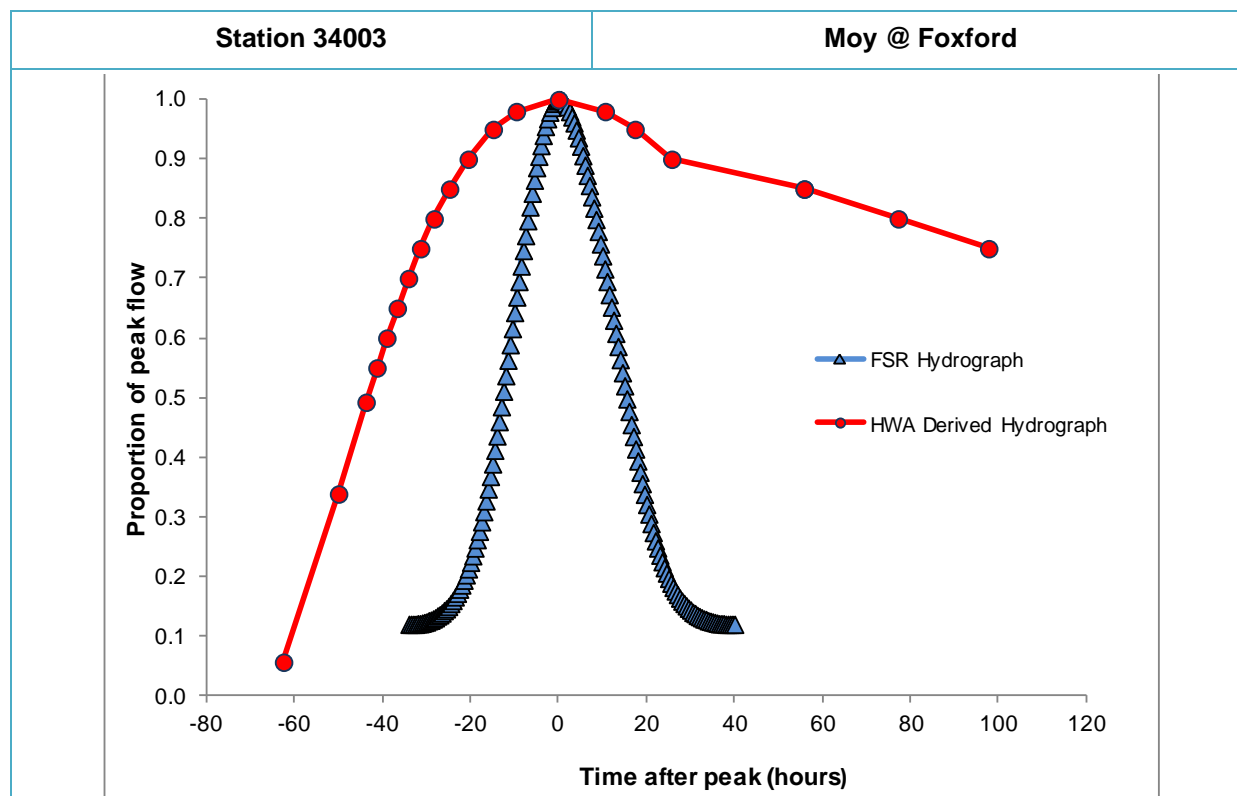
Flood width analysis summary sheet



Flood width analysis summary sheet



Flood width analysis summary sheet



Flood events used in the analysis

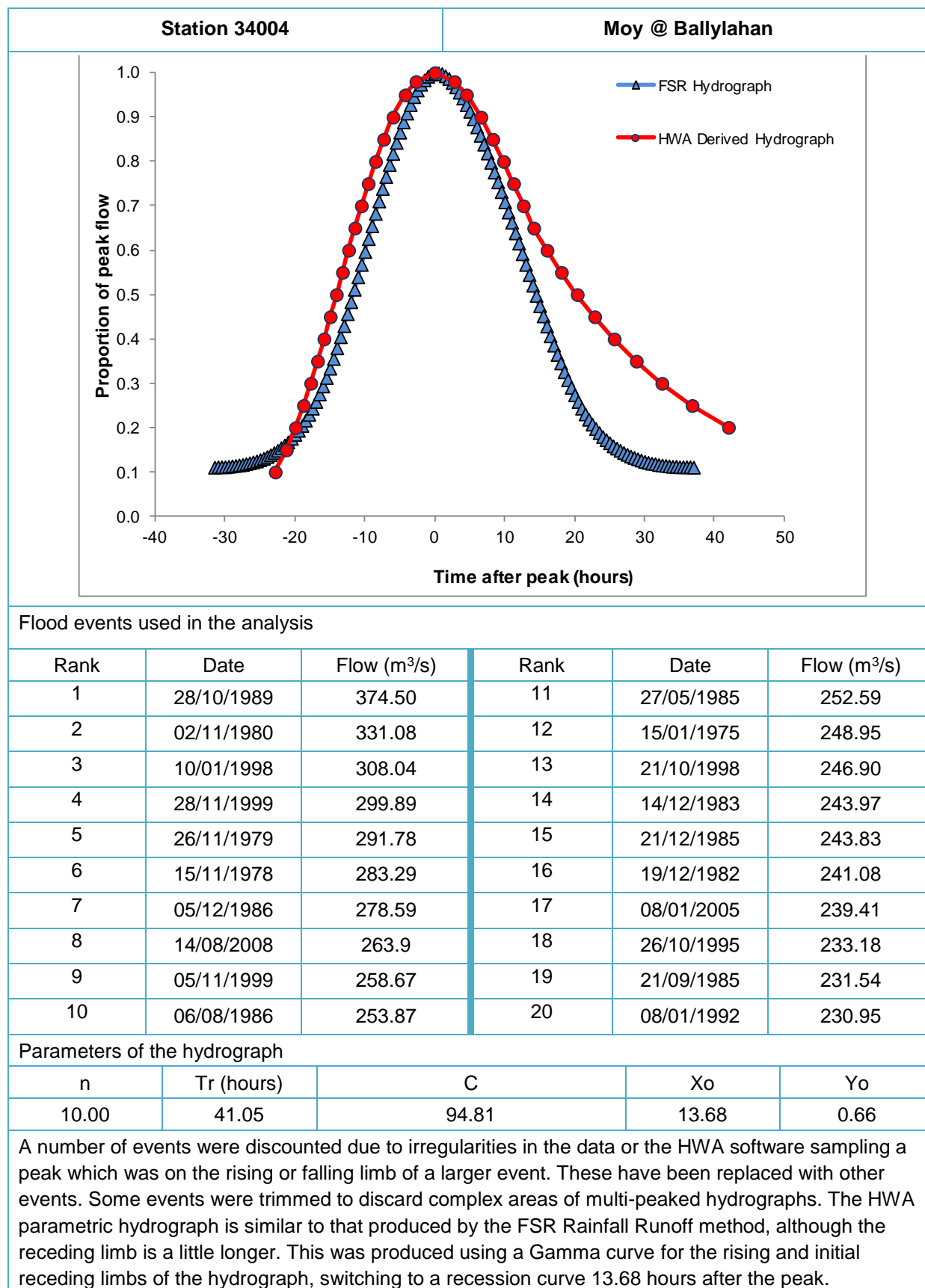
Rank	Date	Flow (m ³ /s)	Rank	Date	Flow (m ³ /s)
1	24/11/2009	259.23	11	17/11/2009	179.40
2	14/12/2006	243.00	12	11/01/2007	174.64
3	20/11/2009	231.33	13	07/02/2011	174.64
4	11/12/2006	223.43	14	16/01/2005	171.83
5	10/12/2007	195.01	15	07/12/2009	171.83
6	02/12/2009	189.89	16	18/01/2007	171.69
7	04/12/2006	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

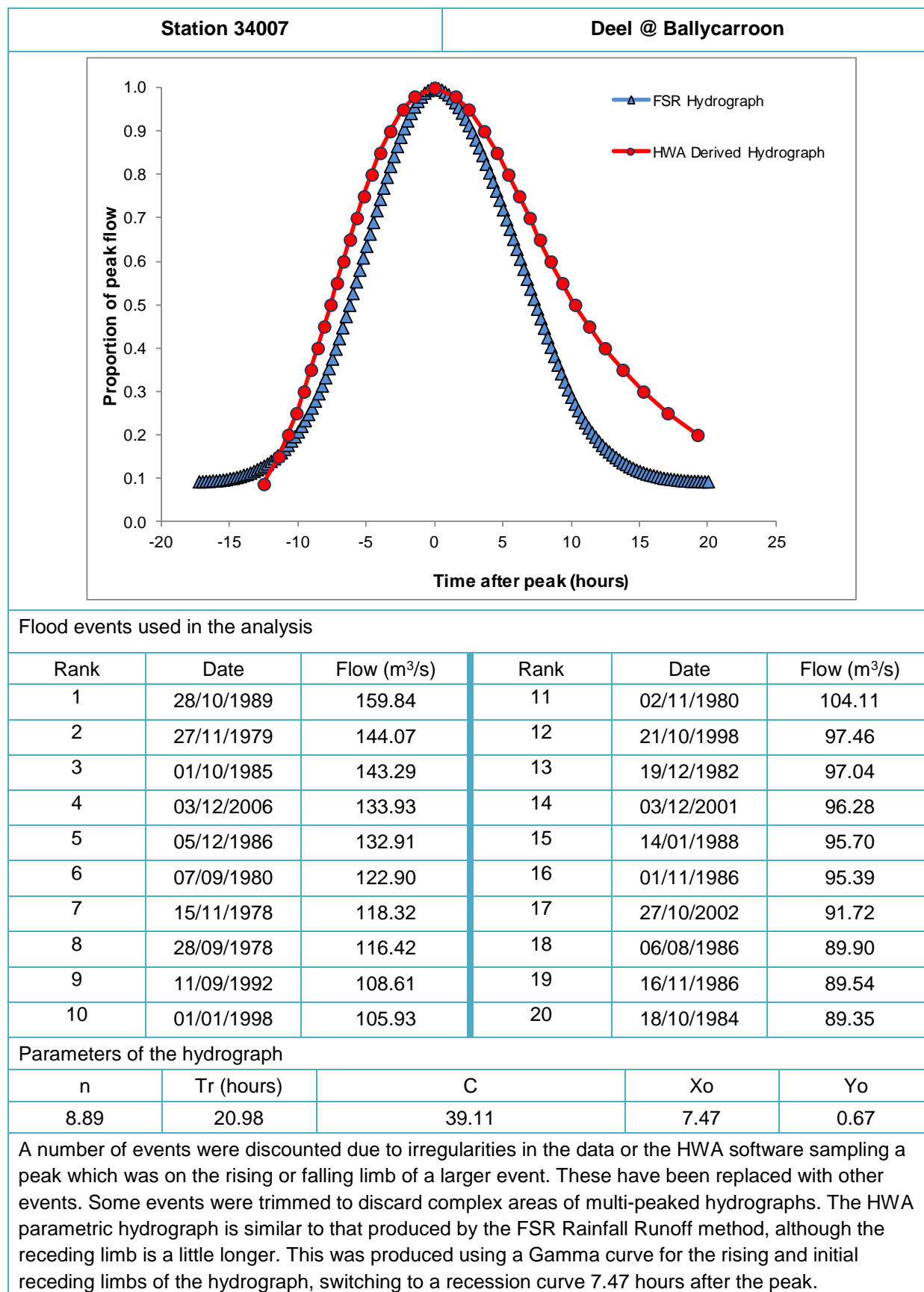
n	Tr (hours)	C	Xo	Yo
2.85	68.27	n/a	n/a	n/a

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).

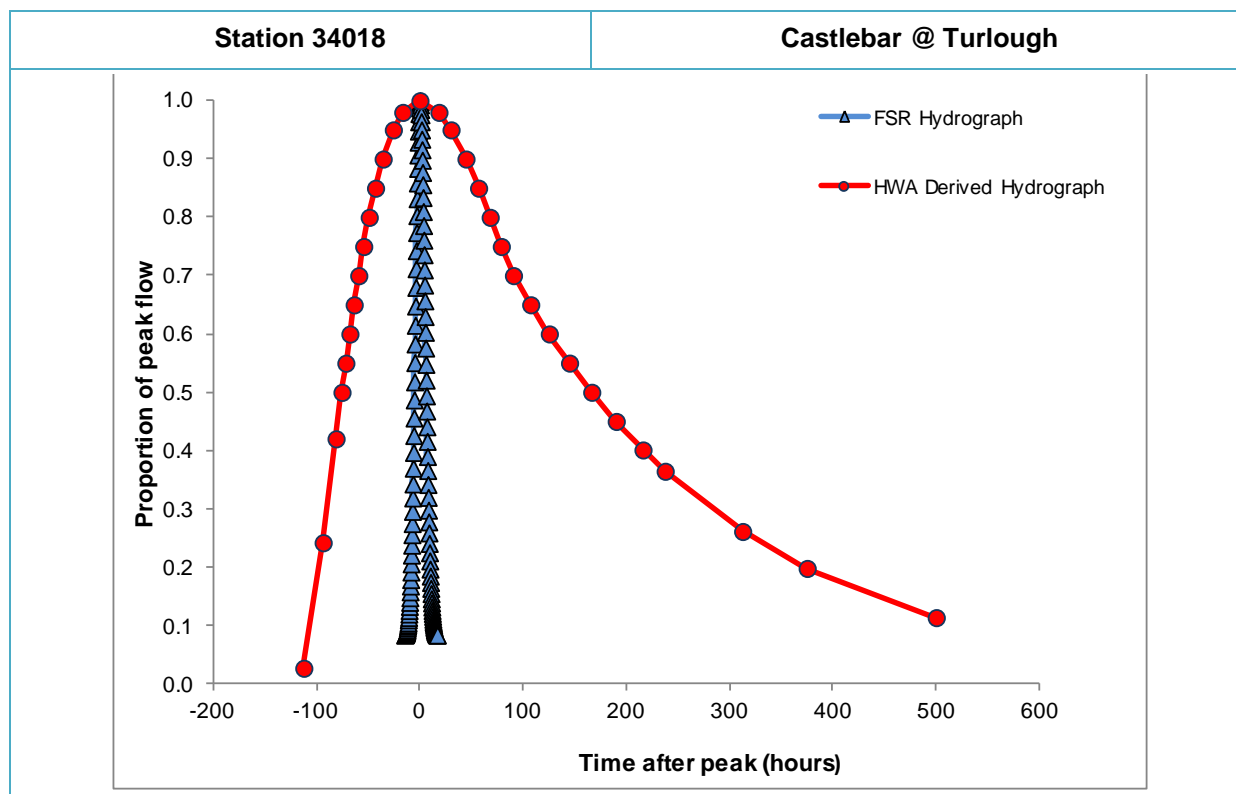
Flood width analysis summary sheet



Flood width analysis summary sheet



Flood width analysis summary sheet



Flood events used in the analysis

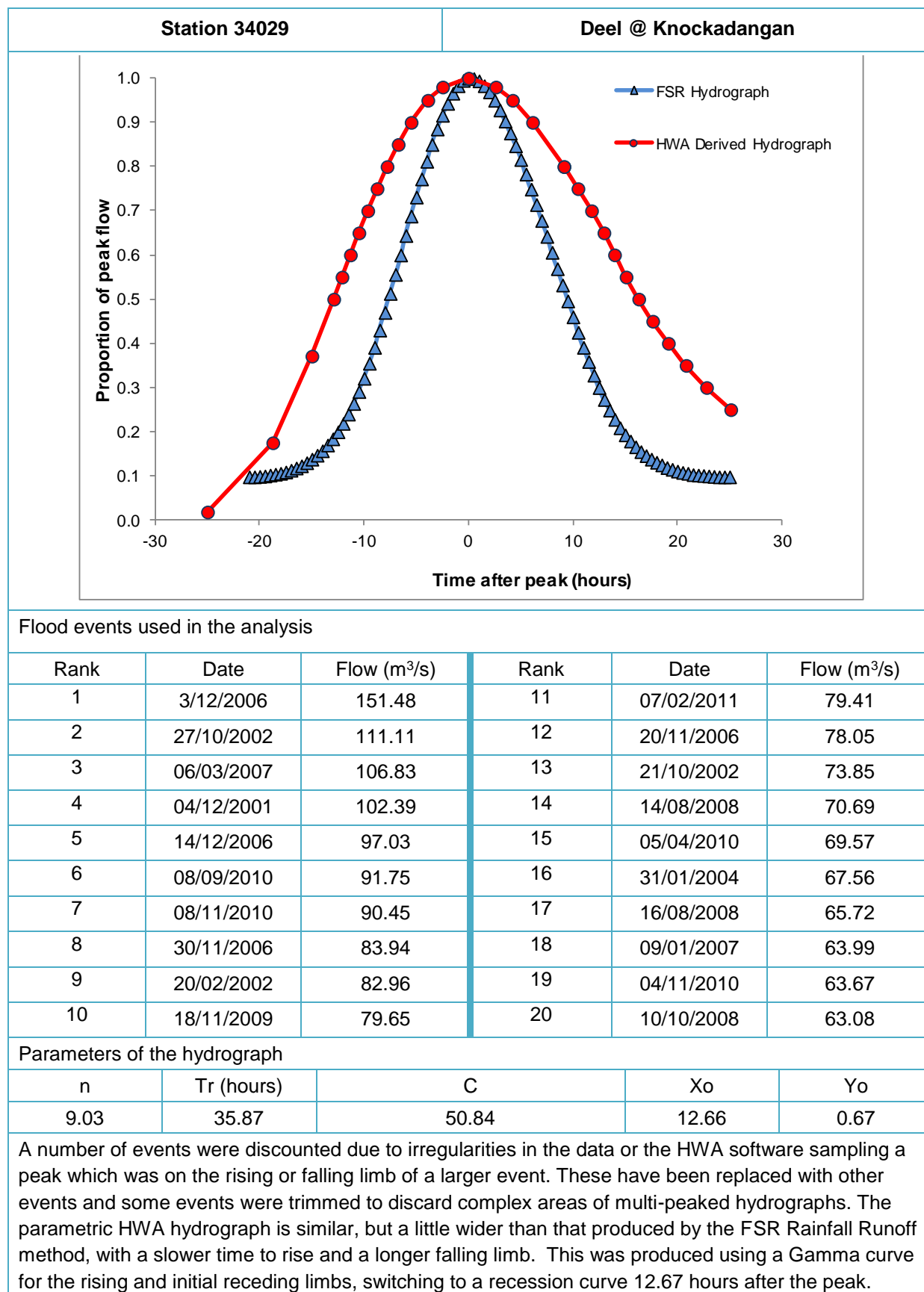
Rank	Date	Flow (m ³ /s)	Rank	Date	Flow (m ³ /s)
1	23/11/2009	19.37	11	05/02/1990	13.77
2	09/12/2007	18.01	12	05/12/2000	13.43
3	30/10/1989	16.37	13	08/02/2011	13.36
4	23/12/1999	15.14	14	29/10/2002	12.63
5	05/01/1991	14.85	15	11/12/1999	12.35
6	20/01/2005	14.50	16	28/01/1995	12.30
7	02/01/1999	14.47	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

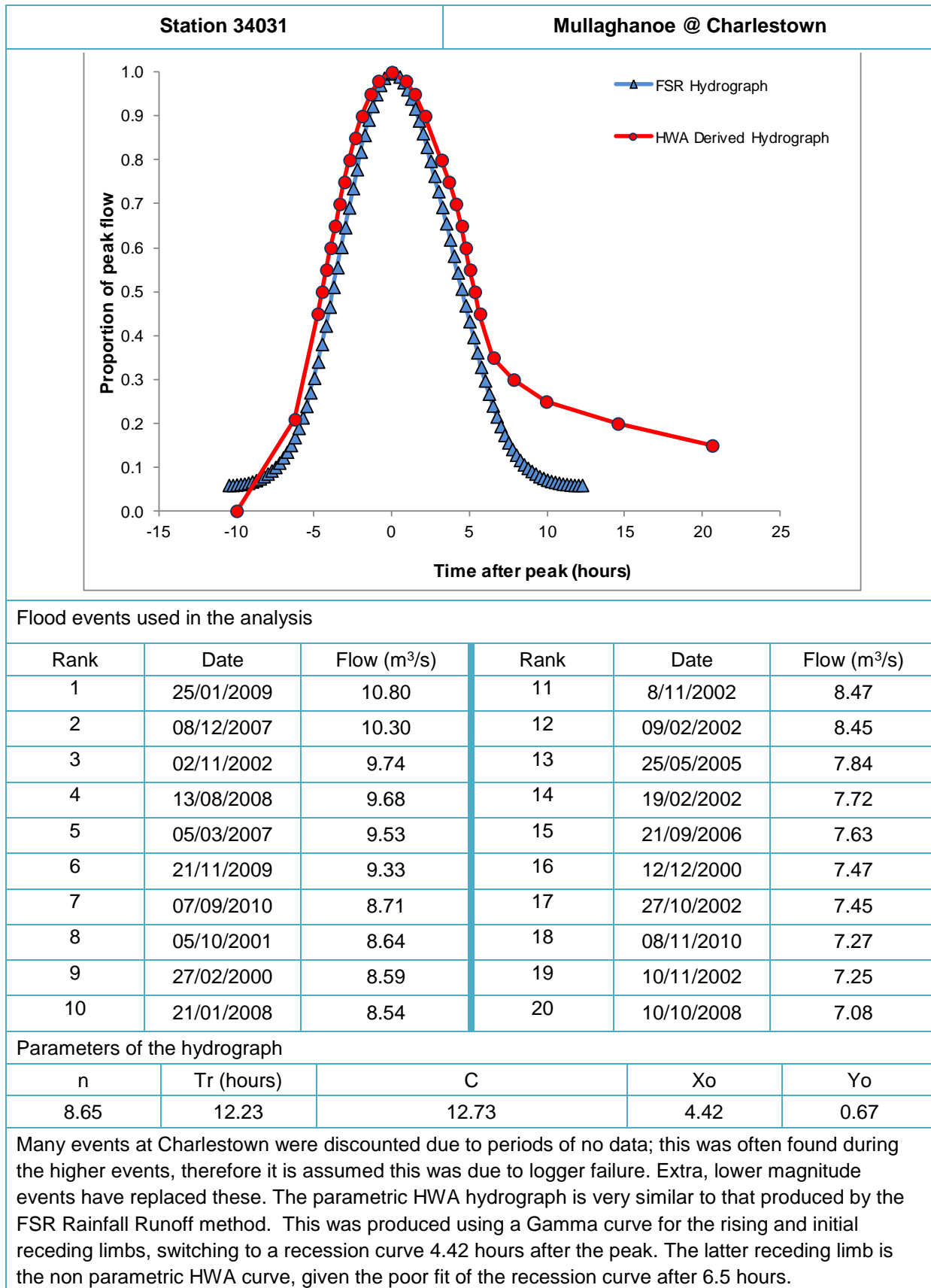
n	Tr (hours)	C	Xo	Yo
2.88	119.25	900.99	87.09	0.71

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.

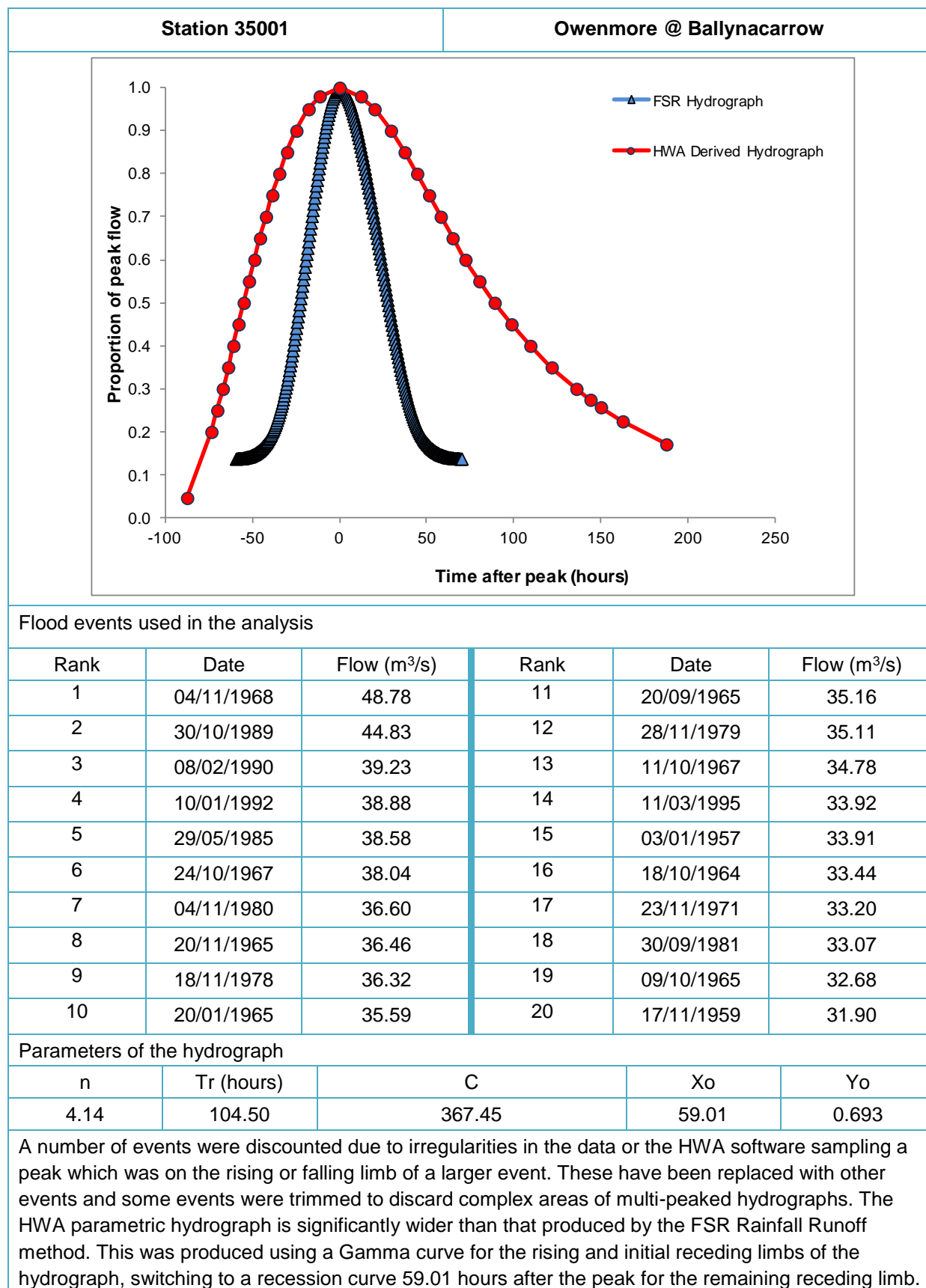
Flood width analysis summary sheet



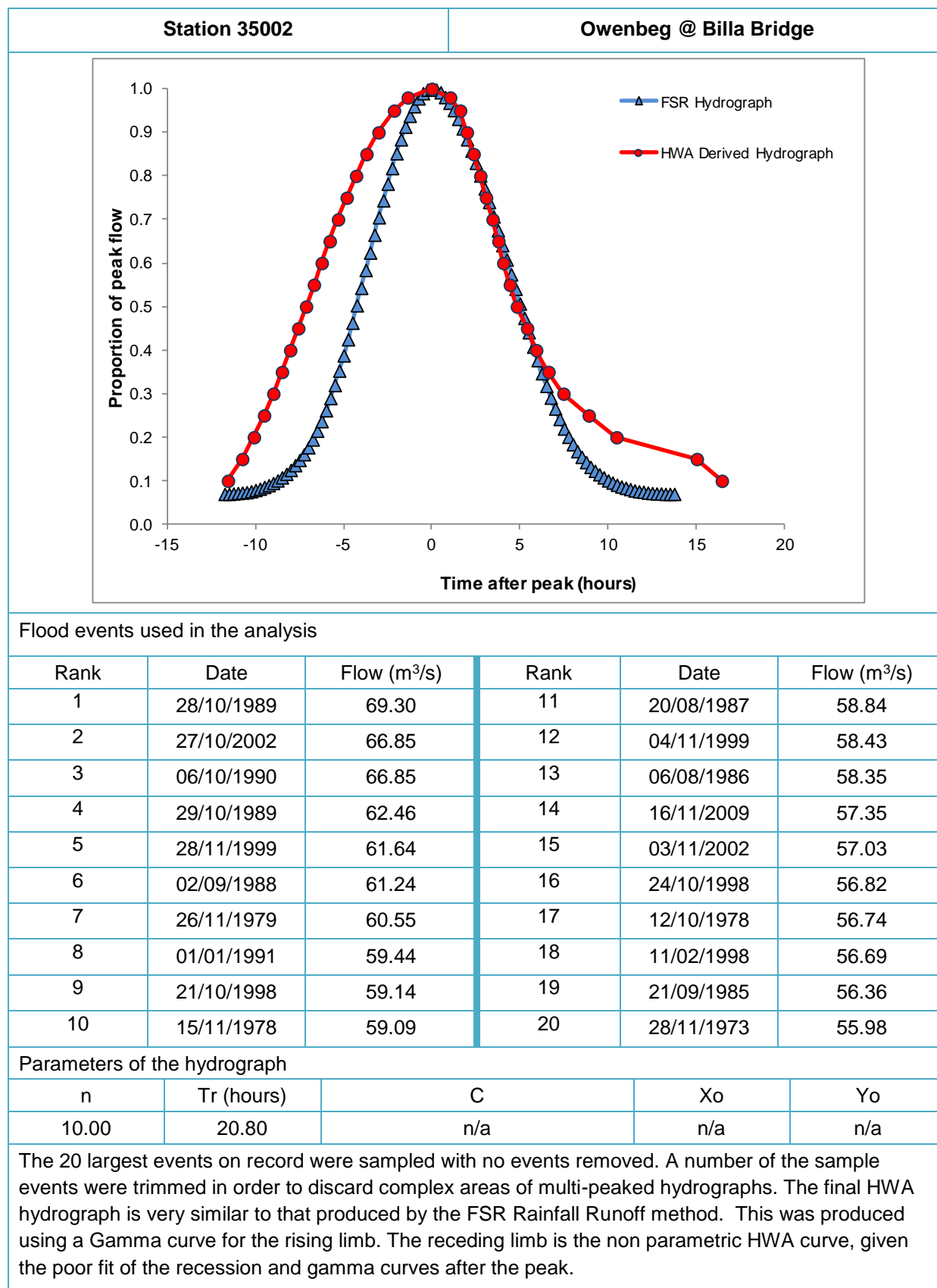
Flood width analysis summary sheet



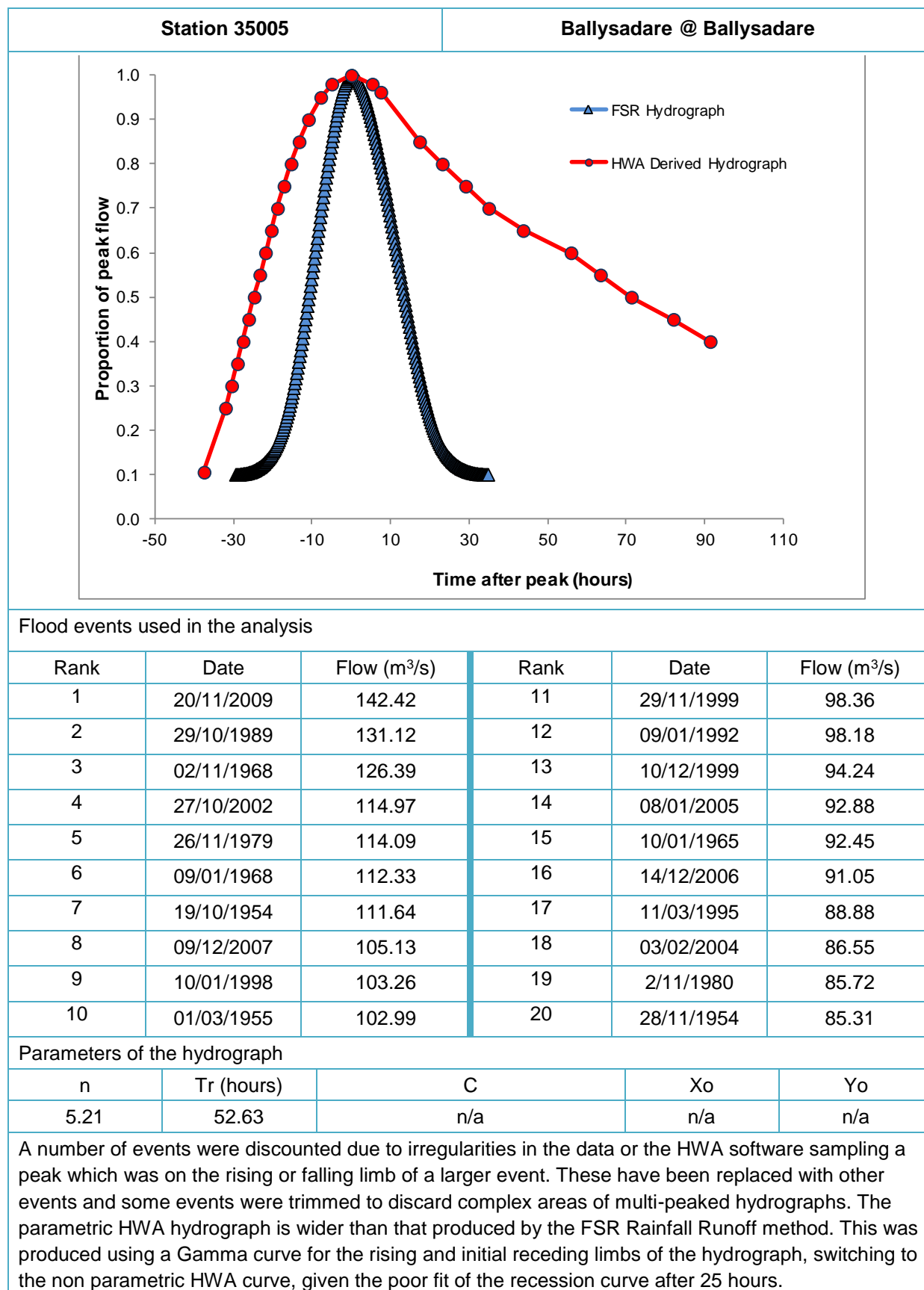
Flood width analysis summary sheet



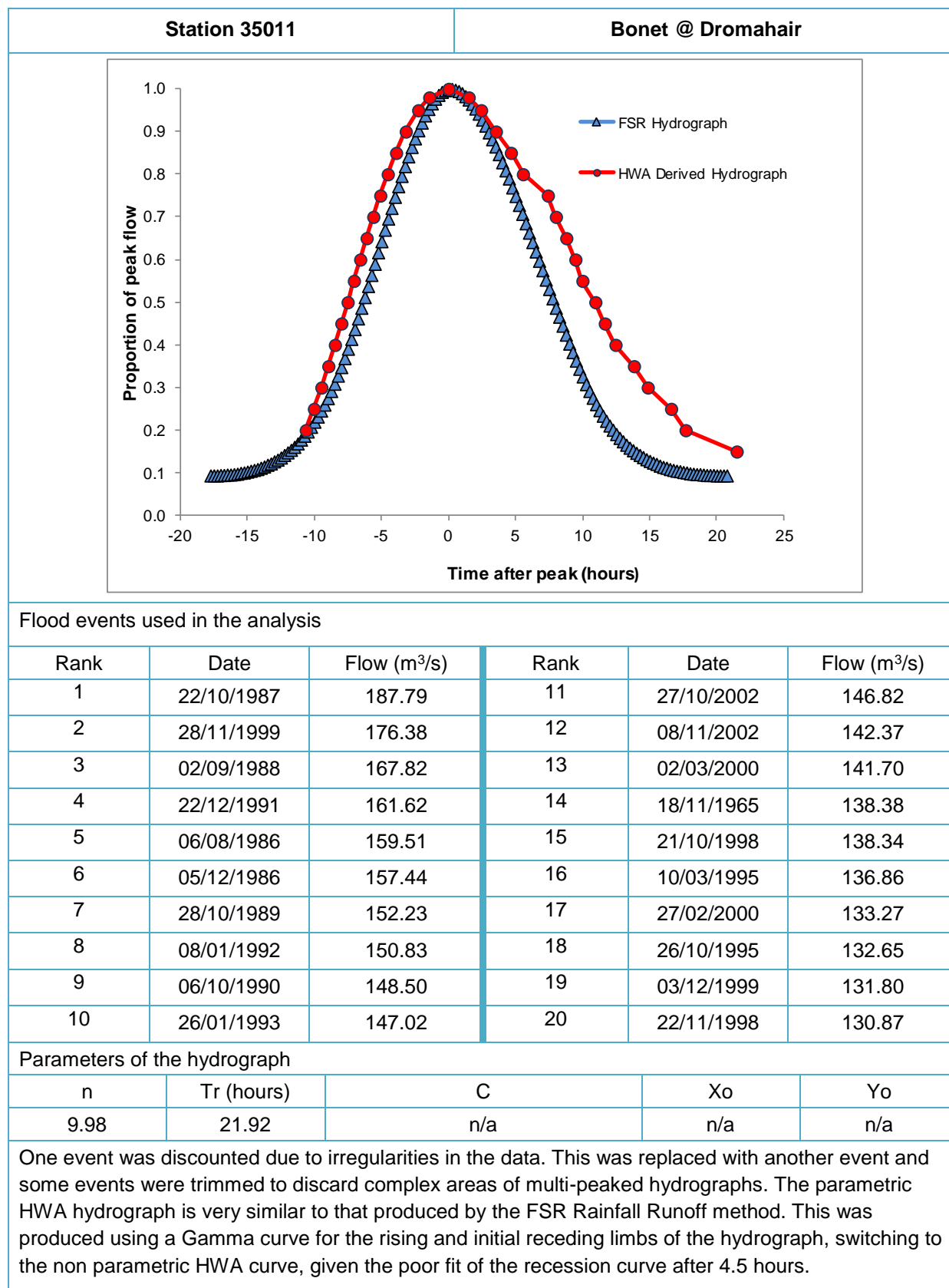
Flood width analysis summary sheet



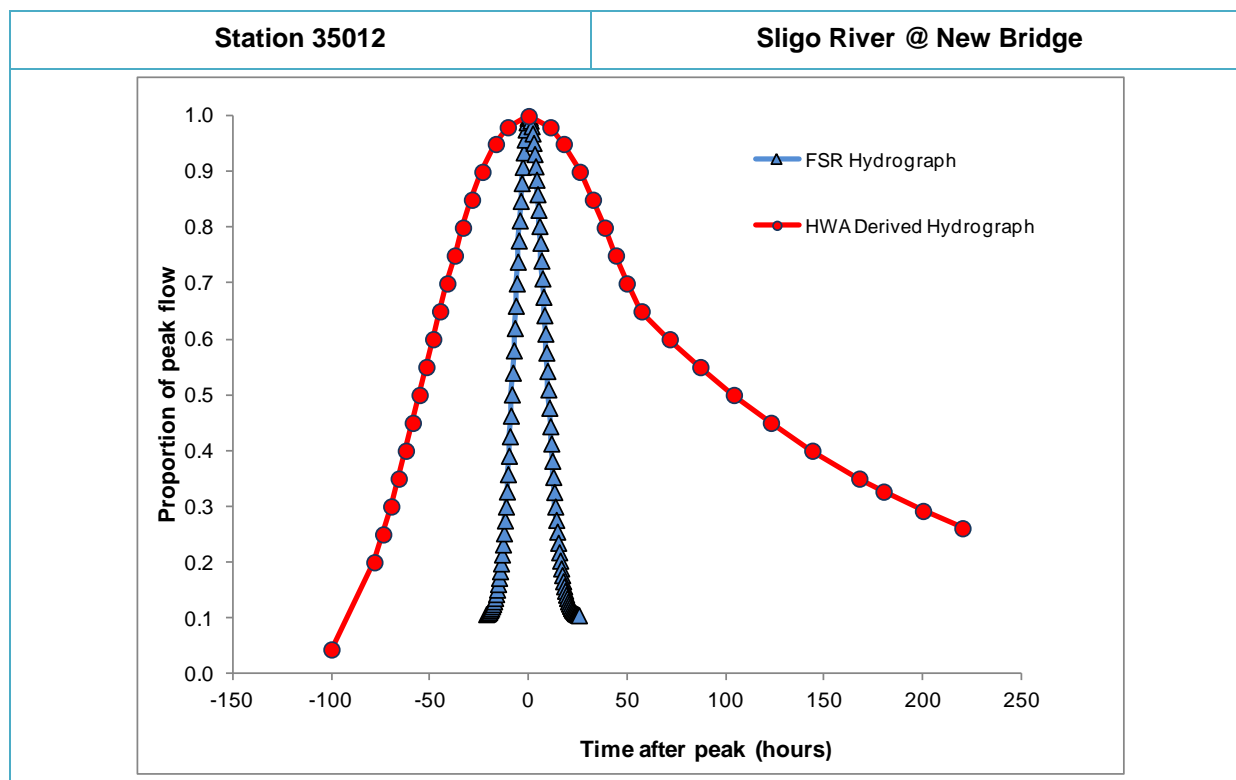
Flood width analysis summary sheet



Flood width analysis summary sheet



Flood width analysis summary sheet



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.

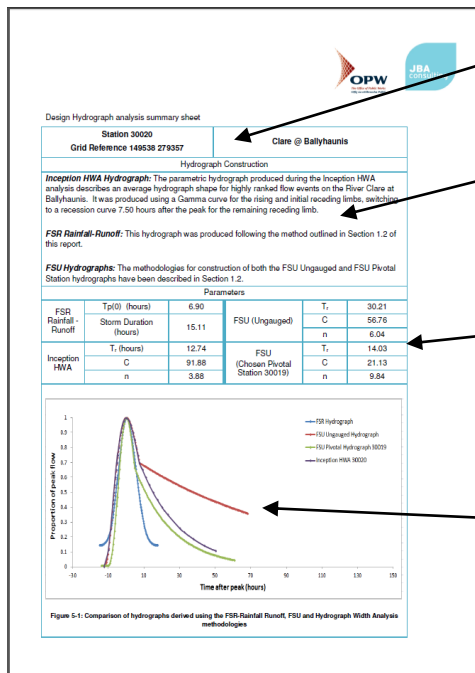
E Comparison of hydrograph shapes

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



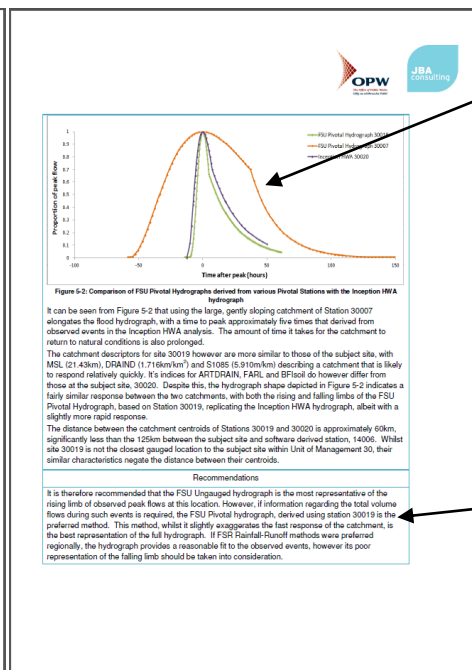
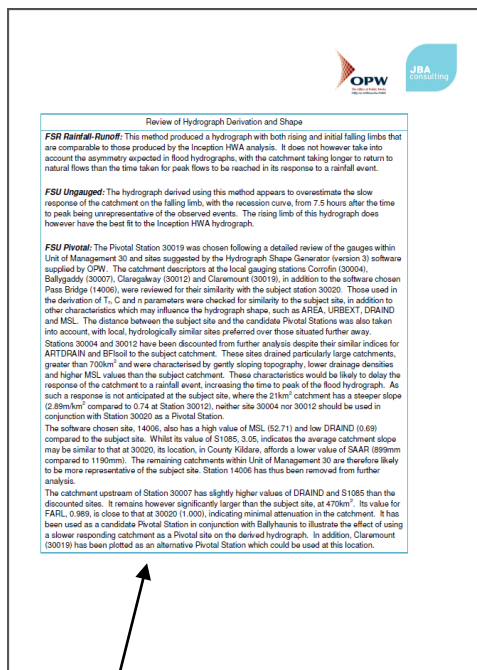
Site Information

Hydrograph construction

Parameters describing hydrograph shape

Plot of hydrographs

Derived using three ungauged-catchment methods (FSR, FSU and pivotal FSU) plus, for gauged catchments, HWA



Plot of FSU hydrographs derived using the candidate pivotal stations

Recommendations

Review of hydrograph derivation and shape

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)	Tr	30.21
	Storm Duration (hours)	15.11		C	56.76
				n	6.04
Inception HWA	Tr (hours)	12.74	FSU (Chosen Pivotal Station 30019)	Tr	14.03
	C	91.88		C	21.13
	n	3.88		n	9.84

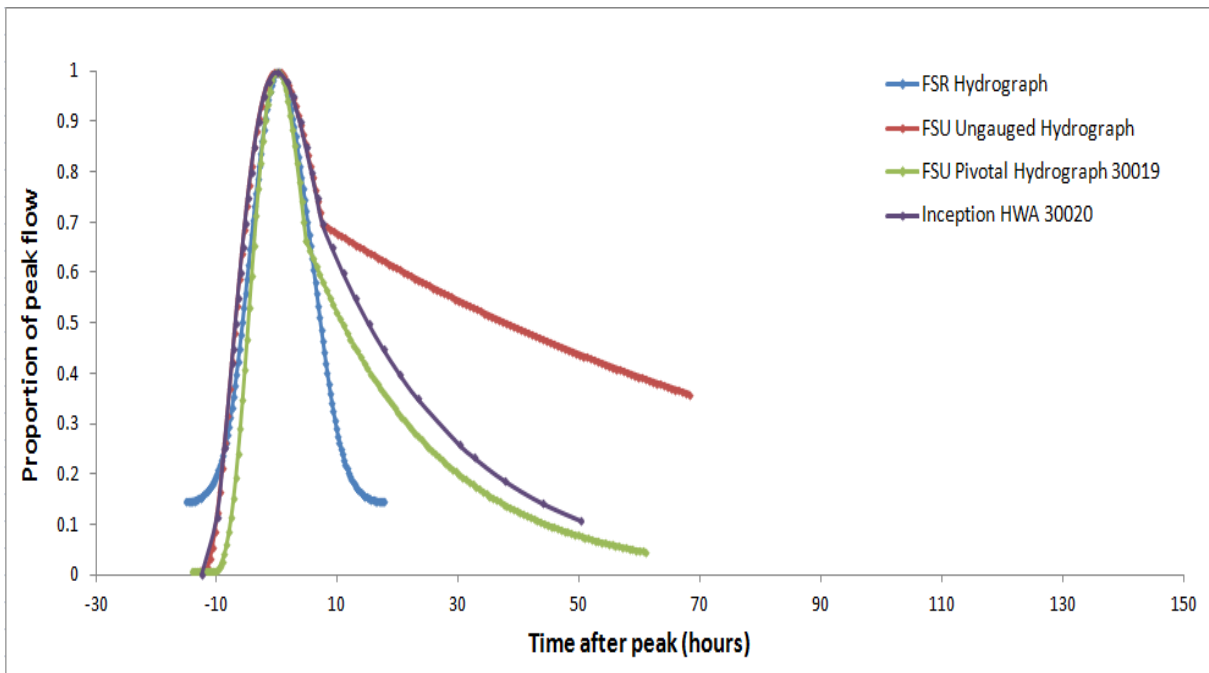


Figure 5-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

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, DRAIN 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 BFIsol to the subject catchment. These sites drained particularly large catchments, greater than 700km² 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 21km² catchment has a steeper slope (2.89m/km² 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 DRAIN (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 DRAIN and S1085 than the discounted sites. It remains however significantly larger than the subject site, at 470km². 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.

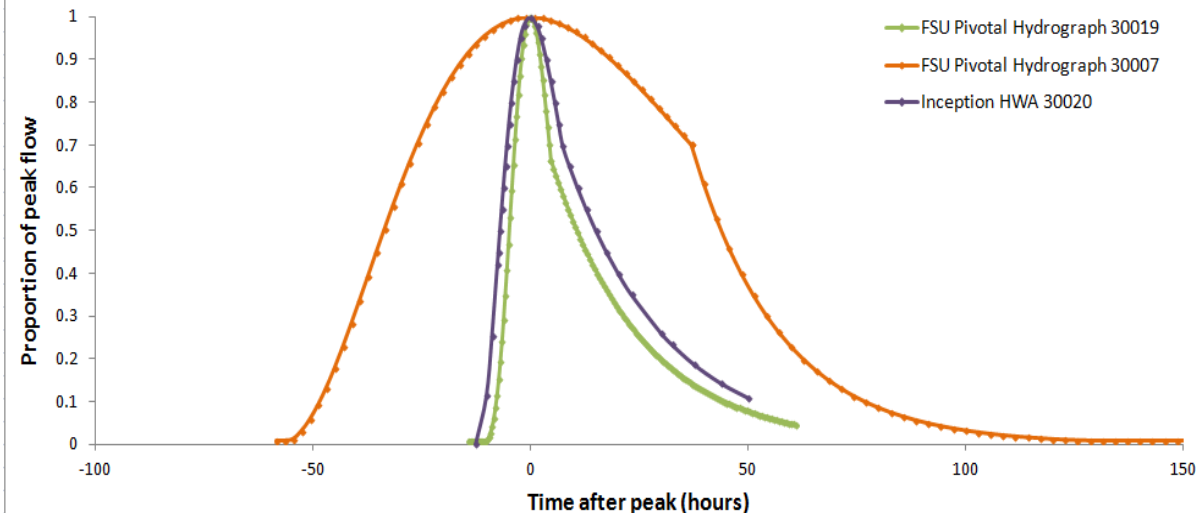


Figure 5-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

It can be seen from Figure 5-2 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), DRAIN (1.716km/km²) and S1085 (5.910m/km) describing a catchment that is likely to respond relatively quickly. Its indices for ARTDRAIN, FARL and BFIsoil do however differ from those at the subject site, 30020. Despite this, the hydrograph shape depicted in Figure 5-2 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

Station 32011 Grid Reference 80906 280601			Bunowen @ Louisburg Weir		
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 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.					
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	4.51	FSU (Ungauged)	Tr	13.93
	Storm Duration (hours)	11.79		C	7.91
				n	10.21
Inception HWA	Tr (hours)	7.59	FSU (Chosen Pivotal Station 32004)	Tr	6.17
	C	15.66		C	9.80
	n	8.30		n	10.00

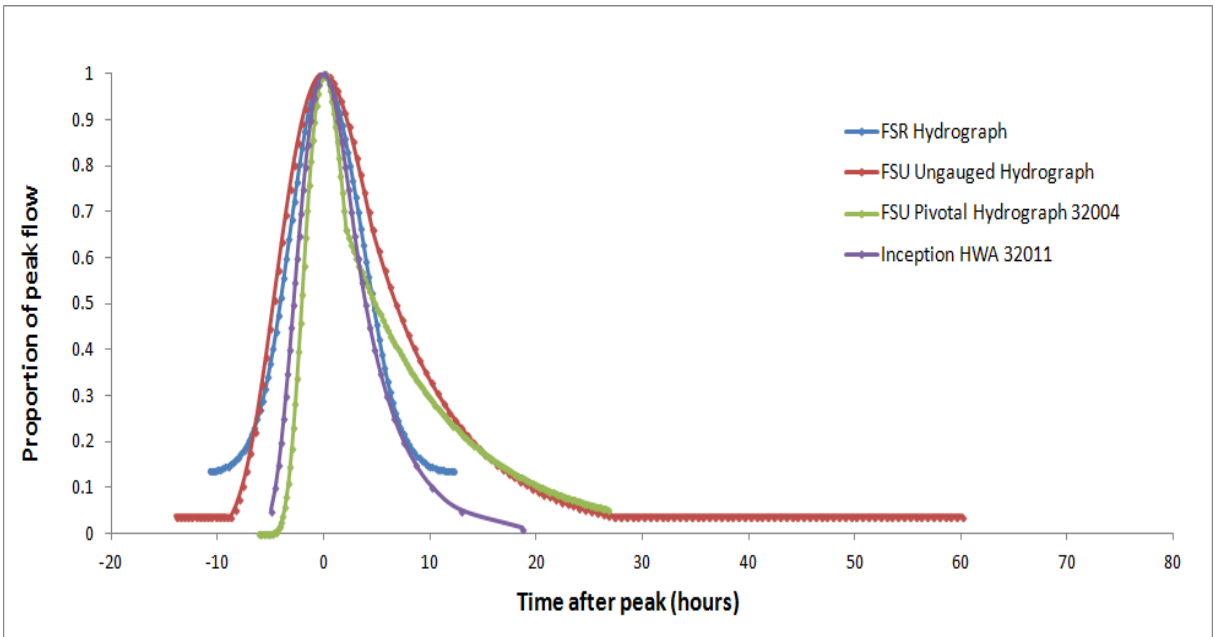


Figure 1-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

Review of Hydrograph Derivation and Shape

FSR Rainfall-Runoff: 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, DRAIN 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:

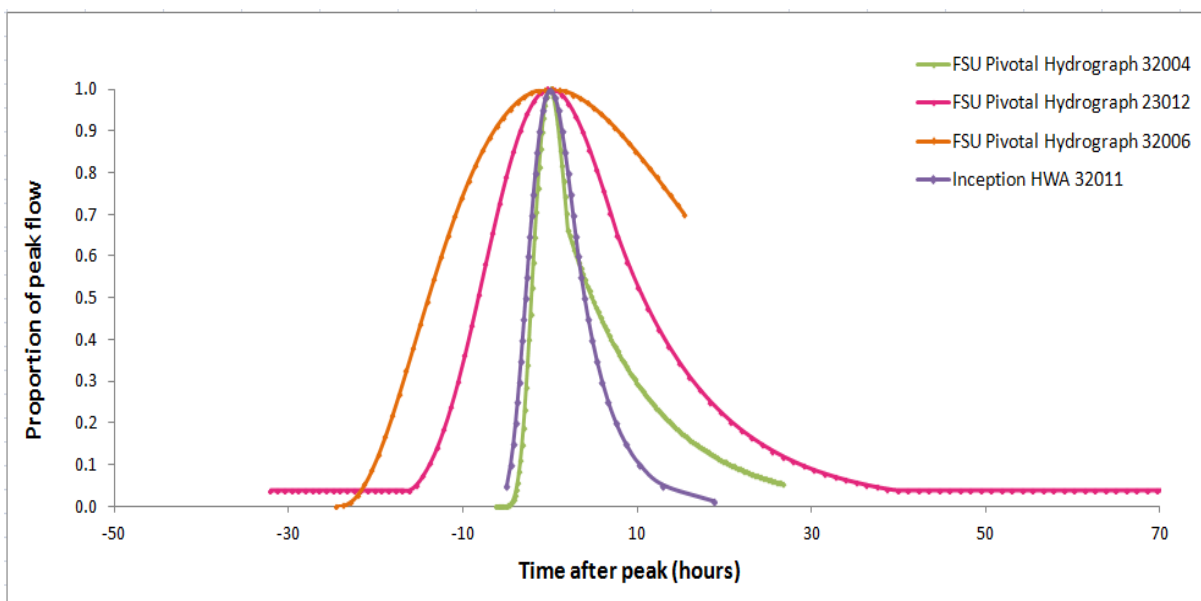


Figure 1-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

The catchment descriptors for these three sites are all fairly similar to the subject site, with station 32006 being the least representative, as BFIsol 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 DRAIN 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 70.1km² and 32.4km² 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 DRAIN and ALLUV. This is represented in Figure 1-2, 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)	Tr	21.30
	Storm Duration (hours)	18.54		C	9.26
				n	9.56
Inception HWA	Tr	20.98	FSU (Chosen Pivotal Station 27001)	Tr	17.72
	C	39.11		C	11.24
	n	8.89		n	5.00

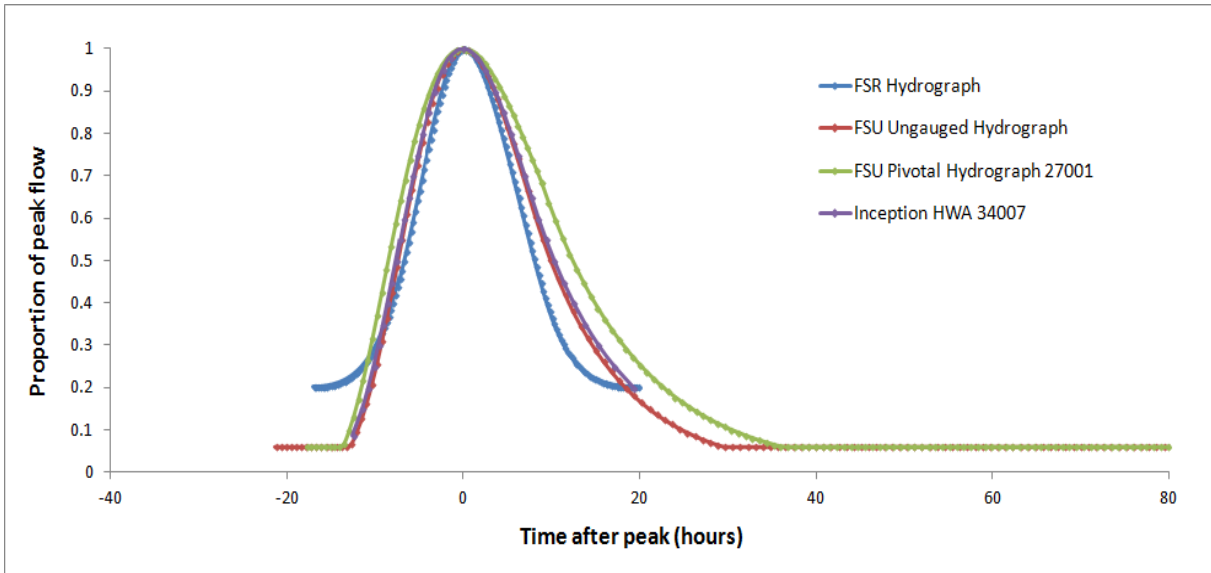


Figure 2-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

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, DRAIN 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 151.7km² and 46.7km² 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)	Tr	45.41
	Storm Duration (hours)	15.98		C	529.60
				n	2.02
Inception HWA	Tr	119.25	FSU (Chosen Pivotal Station 34003)	Tr	68.27
	C	900.99		C	n/a
	n	2.88		n	2.85

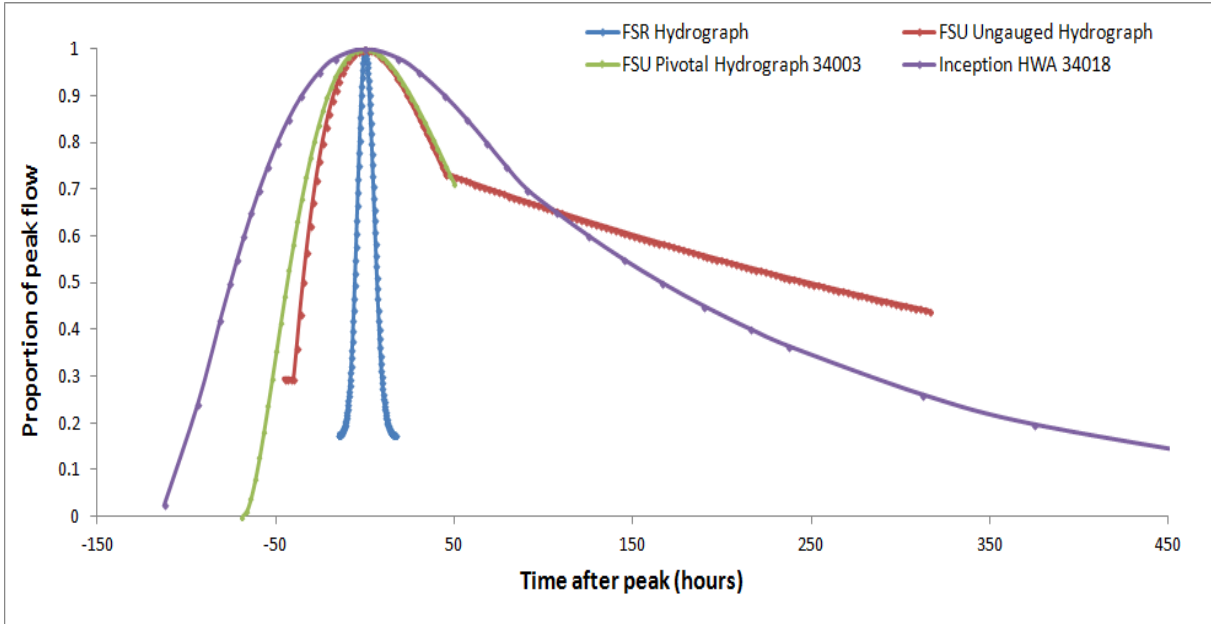


Figure 4-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

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, DRAIN 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:

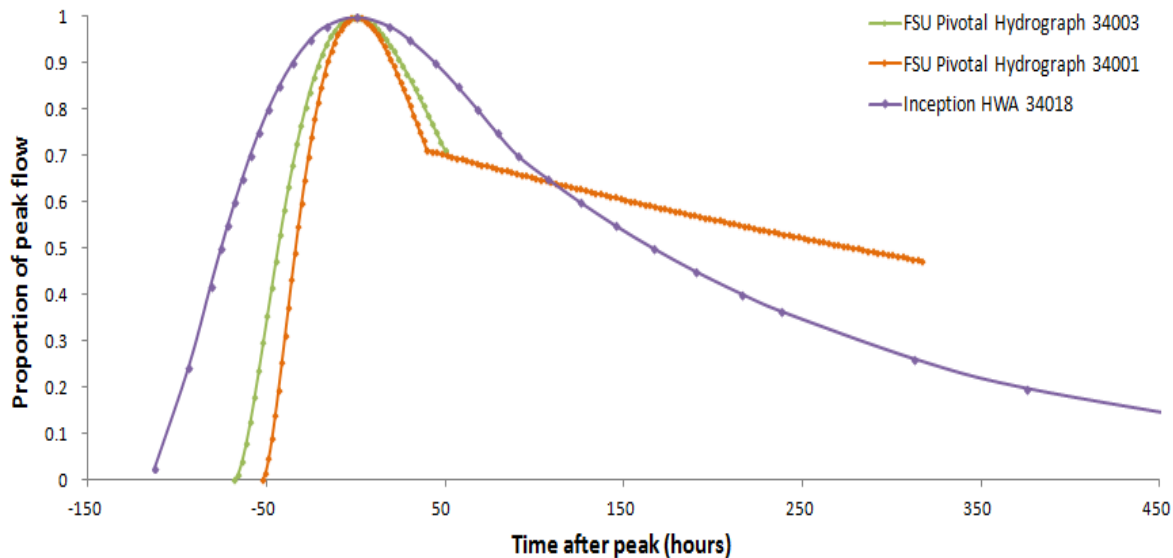


Figure 4-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

Whilst the values of ARTDRAIN, FARL, BFIsol 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. It would be beneficial to undertake a more detailed investigation into the hydraulics of the watercourse, as noted in the inception phase, in order to increase confidence in the observed data used during this comparison. This will be possible once the hydraulic model of the Castlebar River is complete. For such sites, neither the FSU Ungauged Hydrograph nor the FSR Rainfall-Runoff methodologies should be used. 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

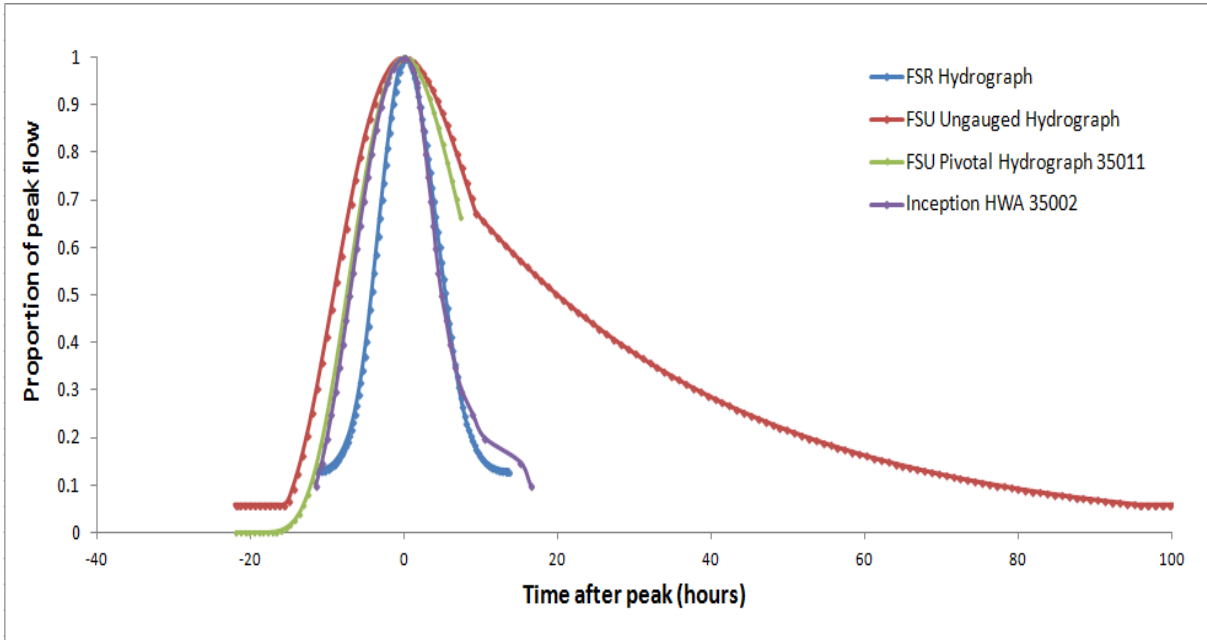


Figure 3-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff, FSU and Hydrograph Width Analysis methodologies

Review of Hydrograph Derivation and Shape	
FSR Rainfall-Runoff: This method produced a hydrograph with a 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	

Review of Hydrograph Derivation and Shape

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:

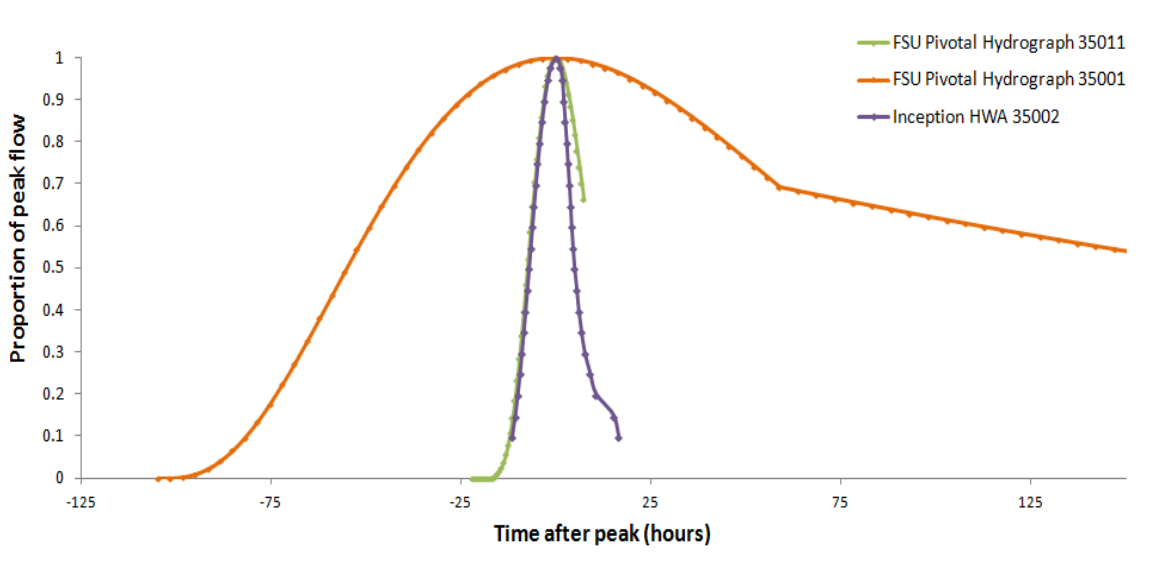


Figure 3-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations with the Inception HWA hydrograph

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 3-2. 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

Ungauged Site Grid Reference 152472 229990			Clarinbridge @ Athenry		
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	7.87	FSU (Ungauged)	T _r	50.03
				C	42.48
				n	6.54
	Storm Duration (hours)	16.6	FSU (Pivotal site 26022)	T _r	53.64
				C	66.94
				n	5.00

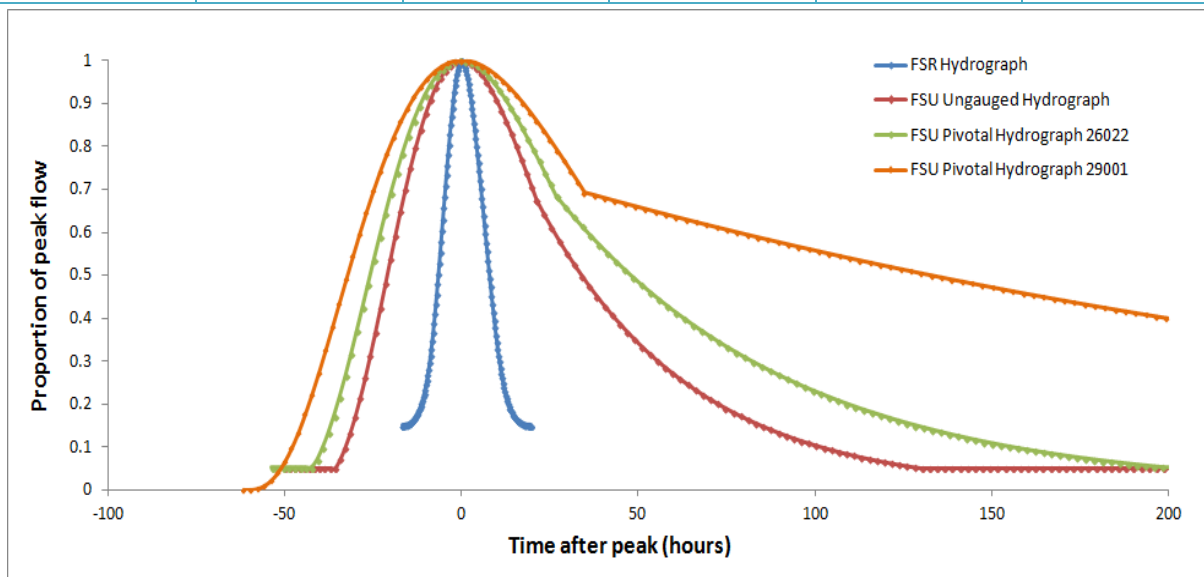


Figure 7-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

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, DRAIN 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 DRAIN, 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 (193km² compared to 32km²) 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 7-1. The values of DRAIN, 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 7-1 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 DRAIN 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 7-1.

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.

Design Hydrograph analysis summary sheet

Ungauged Site Grid Reference 166343 315264			Carrigans Upper @ Ballymote		
Parameters					
FSR Rainfall - Runoff	Tp(0) (hours)	3.08	FSU (Ungauged)	Tr	56.31
				C	49.78
				n	6.26
	Storm Duration (hours)	6.67	FSU (Pivotal site 26022)	Tr	41.65
				C	51.98
				n	5.00

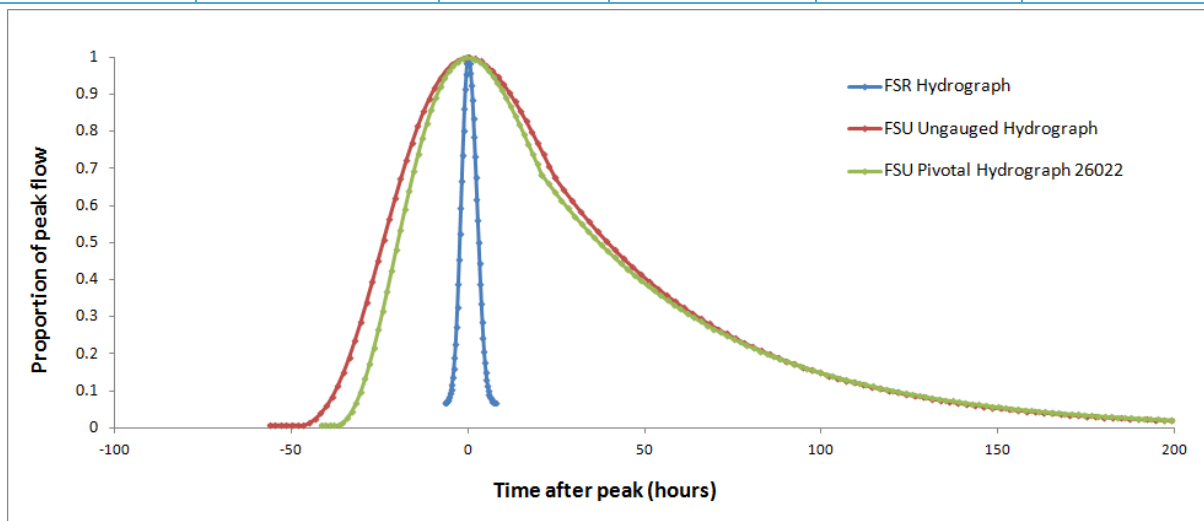


Figure 8-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

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 2.5km², 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, DRAIN 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 (299km² at Station 35001 compared to 2.5km² at the subject site), MSL (24.7km at Ballynacarrow compared to 2.2 at Ballymote) and URBEXT (0.33 at Station 35001 compared to 14.57 at Ballymote). In addition, S1085, which influences the T_r parameter, varies from 0.1 at the candidate pivotal station to 2.6 at the subject site. More suitable values are present for DRAIN, 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, DRAIN 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 8-1, 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 (61.9km² and 13.9km 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 8-1 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.

Design Hydrograph analysis summary sheet

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

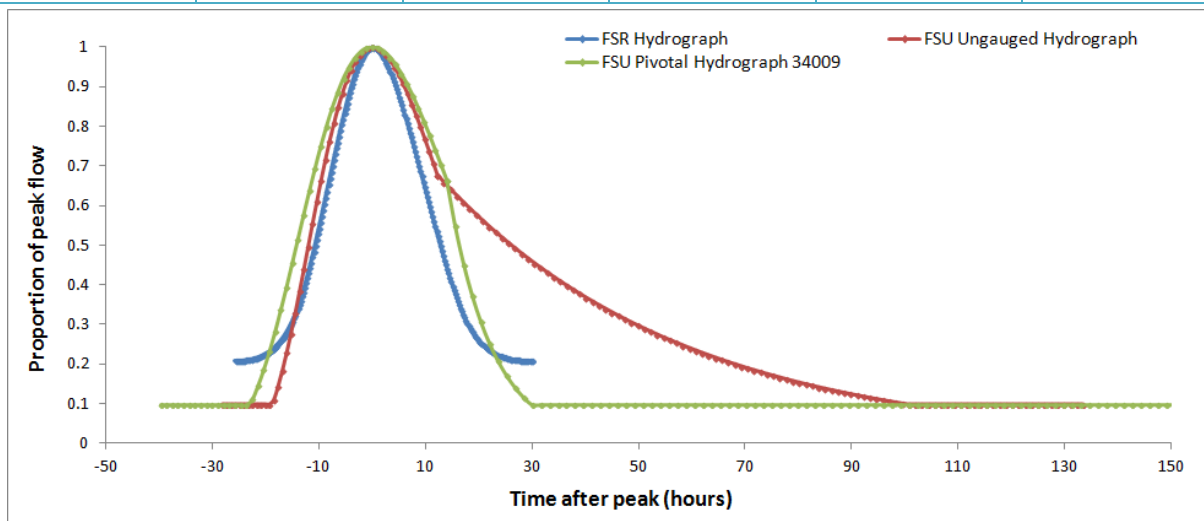


Figure 9-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

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 (BFIsol 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, DRAIN 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.

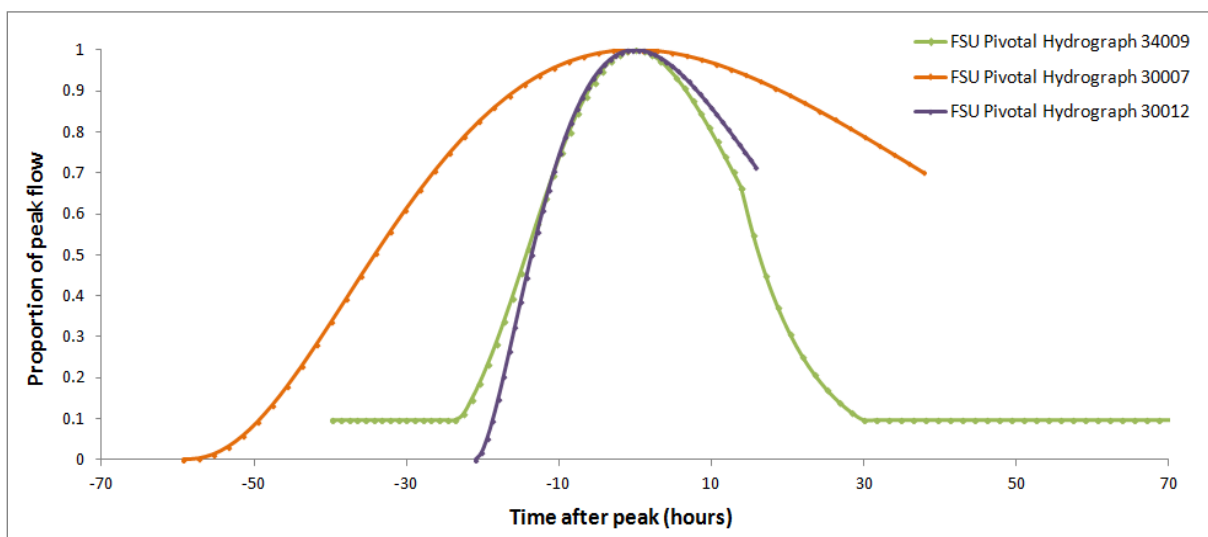


Figure 9-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations

The FSU Pivotal hydrographs have been plotted for these three stations in Figure 9-2. 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, DRAIN, 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 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

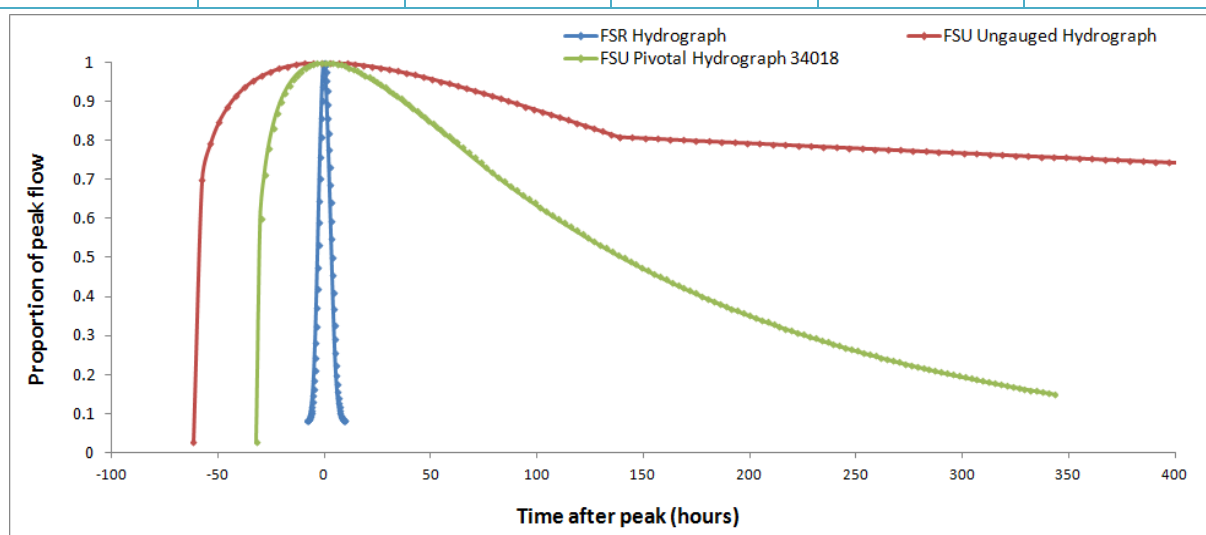


Figure 10-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

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 km²), 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 than its initial rapid 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, DRAIN 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 10-1. Station 34018, whilst still relatively large at 95.4km², 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 10-1.

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

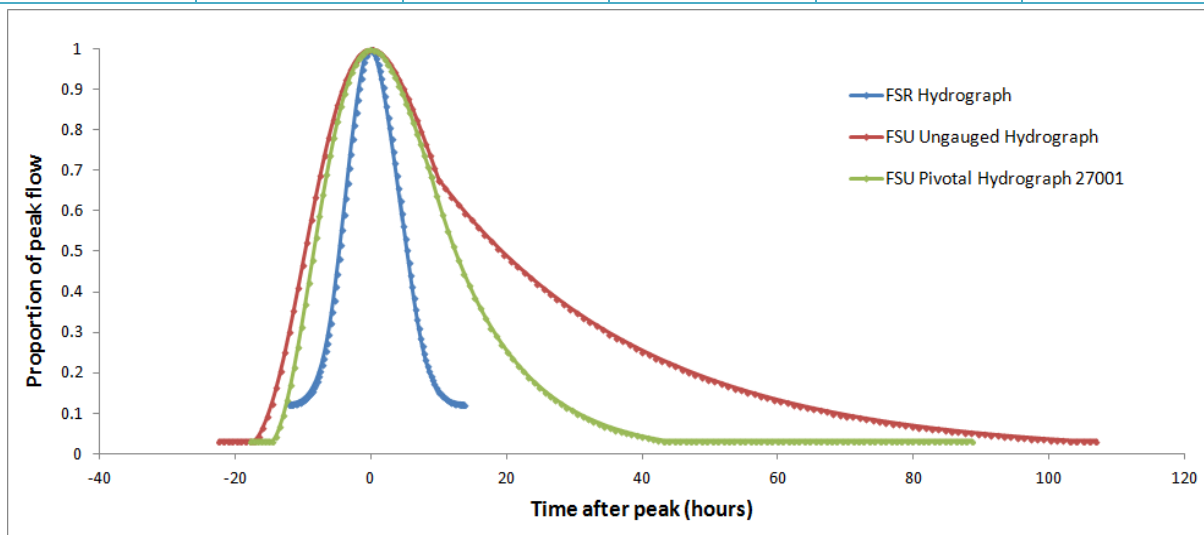


Figure 6-1: Comparison of hydrographs derived using the FSR-Rainfall Runoff and FSU methodologies

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, DRAIN 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 DRAIN 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 DRAIN, 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 6-2 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 6-2. This is likely to be due to the shallow gradient of the 117km² catchment (S1085 is 3.33m/km), which, despite arterial drainage, routes flows relatively slowly to the gauging station. The subject catchment is much smaller (13.2km²) 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 13km² 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.

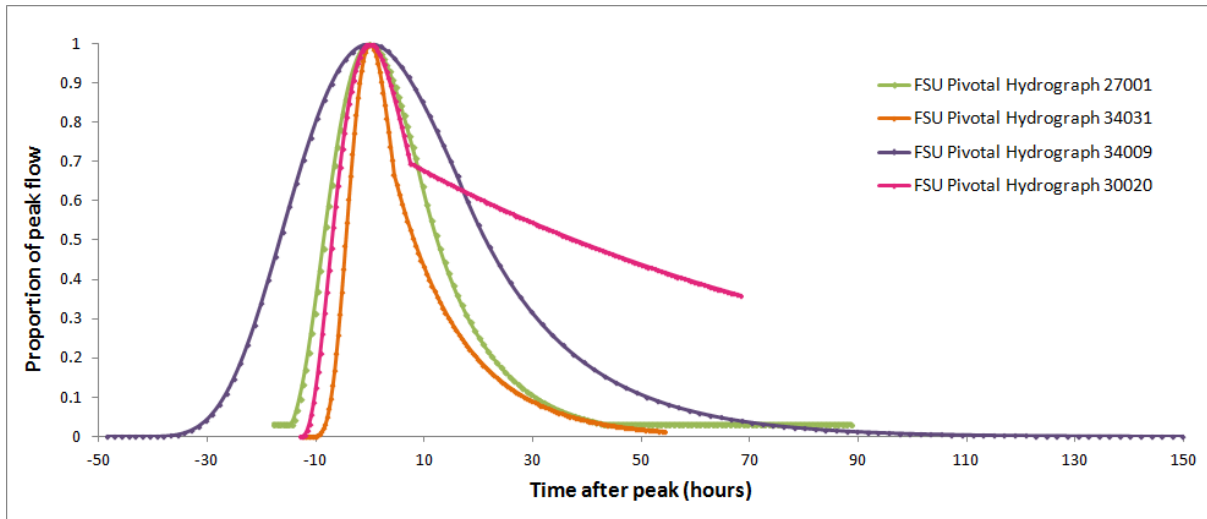


Figure 6-2: Comparison of FSU Pivotal Hydrographs derived from various Pivotal Stations

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



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 (m3/s) by AEP(%)							
						Single- site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
ADN_001	34_1908_1	125407	317472			P	GL	1.134	1.569	0.078	0.103	0.122	0.142	0.173	0.200	0.227	0.314
ADN_002	34_1908_1	125026	318018			P	GL	1.134	1.569	0.313	0.411	0.485	0.565	0.689	0.800	0.907	1.255
ADN_003	34_3982_2	124740	318666			P	GL	1.110	1.460	0.225	0.290	0.336	0.385	0.457	0.520	0.577	0.759
BAL_001	34_3640_2	126643	317985			P	GL	1.124	1.523	0.048	0.063	0.075	0.087	0.106	0.123	0.138	0.187
BAL_002	34_3640_3	126483	318431			P	GL	1.124	1.522	0.285	0.371	0.435	0.503	0.607	0.698	0.784	1.063
BAL_003	34_3986_1	126468	318486			P	GL	1.118	1.495	28.988	34.720	38.596	42.557	48.178	52.833	59.058	79.007
BAL_004	34_3986_5	125397	319579			P	GL	1.118	1.495	28.933	34.766	38.721	42.770	48.530	53.310	59.581	79.675
BLR_001	34_3271_1	148780	302997	Regional	1.00	P	GL	1.125	1.529	5.911	7.278	8.262	9.314	10.881	12.242	13.775	18.721
BLR_002	34_3271_3	147924	302678	Regional	1.00	P	GL	1.124	1.524	6.357	7.726	8.679	9.672	11.114	12.335	13.865	18.800
BLR_003	34_3271_4	147501	302746	Regional	1.00	P	GL	1.124	1.522	6.485	7.882	8.854	9.868	11.339	12.584	14.140	19.152
BUN_001	34_3988_1	127338	319666			P	GL	1.133	1.567	0.140	0.185	0.218	0.254	0.309	0.357	0.404	0.559
BUN_002	34_3988_2	126105	319564			P	GL	1.135	1.574	0.503	0.657	0.771	0.896	1.085	1.252	1.421	1.971
BUN_003	34_3988_3	125627	319670			P	GL	1.138	1.588	0.674	0.885	1.041	1.211	1.470	1.698	1.933	2.697
BUN_004	34_3988_4	125471	319707			P	GL	1.138	1.589	0.710	0.933	1.098	1.277	1.549	1.790	2.038	2.845
CLD_001	34_3951_4	123350	296321	34018	0.68	P	GL	1.116	1.487	22.497	27.767	31.394	35.148	40.552	45.087	50.321	67.051
CLD_002	34_3952_1	123371	296372	34018	0.68	P	GL	1.113	1.474	37.658	44.799	49.681	54.709	61.906	67.915	75.601	100.122
CLD_003	34_289_1	122962	299461	34018	0.68	P	GL	1.113	1.472	37.206	44.134	48.698	53.276	59.642	64.810	72.113	95.402
CLD_004	34_3412_2	122652	300572	34018	0.68	P	GL	1.113	1.471	37.346	44.300	48.881	53.476	59.866	65.054	72.376	95.725
CTB_001	34_1784_1	113906	290216	34018	0.68	P	GL	1.119	1.499	9.385	11.320	12.587	13.852	15.603	17.017	19.038	25.514
CTB_002	34_3392_1	114070	290415	34018	0.68	P	GL	1.119	1.500	9.756	11.767	13.084	14.399	16.218	17.688	19.791	26.529
CTB_003	34_3392_3	114549	290589	34018	0.68	P	GL	1.119	1.500	9.797	11.817	13.139	14.460	16.287	17.763	19.875	26.640

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 (m3/s) by AEP(%)							
						Single- site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
CTB_004	34_3393_1	114594	290606	34018	0.68	P	GL	1.119	1.501	10.399	12.543	13.946	15.348	17.287	18.854	21.101	28.304
CTB_005	34_3393_3	115311	290975	34018	0.68	P	GL	1.119	1.500	10.353	12.487	13.885	15.280	17.211	18.771	21.004	28.160
CTB_006	34_3393_6	116482	291767	34018	0.68	P	GL	1.119	1.499	10.753	12.970	14.421	15.870	17.876	19.496	21.810	29.227
CTB_007	34_1775_1	118407	292519	34018	0.68	P	GL	1.118	1.494	10.909	13.113	14.547	15.974	17.938	19.519	21.815	29.166
CTB_008	34_1741_1	121640	293907	34018	0.68	P	GL	1.117	1.493	11.740	14.112	15.655	17.191	19.305	21.006	23.469	31.352
CTB_008a	34_1772_5	120587	293529	34018	0.68	P	GL	1.117	1.492	11.472	13.635	15.039	16.431	18.345	19.881	22.209	29.659
CTB_009	34_302_1	122043	294098	34018	0.68	P	GL	1.113	1.473	26.548	31.512	34.814	38.147	42.816	46.635	51.902	68.709
CTB_010	34_3953_3	123399	296317	34018	0.68	P	GL	1.114	1.478	29.841	35.732	39.712	43.776	49.539	54.309	60.497	80.257
DEE_006	34_3741_1	125202	302503	34003	1.05	P	GL	1.112	1.471	58.872	71.223	79.528	87.980	99.921	109.768	122.101	161.421
DEE_007	34_3741_3	125865	302383	34003	1.05	P	GL	1.112	1.470	58.510	70.785	79.039	87.439	99.307	109.093	121.335	160.363
DGY_001	34_3706_1	125736	302959			P	GL	1.111	1.466	0.219	0.279	0.322	0.367	0.433	0.491	0.545	0.719
FFD_001	34_1979_1	127905	304273			P	GL	1.131	1.557	0.231	0.296	0.343	0.393	0.469	0.535	0.606	0.833
FFD_002	34_1979_1	127699	304968			P	GL	1.131	1.557	0.754	0.957	1.107	1.269	1.514	1.731	1.959	2.695
FFD_003	34_1979_2	127261	305105			P	GL	1.129	1.547	0.893	1.134	1.311	1.503	1.794	2.050	2.315	3.172
FFD_004	34_1979_4	127214	305707			P	GL	1.126	1.535	1.023	1.289	1.480	1.683	1.987	2.251	2.536	3.455
FFD_005	34_1992_1	127213	305747			P	GL	1.125	1.528	2.271	2.844	3.240	3.649	4.239	4.734	5.326	7.235
FUL_001	34_2290_2	147598	300574			P	GL	1.126	1.531	0.360	0.464	0.539	0.619	0.738	0.842	0.948	1.289
GAR_001	34_3524_1	124669	321243			P	GL	1.139	1.591	0.038	0.050	0.059	0.068	0.083	0.096	0.109	0.153
GAR_002	34_3524_1	124961	320996			P	GL	1.139	1.591	0.034	0.045	0.053	0.062	0.075	0.087	0.099	0.138
GLR_001	34_4000_1	128137	318007			P	GL	1.118	1.495	26.493	31.664	35.098	38.562	43.408	47.366	52.942	70.807
GLR_002	34_3640_3	126476	318456			P	GL	1.118	1.495	29.145	34.908	38.805	42.787	48.438	53.119	59.376	79.426



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 (m3/s) by AEP(%)							
						Single- site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
GWS_001	34_1461_5	129551	298321			P	GL	1.117	1.494	52.064	62.982	70.393	77.985	88.790	97.763	109.239	146.033
KKT_001	34_193_1	123429	318099			P	GL	1.127	1.538	0.457	0.607	0.717	0.836	1.017	1.176	1.325	1.808
KKT_002	34_193_1	123221	317633			P	GL	1.127	1.538	0.513	0.681	0.805	0.939	1.141	1.320	1.488	2.030
KNK_001	34_3984_1	123514	320422			P	GL	1.126	1.536	1.943	2.519	2.946	3.413	4.124	4.756	5.358	7.304
KNK_002	34_3984_2	123823	320224			P	GL	1.126	1.532	2.023	2.622	3.067	3.552	4.293	4.951	5.573	7.585
KNK_003	34_3984_5	124355	319290			P	GL	1.129	1.547	2.688	3.463	4.032	4.649	5.583	6.406	7.232	9.912
KNK_004	34_3984_7	124895	319047			P	GL	1.132	1.560	3.200	4.123	4.800	5.535	6.647	7.627	8.632	11.898
KNR_001	34_3102_4	115705	289191			P	GL	1.119	1.503	0.093	0.117	0.134	0.152	0.179	0.202	0.227	0.304
KNR_002	34_3102_4	115103	289168			P	GL	1.119	1.503	0.395	0.491	0.559	0.630	0.735	0.823	0.922	1.238
KNT_001	34_482_1	114359	291844			P	GL	1.124	1.522	0.702	0.863	0.977	1.096	1.271	1.420	1.596	2.162
KNT_002	34_482_2	114287	291393			P	GL	1.126	1.532	0.809	1.013	1.157	1.311	1.539	1.735	1.953	2.657
KNT_003	34_482_3	114288	290933			P	GL	1.128	1.541	0.901	1.109	1.255	1.408	1.632	1.824	2.057	2.811
KNT_005	34_482_5	114048	290411			P	GL	1.130	1.551	1.035	1.280	1.454	1.637	1.908	2.140	2.419	3.319
LPK_001	34_2475_1	148439	301063			P	GL	1.129	1.548	0.409	0.528	0.614	0.705	0.840	0.958	1.082	1.483
LPK_002	34_2475_2	148267	301297			P	GL	1.128	1.542	0.499	0.643	0.747	0.858	1.024	1.167	1.316	1.800
LPK_003	34_2475_4	147753	301657			P	GL	1.130	1.549	0.729	0.936	1.085	1.245	1.484	1.691	1.910	2.620
MIL_000	34_3102_2	114160	288263			P	GL	1.120	1.508	3.926	4.780	5.379	6.005	6.919	7.696	8.623	11.602
MIL_001	34_3102_3	114736	288495			P	GL	1.120	1.506	4.279	5.211	5.863	6.546	7.543	8.389	9.396	12.632
MIL_002	34_3102_4	114968	289005			P	GL	1.119	1.503	4.276	5.152	5.758	6.387	7.295	8.059	9.022	12.113
MNU_001	34_1731_2	122045	294015	34018	0.68	P	GL	1.114	1.478	17.831	21.472	23.893	26.337	29.762	32.562	36.274	48.134
MOY_001	34_750_1	140357	304687	Weighted	1.02	P	GL	1.116	1.486	193.203	229.260	254.110	279.849	316.928	348.078	388.391	517.223

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 (m3/s) by AEP(%)							
						Single- site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
				34004 and 34031													
MOY_001a	34_1456_2	140335	304734	Weighted 34004 and 34031	1.00	P	GL	1.115	1.484	171.676	203.077	224.452	246.396	277.701	303.755	338.807	450.764
MOY_002	34_807_1	140451	304446	Weighted 34004 and 34031	1.03	P	GL	1.116	1.487	206.826	245.426	272.028	299.582	339.275	372.621	415.873	554.140
MOY_003	34_761_3	139131	302329	Weighted 34004 and 34031	1.03	P	GL	1.115	1.481	196.384	234.758	261.947	290.676	332.975	369.263	411.605	546.787
MOY_004	34_793_3	134876	301555	Weighted 34004 and 34031	1.04	P	GL	1.112	1.466	173.362	207.448	231.810	257.716	296.121	329.289	366.009	482.890
MOY_005	34_820_1	134800	301481	Weighted 34004 and 34031	1.04	P	GL	1.112	1.467	179.730	215.067	240.325	267.181	306.997	341.384	379.494	500.821
MOY_006	34_788_3	131475	300211	Weighted 34004 and 34031	1.05	P	GL	1.110	1.461	171.656	205.407	229.530	255.180	293.207	326.049	362.015	476.370
MOY_007	34_1462_3	129636	298412	Weighted 34004 and 34031	1.06	P	GL	1.111	1.463	185.072	220.438	245.544	272.110	311.283	344.941	383.127	504.593
MOY_008	34_1463_1	129487	298492	Weighted 34004 and 34031	1.14	SS	G	1.112	1.470	240.596	284.385	315.662	345.496	384.232	413.344	459.748	607.653
MOY_009	34_1450_2	127403	299247	34004	1.15	SS	G	1.112	1.469	231.099	273.159	303.202	331.858	369.065	397.028	441.540	583.396
MOY_009a	34_1450_1	127575	299304	34004	1.15	SS	G	1.112	1.470	231.563	273.707	303.810	332.524	369.805	397.824	442.445	584.654
MOY_010	34_1935_2	125918	302369	34003	1.05	SS	G	1.112	1.467	211.618	250.132	277.643	303.883	337.954	363.560	404.156	533.476
MOY_011	34_1925_1	125901	302441	34003	1.05	P	GL	1.112	1.469	178.197	213.298	236.367	259.461	291.513	317.489	353.026	466.268
MOY_012	34_3416_1	126514	303916	34003	1.05	P	GL	1.112	1.468	175.999	210.667	233.452	256.261	287.917	313.573	348.596	460.173

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 (m3/s) by AEP(%)							
						Single-site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
MOY_012a	34_3416_2	126849	304896	34003	Gauged QMED	SS	G	1.112	1.467	175.790	211.310	234.750	257.290	286.500	308.310	345.140	463.390
MOY_013	34_1990_1	127084	306257	34003	1.05	P	GL	1.111	1.467	176.868	211.707	234.604	257.526	289.339	315.121	350.249	462.136
MOY_014	34_3551_2	126817	309107	Weighted 34001 and 34003	1.02	P	GL	1.111	1.464	175.389	209.938	232.643	255.373	286.920	312.487	347.140	457.445
MOY_015	34_737_4	124972	312303	Weighted 34001 and 34003	1.00	P	GL	1.111	1.462	174.131	208.432	230.974	253.541	284.862	310.245	344.554	453.728
MOY_016	34_1107_1	125127	316327	Weighted 34001 and 34003	0.99	P	GL	1.110	1.461	172.182	206.098	228.388	250.703	281.673	306.772	340.614	448.273
MOY_017	34_1188_1	124390	317649	Weighted 34001 and 34003	0.98	P	GL	1.110	1.460	171.181	204.901	227.061	249.246	280.036	304.989	338.546	445.267
MOY_018	34_3982_2	124666	318761	34001	0.98	P	GL	1.110	1.460	171.638	205.448	227.668	249.912	280.784	305.804	339.457	446.485
MUL_001	34_356_1	147857	300006	Regional	1.00	P	GL	1.125	1.528	11.703	14.592	16.553	18.564	21.427	23.806	26.781	36.376
MUL_002	34_357_1	147603	300698	Regional	1.00	P	GL	1.124	1.526	12.476	15.556	17.648	19.791	22.843	25.379	28.537	38.718
MUL_003	34_357_3	147736	301660	Regional	1.00	P	GL	1.124	1.522	12.534	15.628	17.729	19.882	22.949	25.496	28.646	38.796
MUL_004	34_355_1	147805	301783	Regional	1.00	P	GL	1.124	1.523	13.537	16.879	19.149	21.475	24.787	27.538	30.947	41.931
MUL_005	34_355_2	147564	302167	Regional	1.00	P	GL	1.123	1.520	12.501	15.597	17.706	19.873	22.965	25.541	28.687	38.819
MUL_006	34_354_1	147530	302237	Regional	1.00	P	GL	1.123	1.521	13.563	16.921	19.209	21.560	24.915	27.709	31.129	42.146
MUL_007	34_354_2	147513	302712	Regional	1.00	P	GL	1.123	1.519	13.526	16.876	19.158	21.502	24.848	27.635	31.031	41.968
MUL_008	34_3_1	147452	302752	Regional	1.00	P	GL	1.123	1.517	19.459	24.172	27.450	30.869	35.832	40.031	44.940	60.742
MUL_009	34_3906_1	146147	303321	Regional	1.00	P	GL	1.122	1.515	20.829	25.873	29.383	33.043	38.355	42.849	48.082	64.921
MUL_010	34_2473_2	144216	304115	Regional	1.00	P	GL	1.121	1.512	21.447	25.745	28.678	31.693	36.004	39.597	44.405	59.870

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 (m3/s) by AEP(%)							
						Single- site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
MUL_011	34_2464_9	140371	304711	Regional	1.00	P	GL	1.120	1.506	22.389	26.875	29.937	33.085	37.584	41.335	46.298	62.240
NPK_001	34_618_1	138795	298449	Regional	1.00	P	GL	1.129	1.547	0.366	0.475	0.554	0.639	0.767	0.879	0.993	1.361
NPK_002	34_618_2	138595	298698	Regional	1.00	P	GL	1.127	1.539	0.551	0.714	0.833	0.961	1.153	1.321	1.489	2.033
NPK_003	34_3461_1	138220	299027	Regional	1.00	P	GL	1.126	1.532	0.771	0.995	1.157	1.330	1.589	1.814	2.042	2.780
NPK_004	34_3461_4	137751	299906	Regional	1.00	P	GL	1.128	1.541	1.166	1.479	1.705	1.947	2.308	2.623	2.958	4.042
QUK_001	34_3524_1	124717	319980			P	GL	1.139	1.591	0.027	0.036	0.042	0.049	0.059	0.069	0.078	0.109
QUK_002	34_3524_1	124819	320350			P	GL	1.139	1.591	0.079	0.104	0.123	0.143	0.174	0.201	0.229	0.320
QUK_003	34_3524_1	125120	320722			P	GL	1.139	1.591	0.465	0.616	0.726	0.844	1.023	1.180	1.343	1.877
QUK_004	34_3524_2	125450	320720			P	GL	1.139	1.591	0.527	0.698	0.822	0.957	1.159	1.337	1.522	2.127
QUR_001	34_151_1	127386	320509			P	GL	1.135	1.575	0.166	0.220	0.259	0.302	0.366	0.423	0.480	0.666
QUR_002	34_151_1	126996	320562			P	GL	1.135	1.575	0.243	0.320	0.376	0.437	0.530	0.613	0.696	0.965
QUR_003	34_151_1	126565	320677			P	GL	1.135	1.575	0.337	0.443	0.521	0.606	0.735	0.850	0.964	1.338
QUR_004	34_151_4	125770	321172			P	GL	1.137	1.581	0.629	0.827	0.972	1.131	1.372	1.586	1.803	2.508
RNN_001	34_1978_1	129190	304858			P	GL	1.131	1.552	0.421	0.540	0.629	0.727	0.876	1.010	1.142	1.567
RNN_002	34_1978_2	128675	305251			P	GL	1.127	1.539	0.651	0.827	0.956	1.096	1.308	1.495	1.686	2.300
RNN_003	34_1978_3	128311	305539			P	GL	1.126	1.532	0.831	1.056	1.221	1.400	1.671	1.910	2.150	2.926
RNN_004	34_1978_6	127211	305726			P	GL	1.124	1.524	1.287	1.603	1.824	2.056	2.394	2.682	3.015	4.087
SAL_001	34_1822_1	114448	289107			P	GL	1.121	1.509	3.481	4.223	4.731	5.256	6.009	6.639	7.441	10.020
SAL_002	34_1557_2	114278	289183			P	GL	1.121	1.508	3.702	4.471	4.996	5.536	6.309	6.954	7.792	10.486
SAL_003	34_619_1	114259	289205			P	GL	1.121	1.508	3.895	4.704	5.257	5.825	6.639	7.318	8.199	11.033
SAL_004	34_619_4	113831	290060			P	GL	1.121	1.511	4.072	4.971	5.587	6.225	7.140	7.907	8.866	11.949

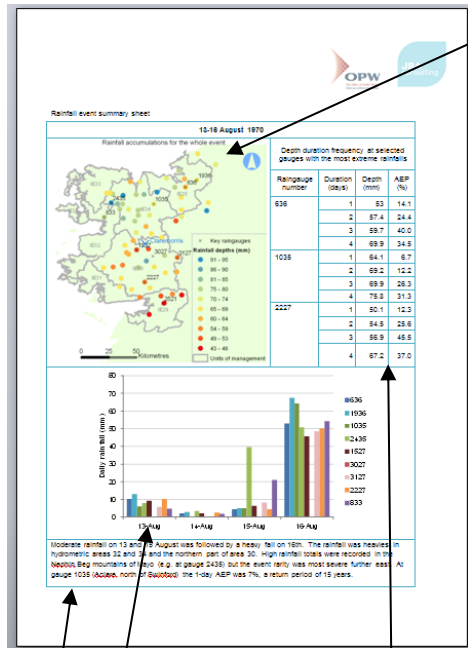
HEP label	FSU node	X	Y	QMED adjustment source (none if blank)	QMED adjustment (none if blank)	Growth curve		FSR Rainfall Runoff ratio applied to 1% AEP peak flow by AEP(%)		Peak Flow (m3/s) by AEP(%)							
						Single- site /Pooled	Distribution	0.5	0.1	50	20	10	5	2	1	0.5	0.1
SAR_001	34_2502_1	149321	302120			P	GL	1.122	1.514	0.154	0.200	0.234	0.269	0.322	0.367	0.412	0.556
SAR_002	34_2502_1	148721	302157			P	GL	1.122	1.514	0.293	0.377	0.437	0.503	0.600	0.685	0.769	1.038
SAR_003	34_2502_2	148235	302083			P	GL	1.122	1.515	0.457	0.582	0.673	0.770	0.915	1.040	1.167	1.576
SAR_004	34_2502_4	147544	302201			P	GL	1.126	1.531	0.782	0.995	1.147	1.308	1.547	1.753	1.973	2.683
SFD_001	34_3462_7	139309	299877	Regional	1.00	P	GL	1.122	1.517	4.255	5.147	5.770	6.420	7.365	8.167	9.167	12.387
SFD_002	34_3462_9	138637	299815	Regional	1.00	P	GL	1.122	1.515	4.383	5.350	6.037	6.765	7.837	8.759	9.828	13.267
SFD_003	34_3462_11	137824	299968	Regional	1.00	P	GL	1.122	1.514	4.600	5.565	6.237	6.940	7.962	8.829	9.905	13.367
SFD_004	34_3463_1	137707	299962	Regional	1.00	P	GL	1.122	1.516	5.577	6.696	7.481	8.304	9.506	10.529	11.817	15.963
SFD_005	34_2446_1	137031	300379	Regional	1.00	P	GL	1.122	1.517	6.006	7.211	8.056	8.943	10.237	11.339	12.728	17.197
SFD_006	34_2499_5	134849	301491	Regional	1.00	P	GL	1.122	1.513	6.687	7.970	8.830	9.703	10.933	11.946	13.400	18.078
SLN_001	34_619_1	114481	289649			P	GL	1.121	1.508	0.101	0.129	0.149	0.171	0.204	0.233	0.261	0.351
SLN_002	34_619_1	114266	289197			P	GL	1.121	1.508	0.185	0.234	0.269	0.307	0.362	0.410	0.459	0.618
SON_001	34_2_5	140471	304483			P	GL	1.122	1.516	15.109	17.854	19.717	21.626	24.343	26.600	29.854	40.329
TLG_001	34_667_2	122714	317854			P	GL	1.126	1.536	2.584	3.322	3.864	4.453	5.345	6.132	6.908	9.417
TLG_002	34_667_4	123228	317588			P	GL	1.126	1.533	2.724	3.503	4.075	4.696	5.638	6.468	7.282	9.913
TLG_003	34_193_1	123338	317610			P	GL	1.127	1.538	3.173	4.063	4.713	5.416	6.478	7.410	8.351	11.394
TLG_004	34_1106_3	124349	317576			P	GL	1.128	1.543	4.279	5.473	6.312	7.193	8.482	9.582	10.809	14.785

G Analysis of rainfall data

Introduction to Rainfall event summary sheets

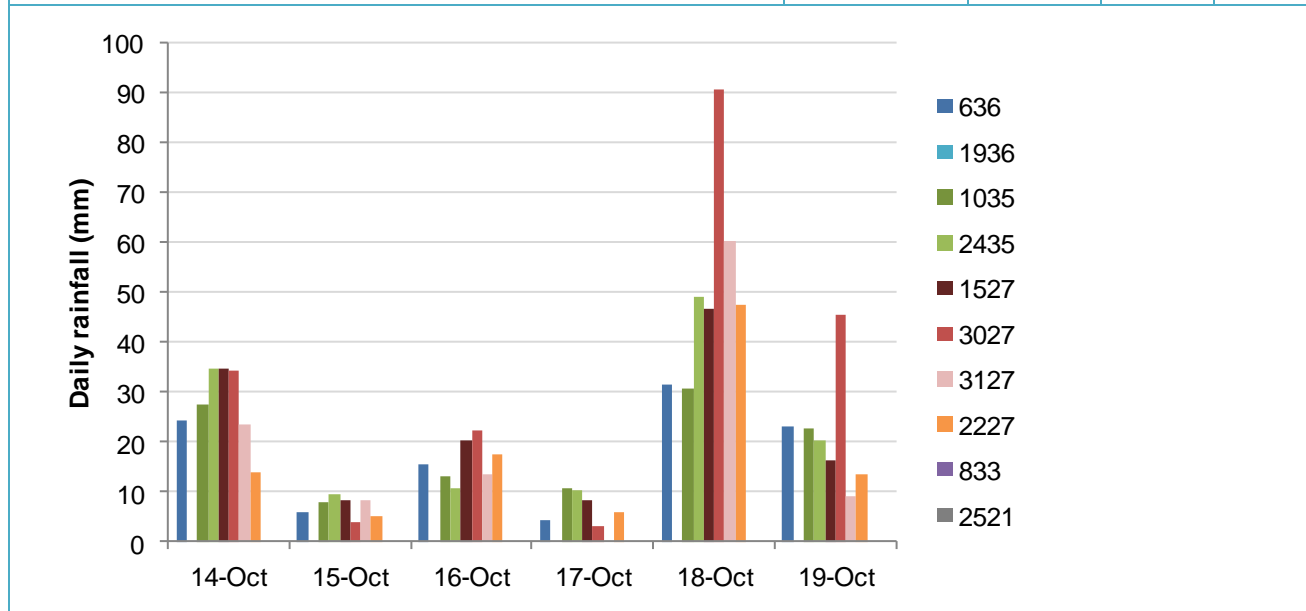
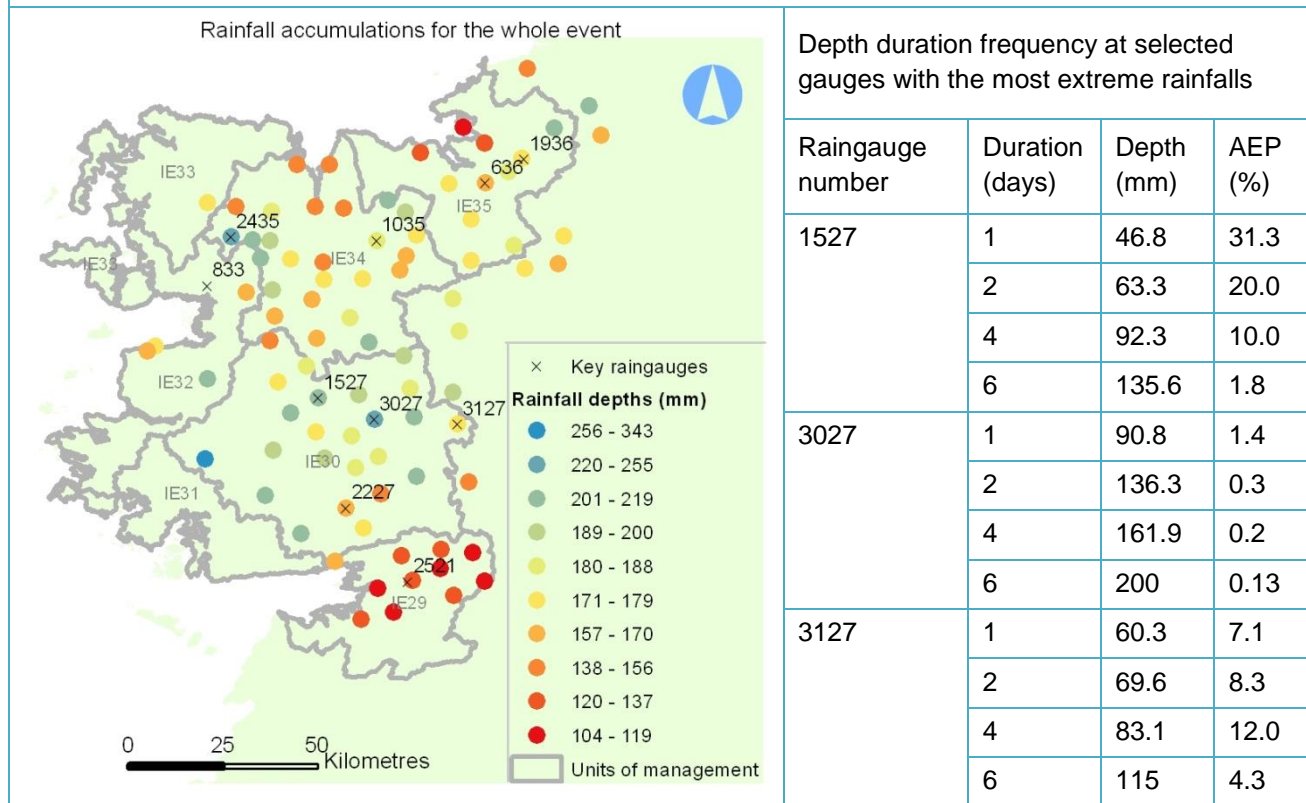
This appendix provides results from analysis of rainfall events. Most of the analysis has been carried out using daily rainfall data as there are very few sub-daily gauges in the study area. However, some more simplified sheets show analysis of sub-daily data to aid in understanding the characteristics of short-duration rainfall events.

Information provided in the summary sheets



Rainfall event summary sheet

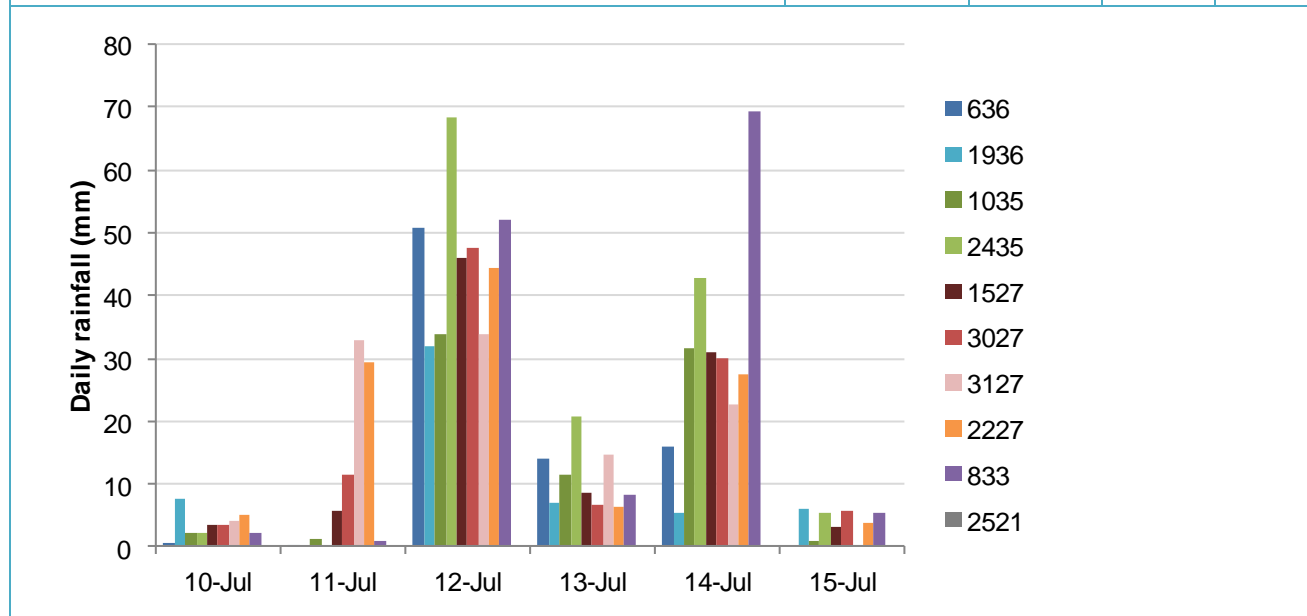
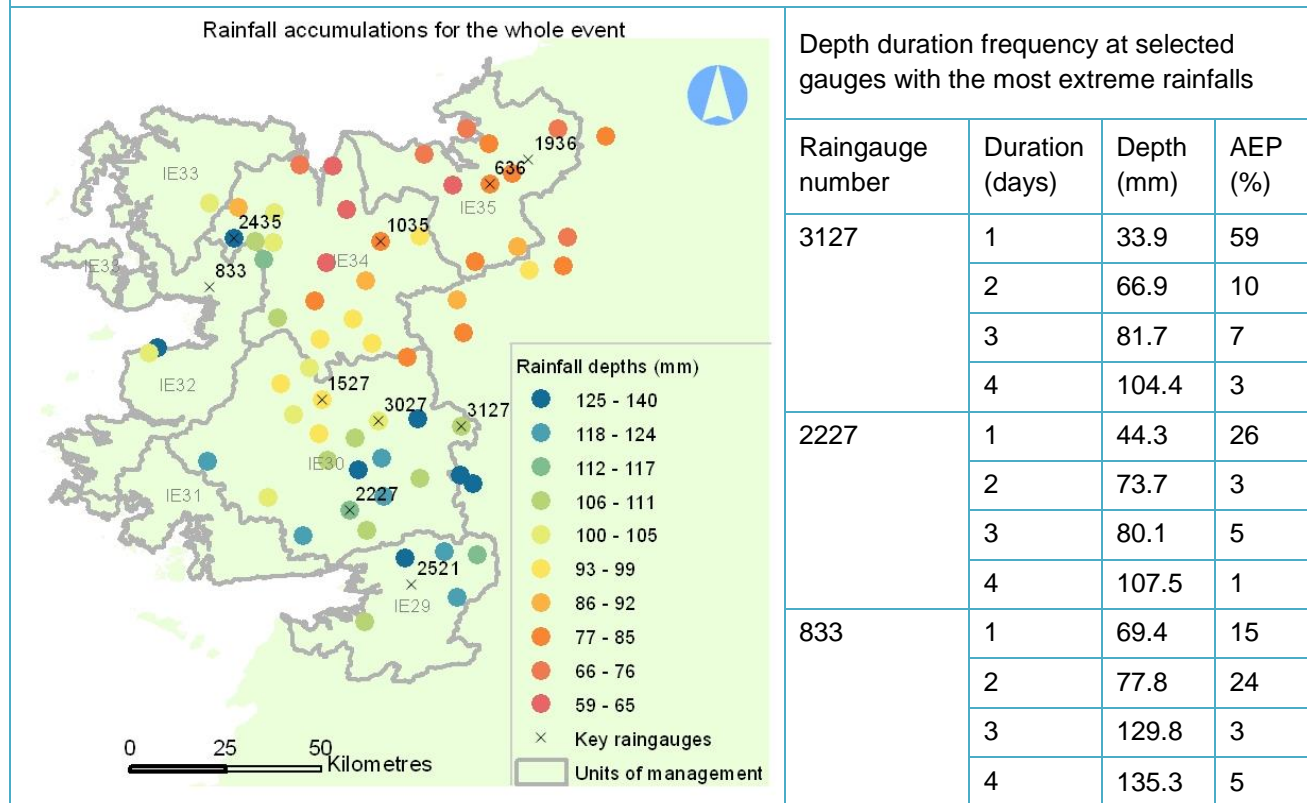
14 to 19 October 1954



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

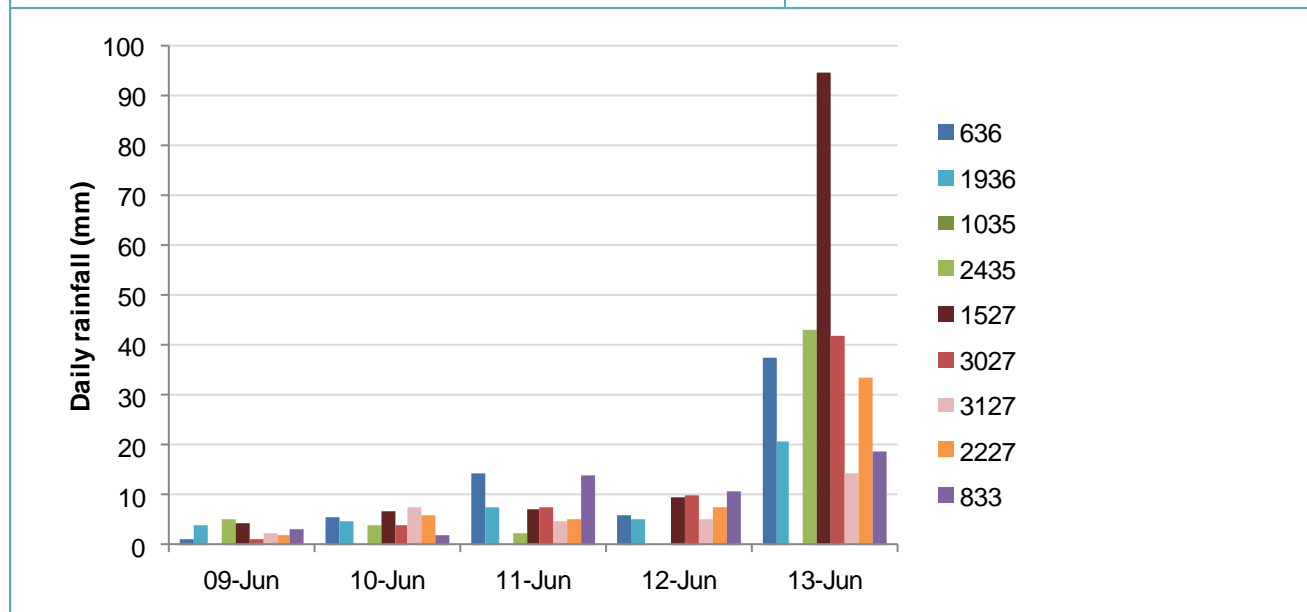
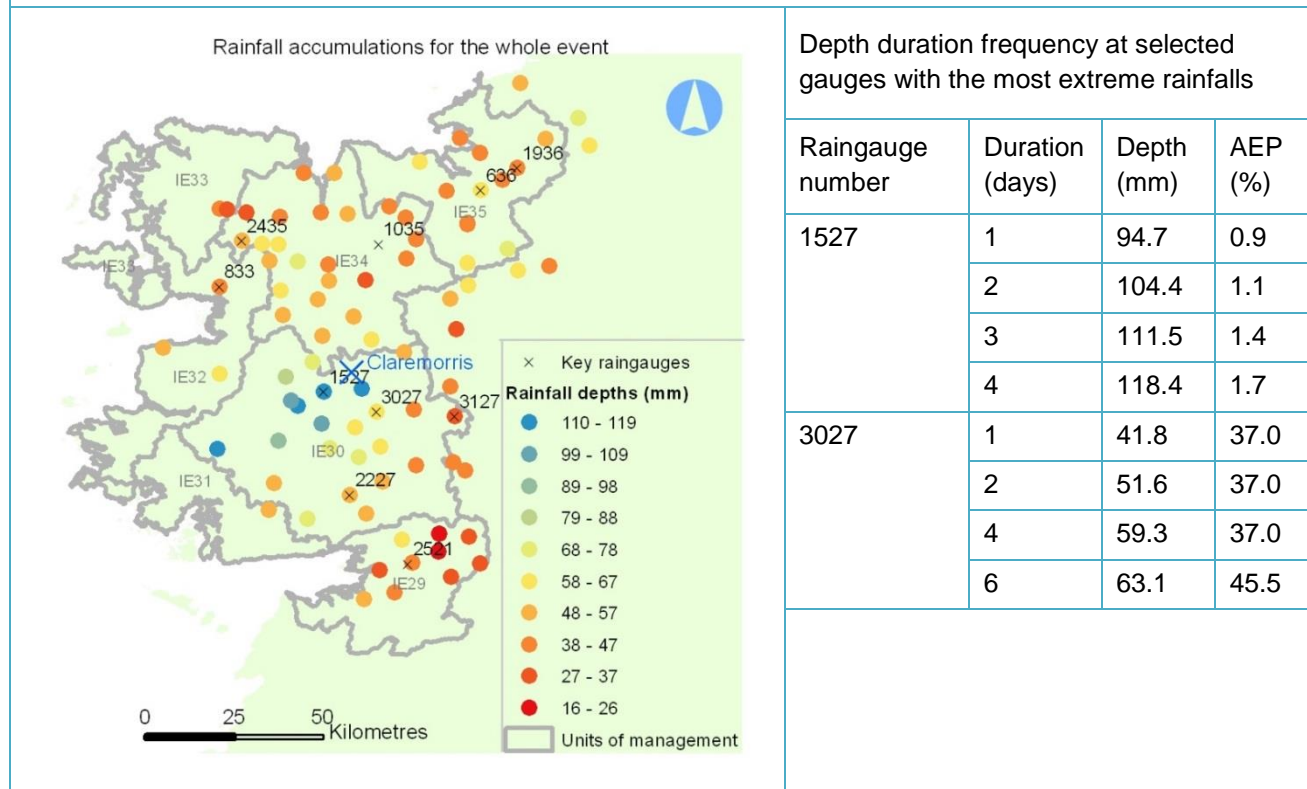
10 to 15 July 1961



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



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.

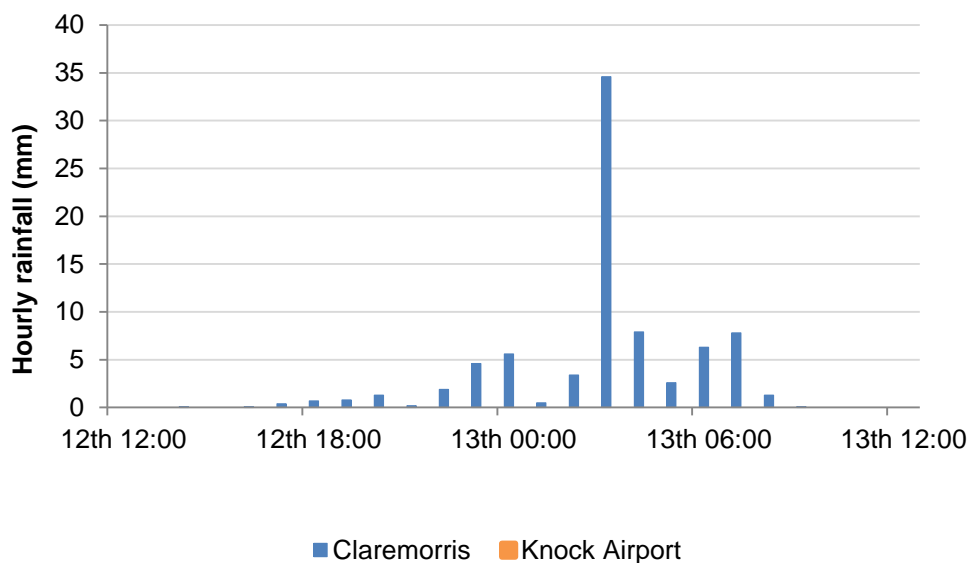
Analysis of hourly rainfall data

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

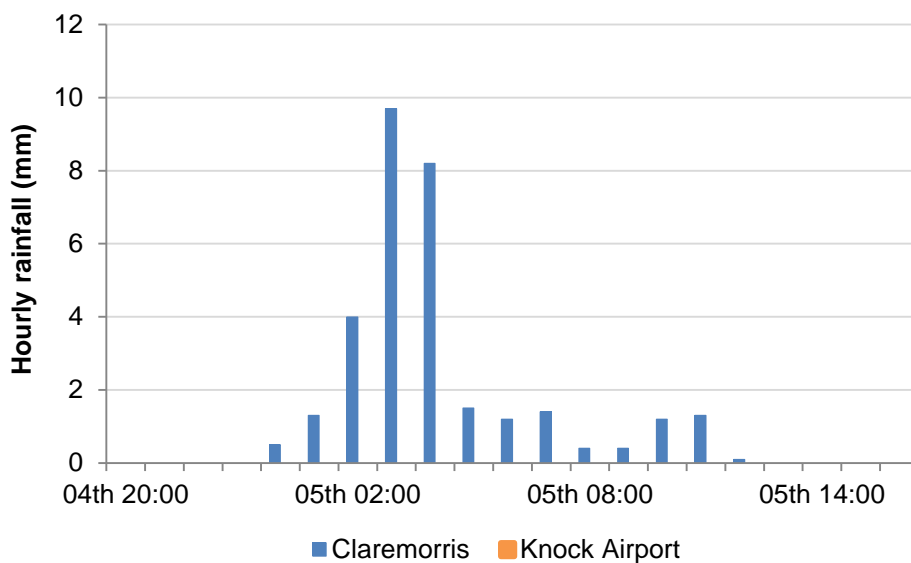
5 October 1964

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

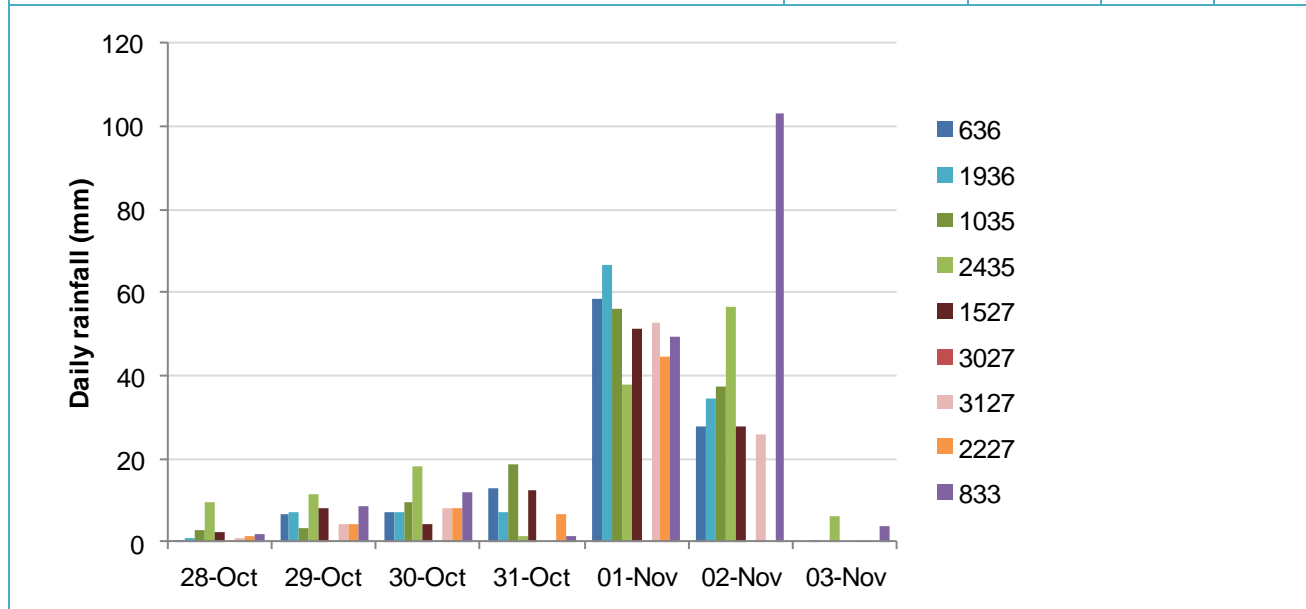
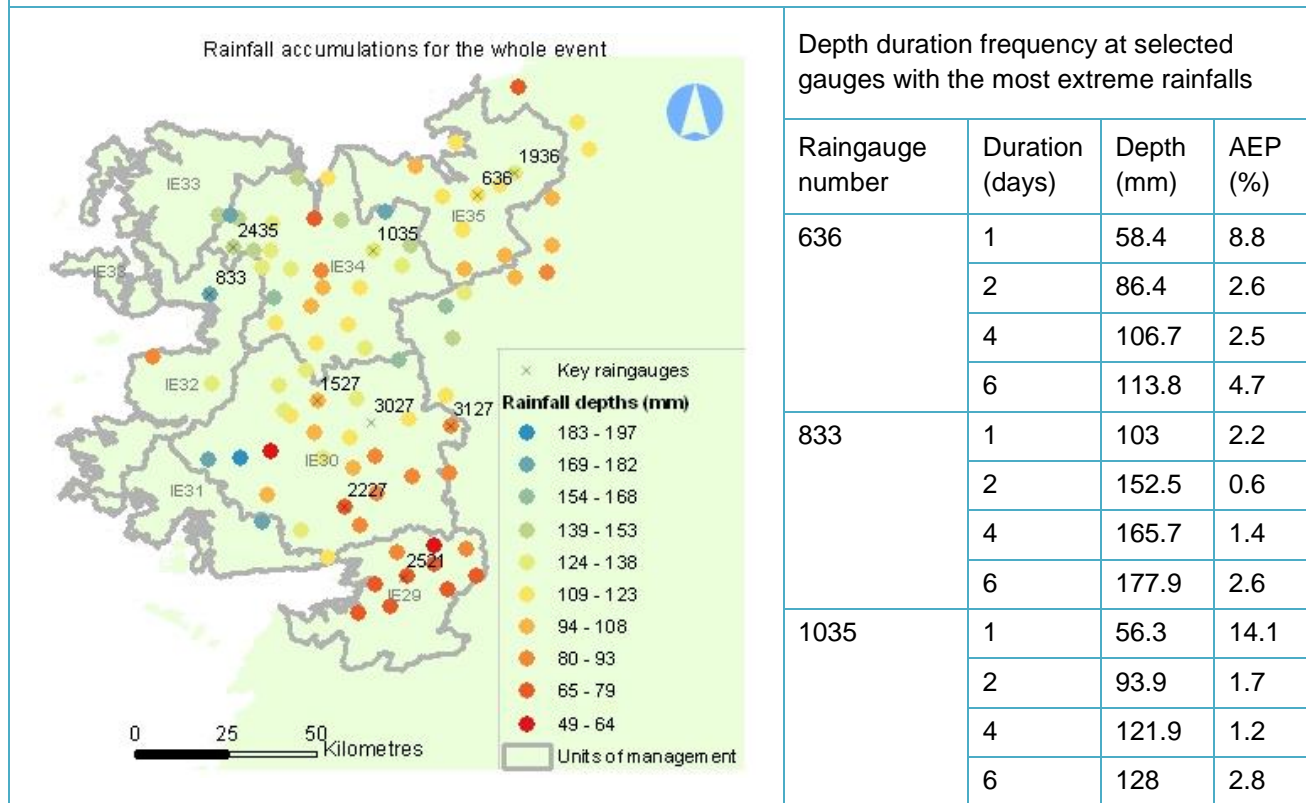
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based 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

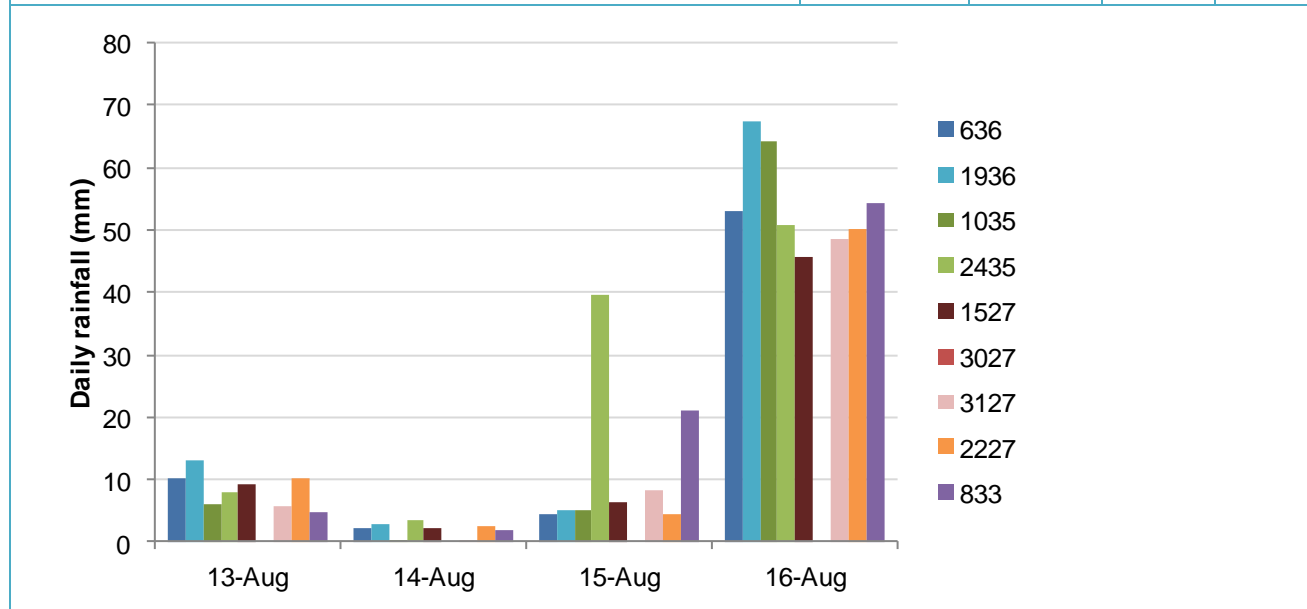
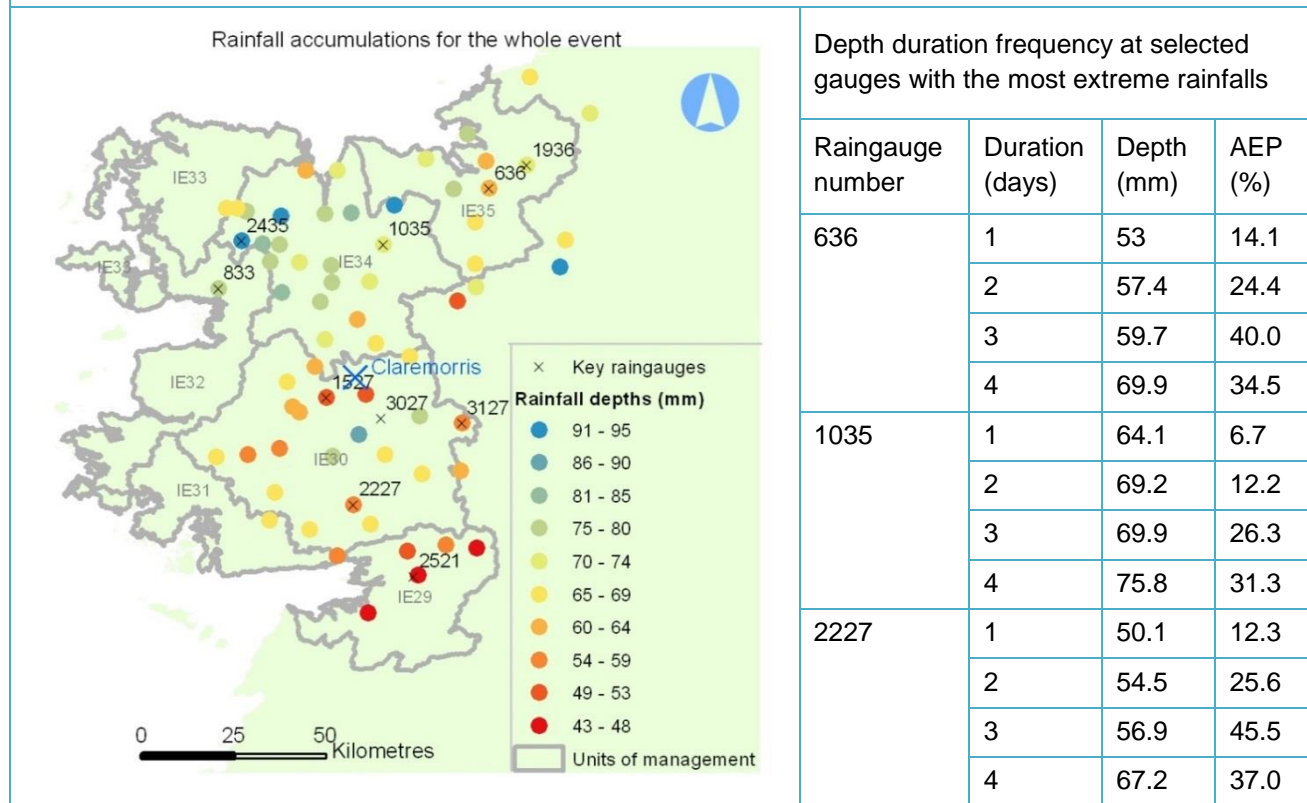
29 October to 2 November 1968



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



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.

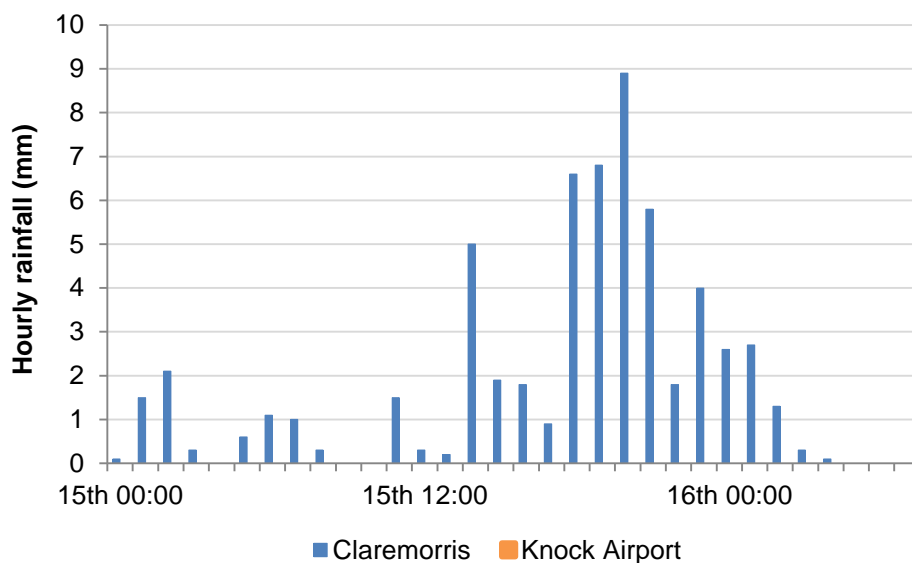
Analysis of hourly rainfall data

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

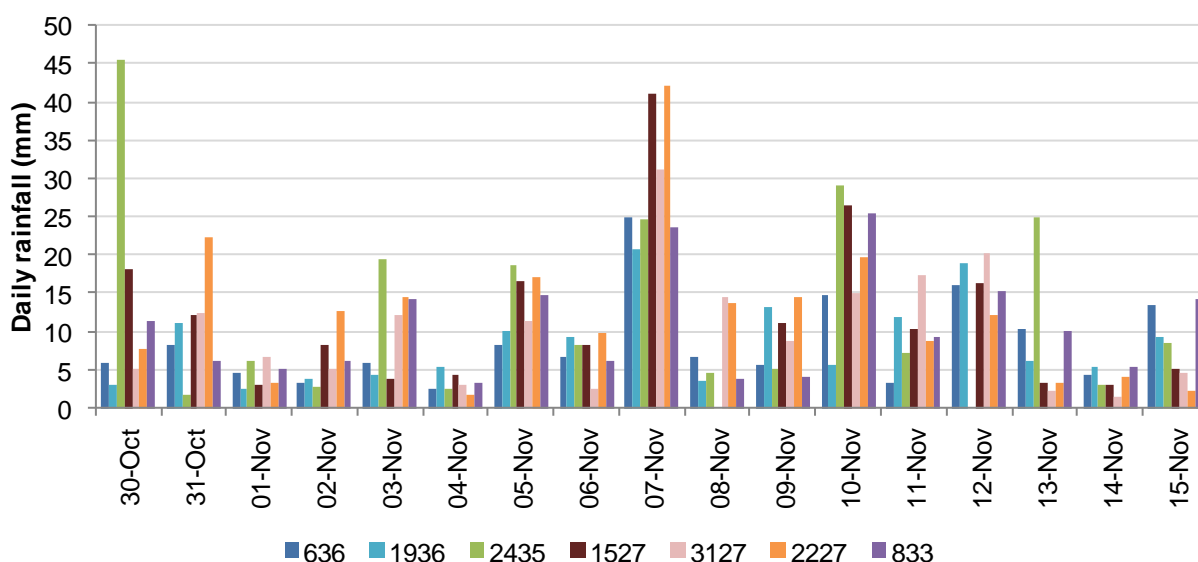
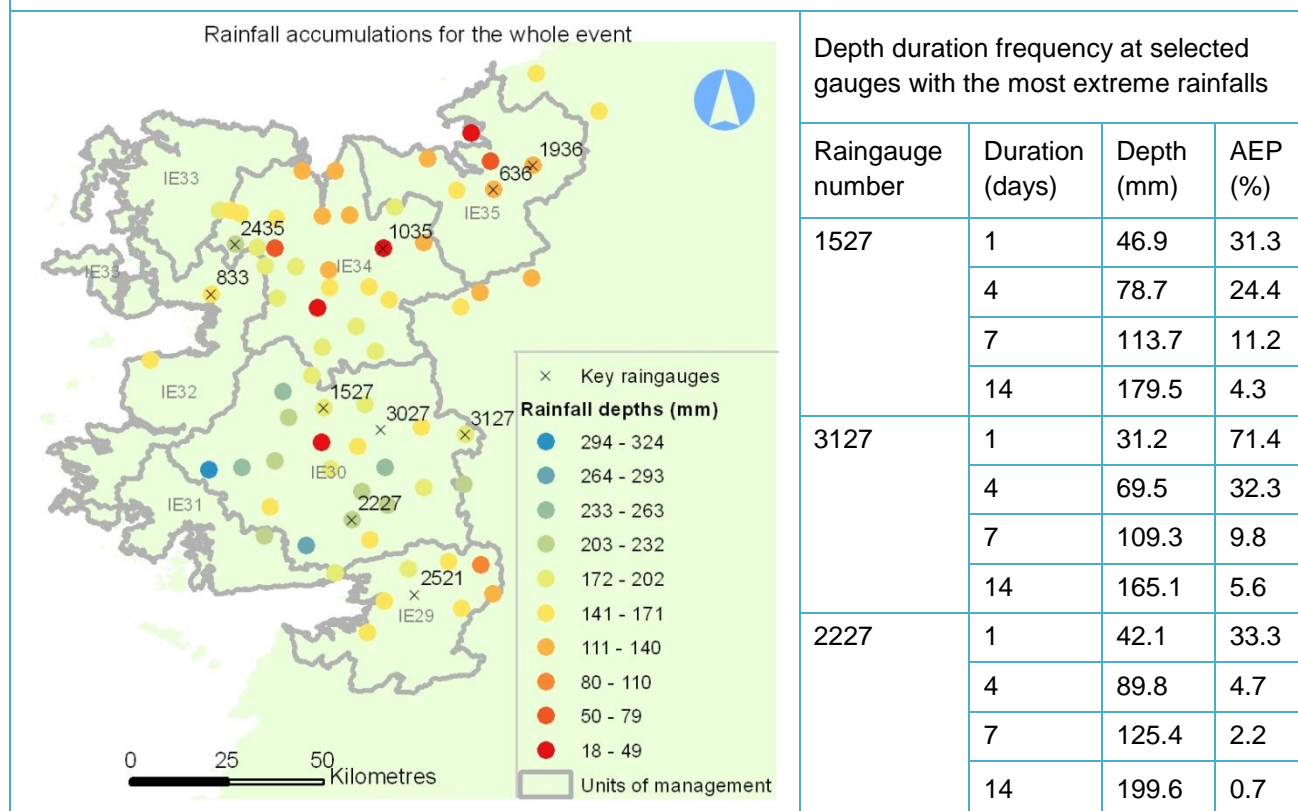
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based 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



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

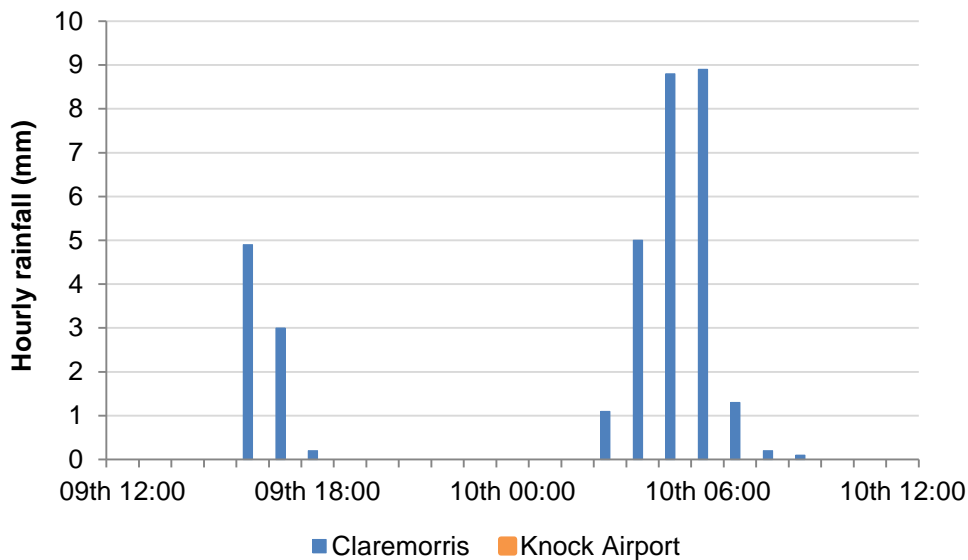
10 September 1981

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

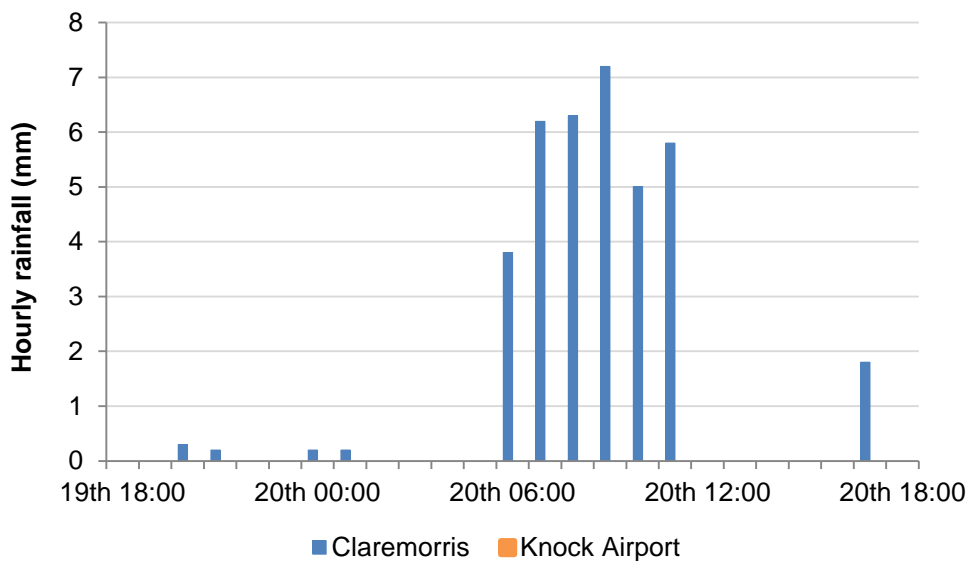
20 August 1987

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

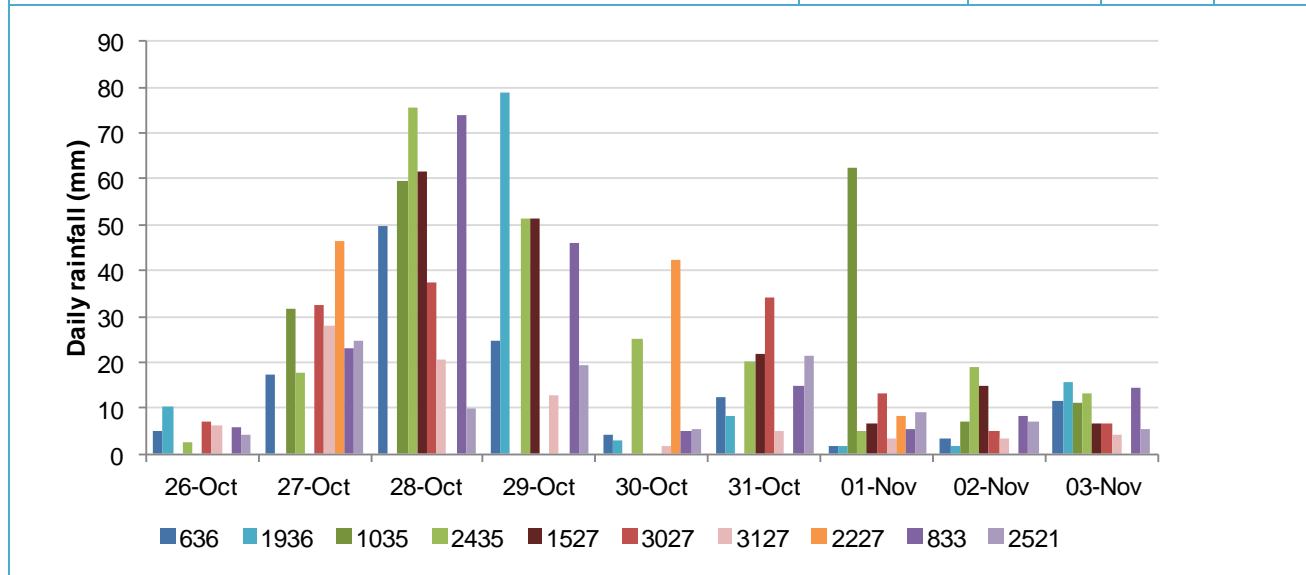
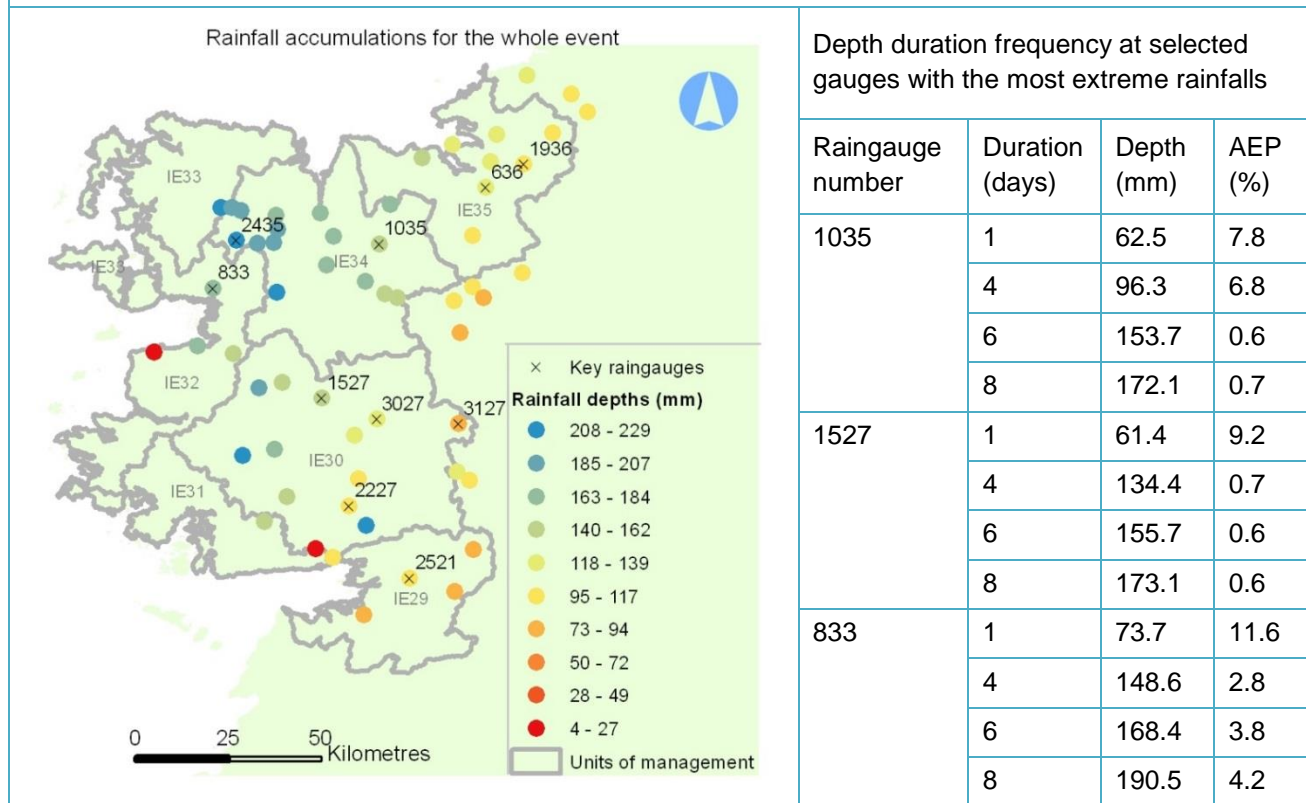
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based 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

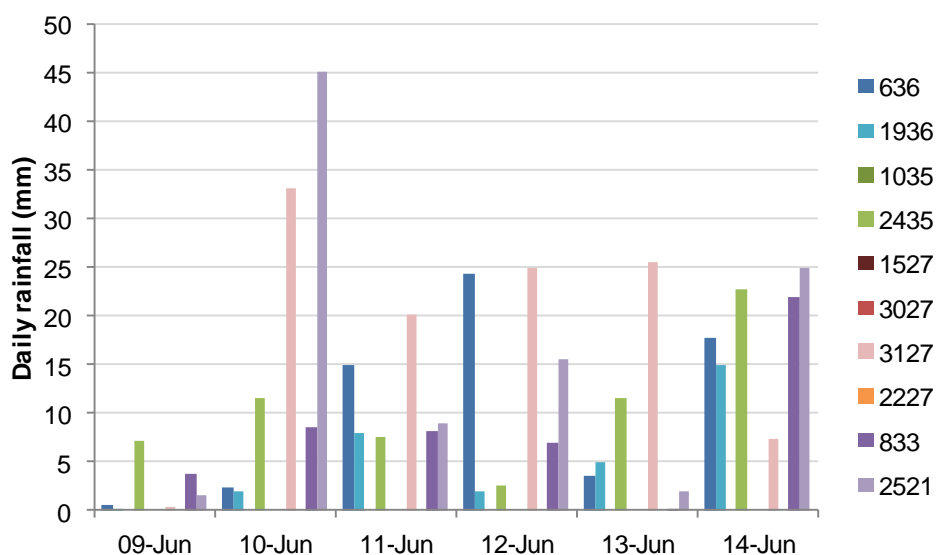
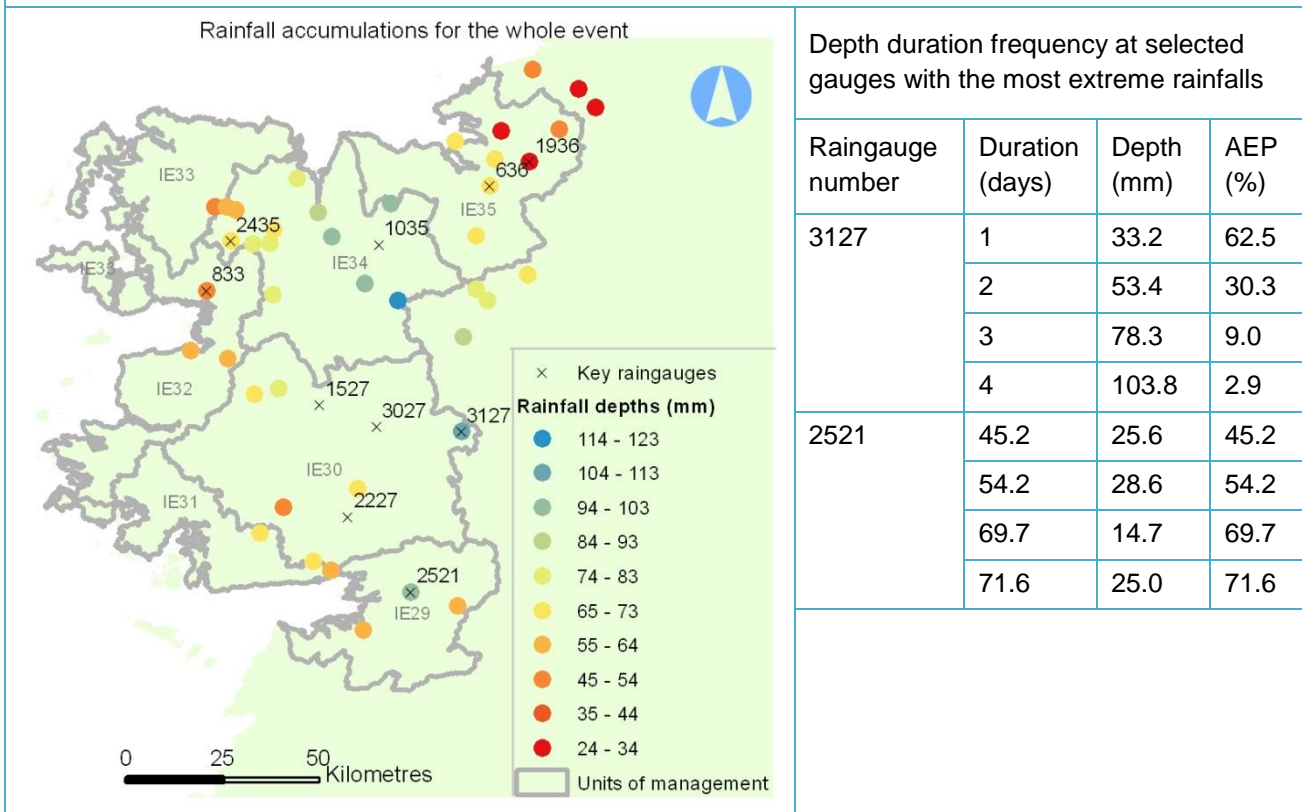
26 October to 2 November 1989



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



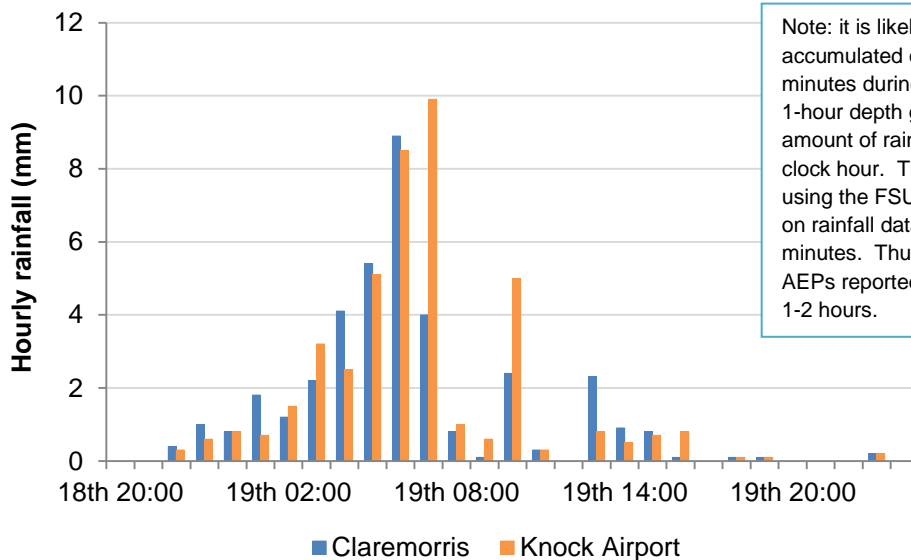
Note that data is missing from several of the key gauges during this event. Rain was caused by a cool northerly airflow due to a depression centred over England and Wales. On 11 June there was very heavy rain in the east midlands and north of Ireland. In the Western RBD, the rainfall over this period 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).

Sub-daily rainfall event summary sheet

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



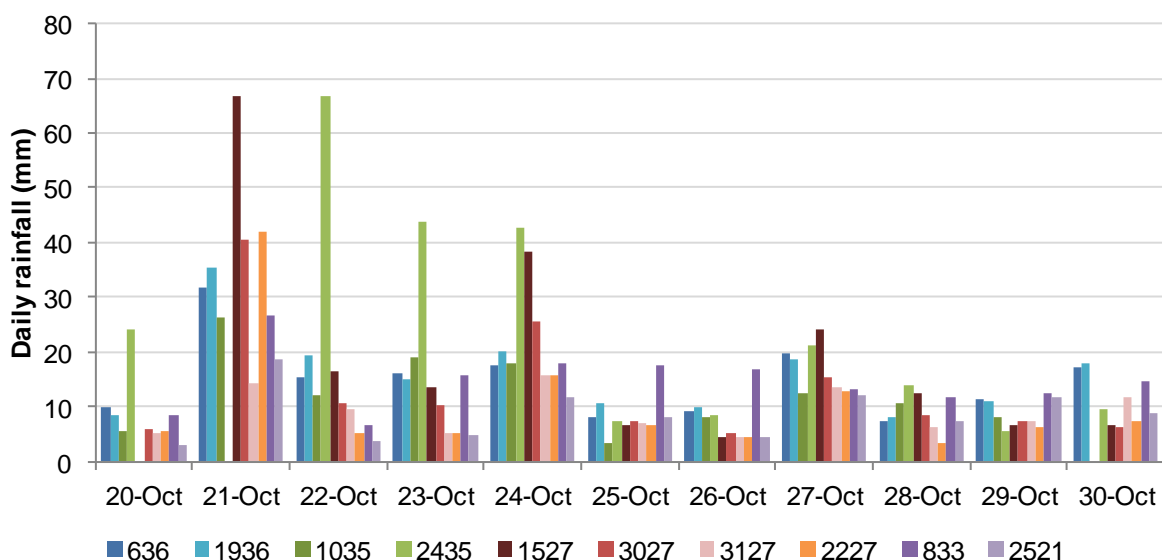
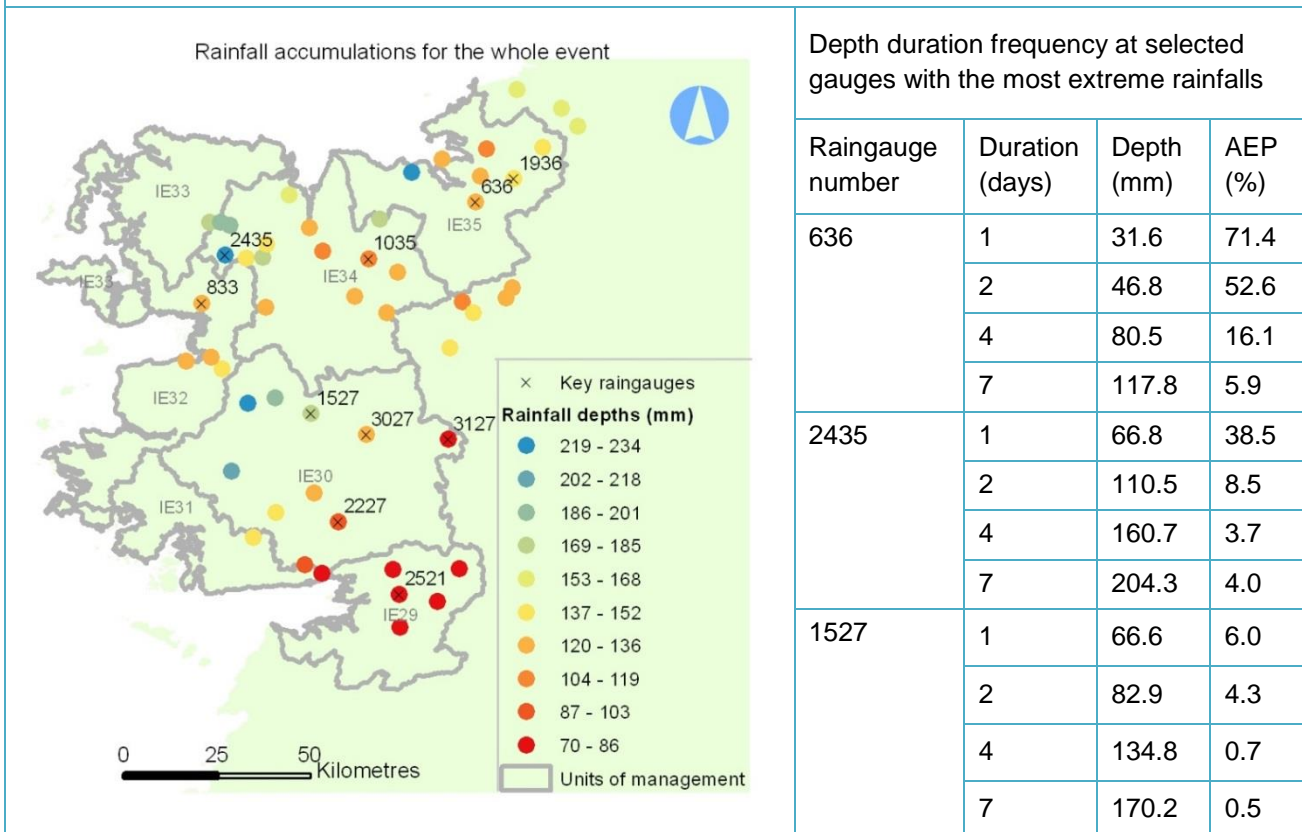
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based 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



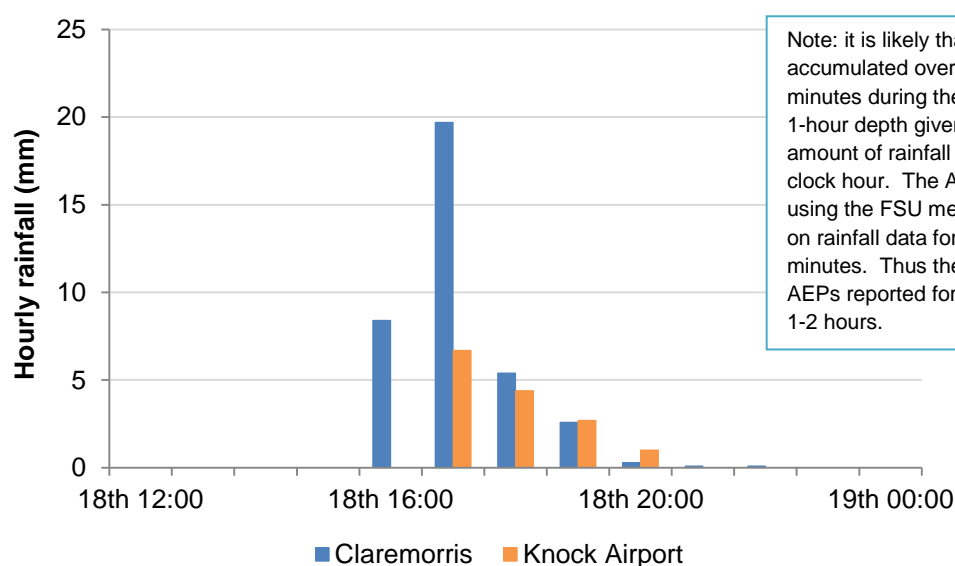
On 20-21 October a deepening depression moved northwards to the west of Ireland bringing heavy frontal rainfall driven by south-easterly gales. There was more widespread and heavier rainfall on 25th. Total October rainfall was near-normal for the western RBD whereas in the SW of Ireland it was the wettest 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

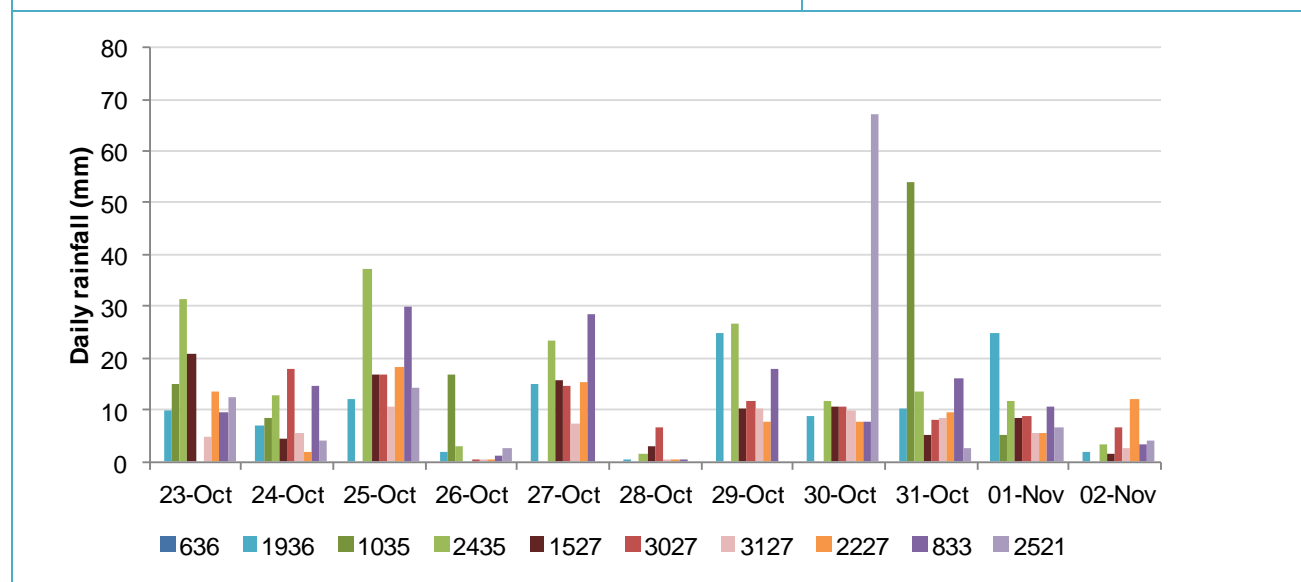
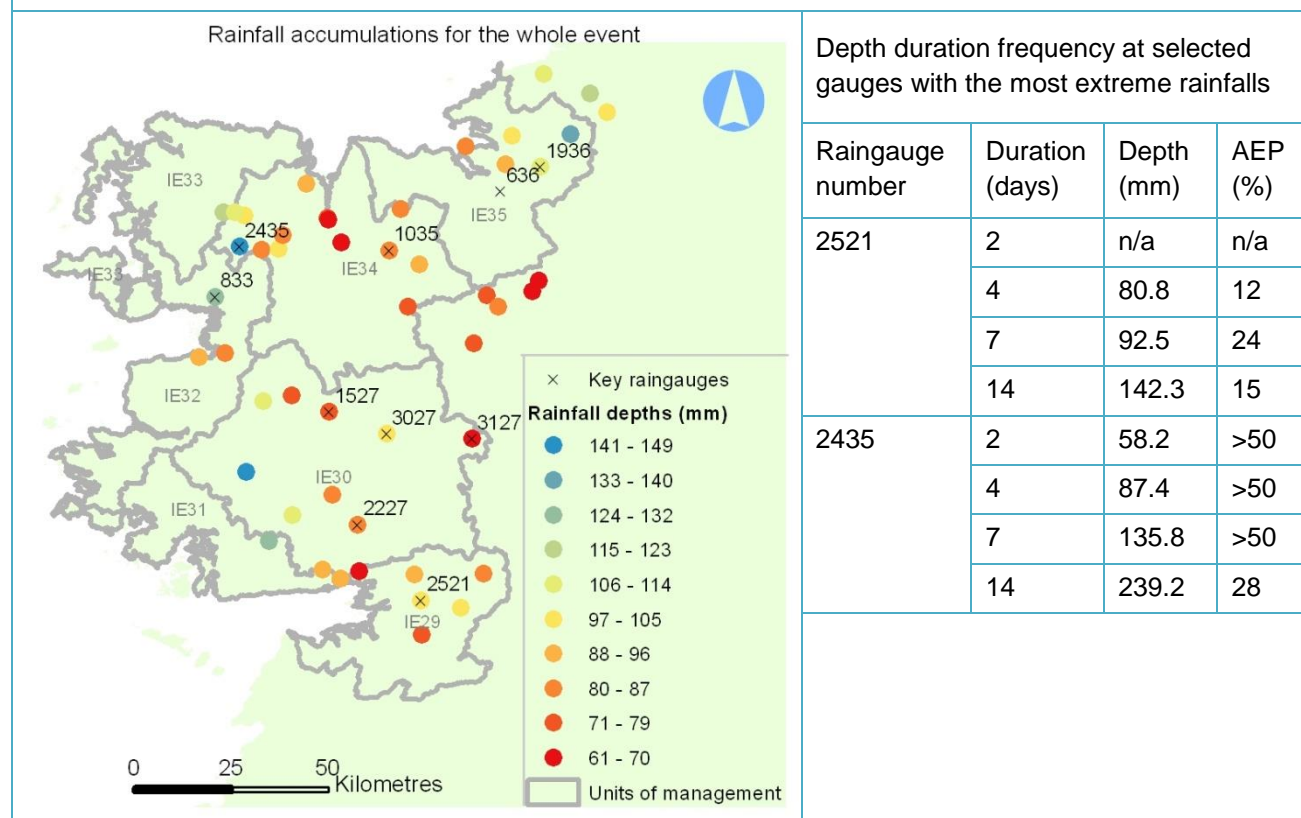


August 2000 was warm and there were frequent thunderstorms between 16th and 21st. On 18th thunder showers were confined to the north-west of Ireland, with temperatures between 16° and 19° C.

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

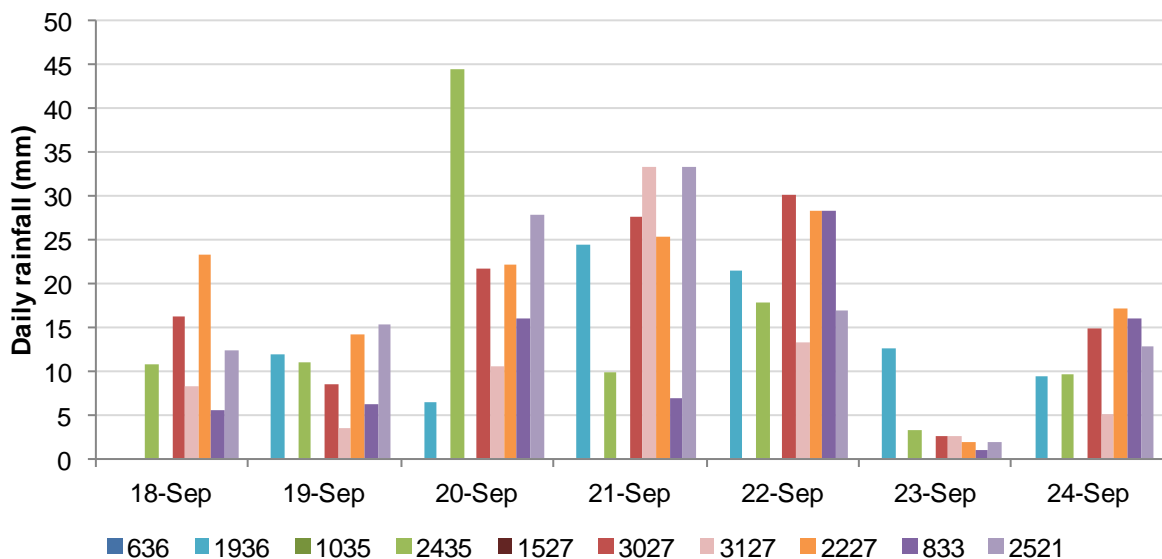
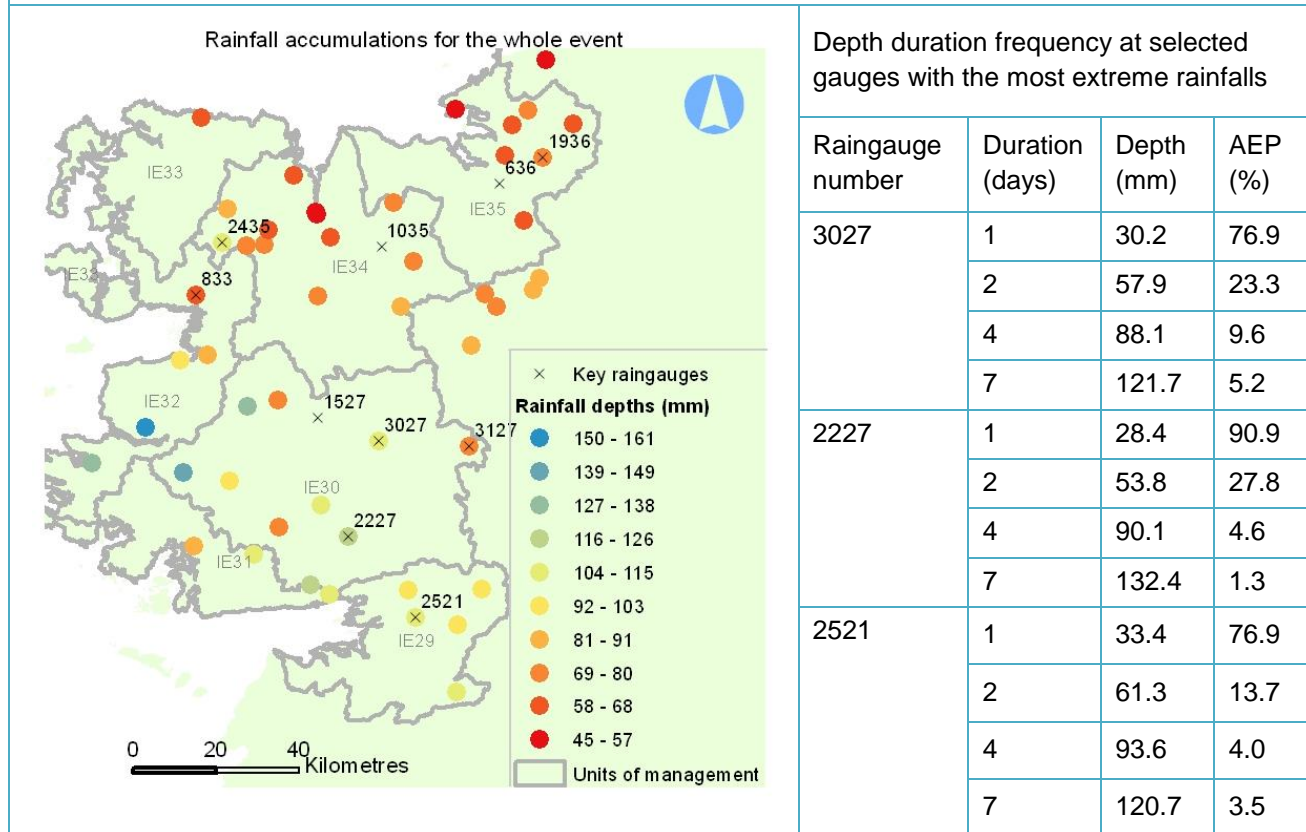


This event affected all of the Western RBD. A succession of Atlantic depressions brought rain almost every day from late August to mid December 2000. The highest totals were observed in late Oct and early Nov, although the event was not particularly severe at any of the key gauges analysed. The lowest 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

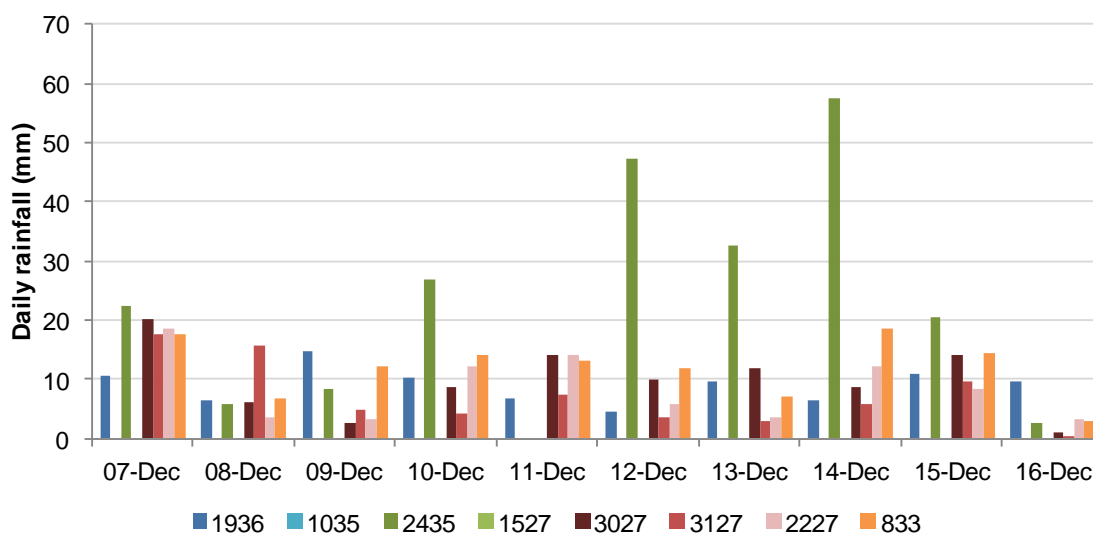
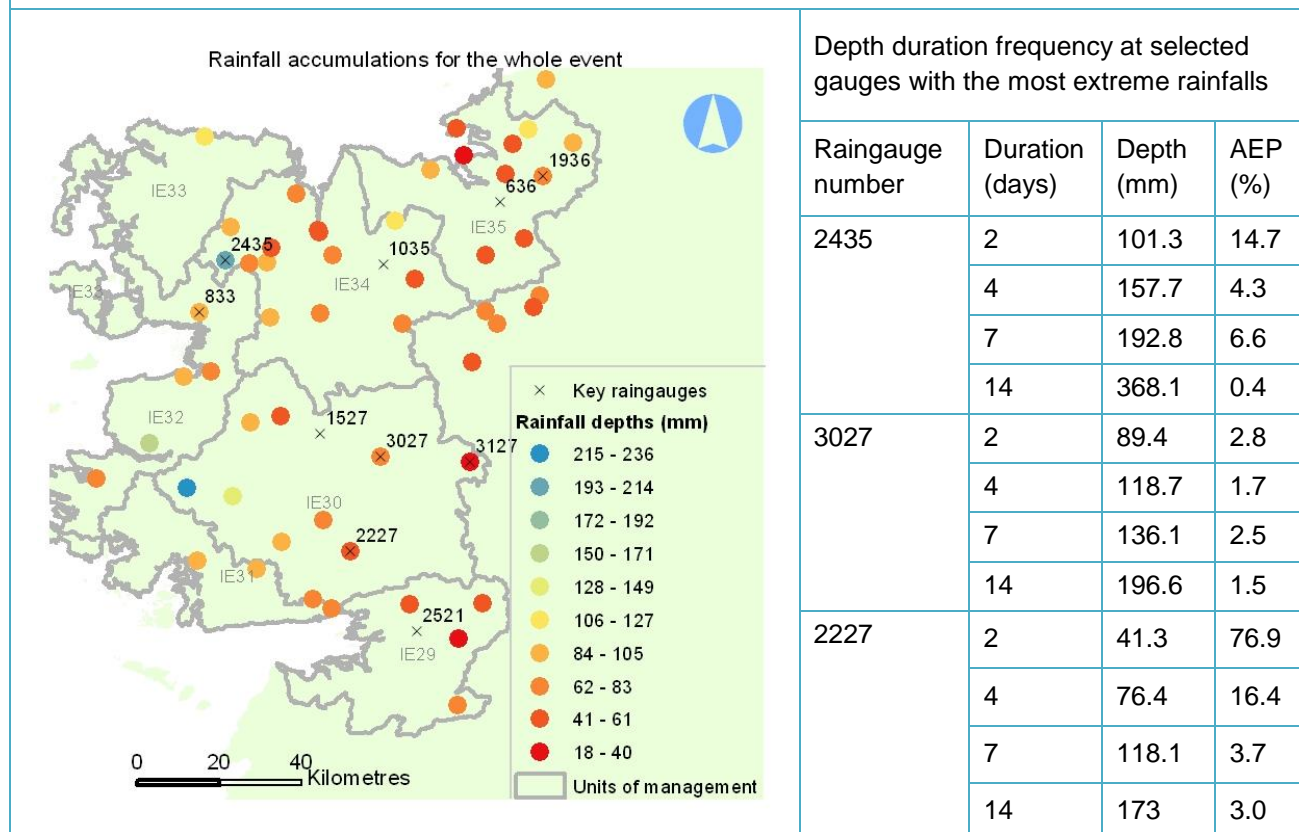
17 to 23 September 2006



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



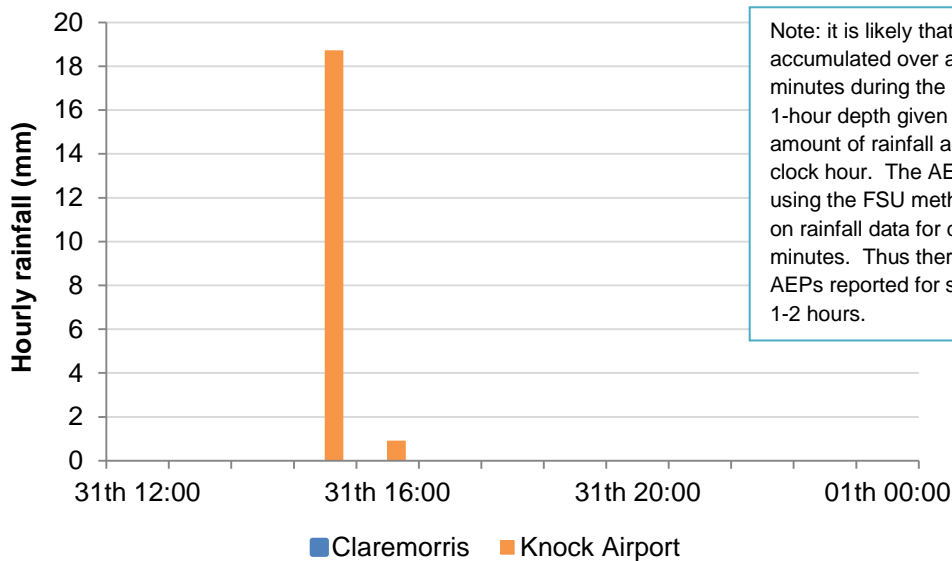
A series of very deep depressions passing to the northwest of Ireland brought rain, accompanied by strong south-westerly winds. There was rain almost every day from 7 November to mid-December. During 9-15 Dec there were exceptionally high totals in the western mountainous areas, particularly at gauge 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



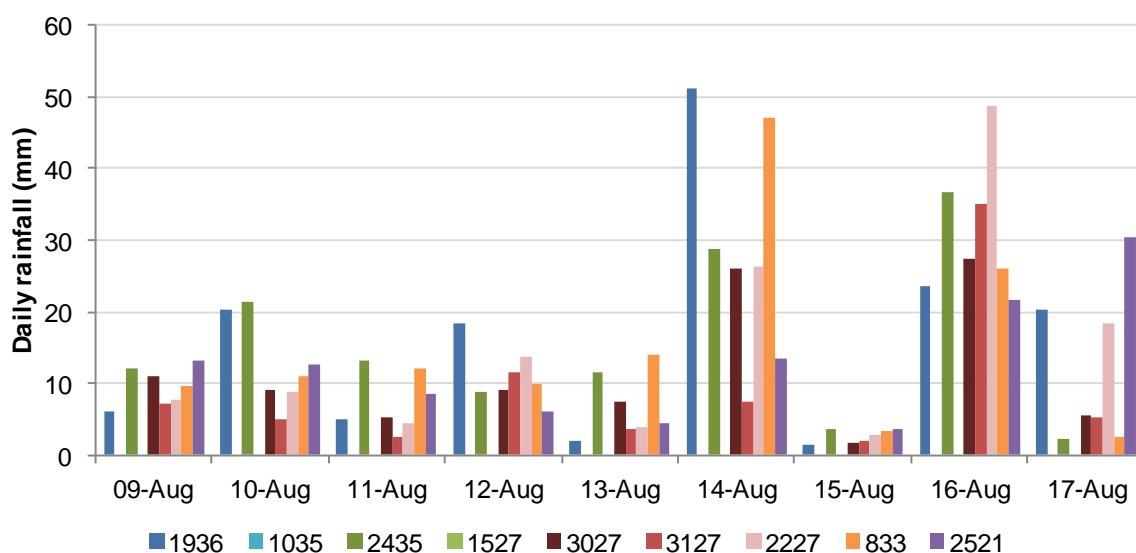
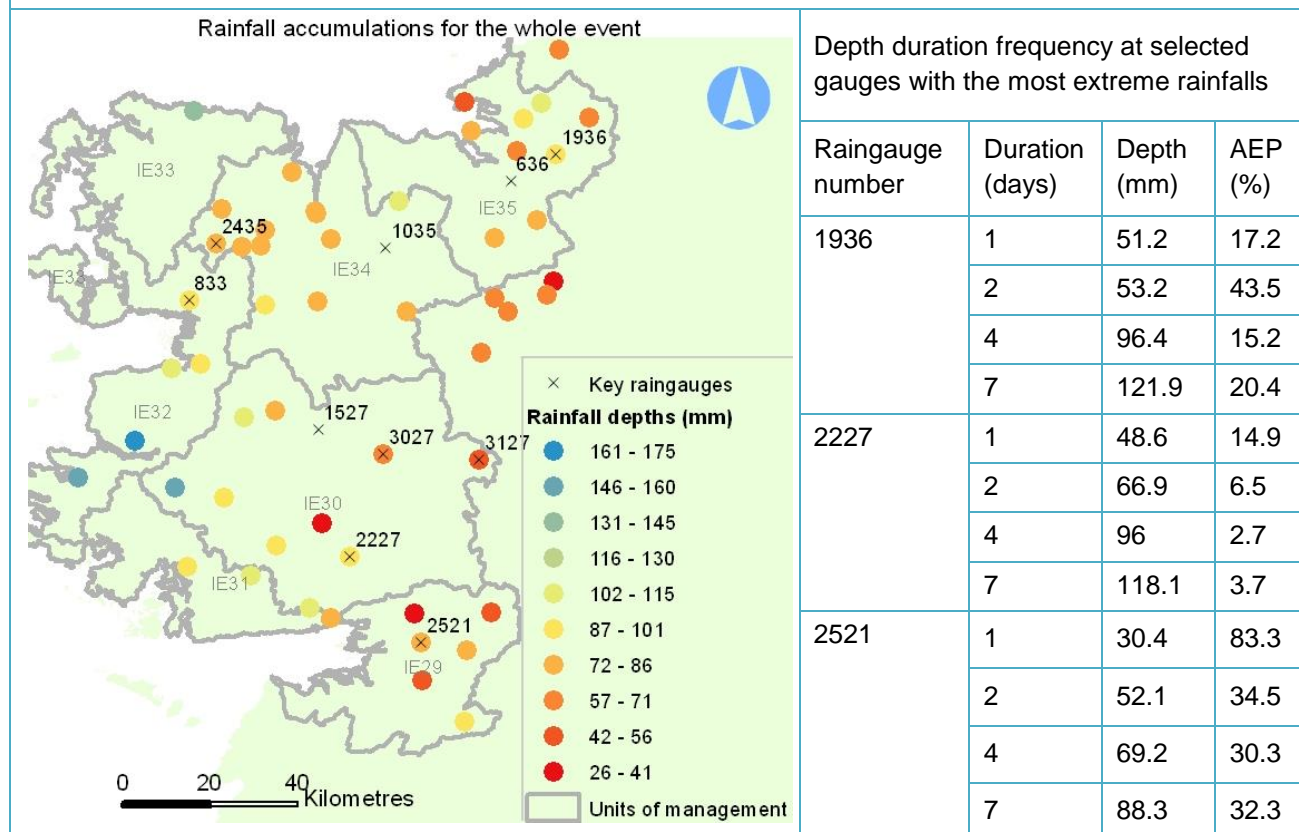
Note: it is likely that the maximum rainfall accumulated over a sliding duration of 60 minutes during the event was higher than the 1-hour depth given here which refers to the amount of rainfall accumulated within each clock hour. The AEPs here are calculated using the FSU methodology which was based 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



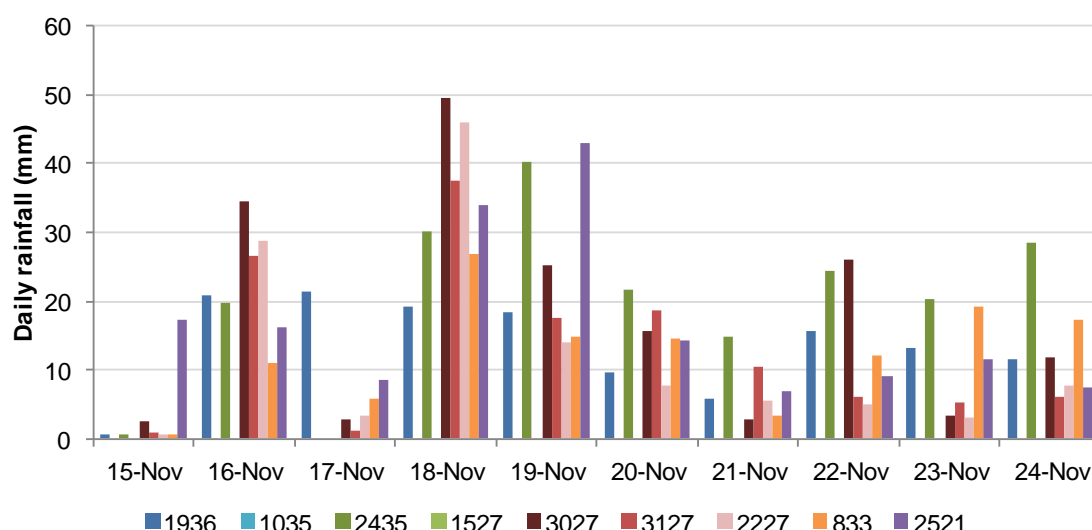
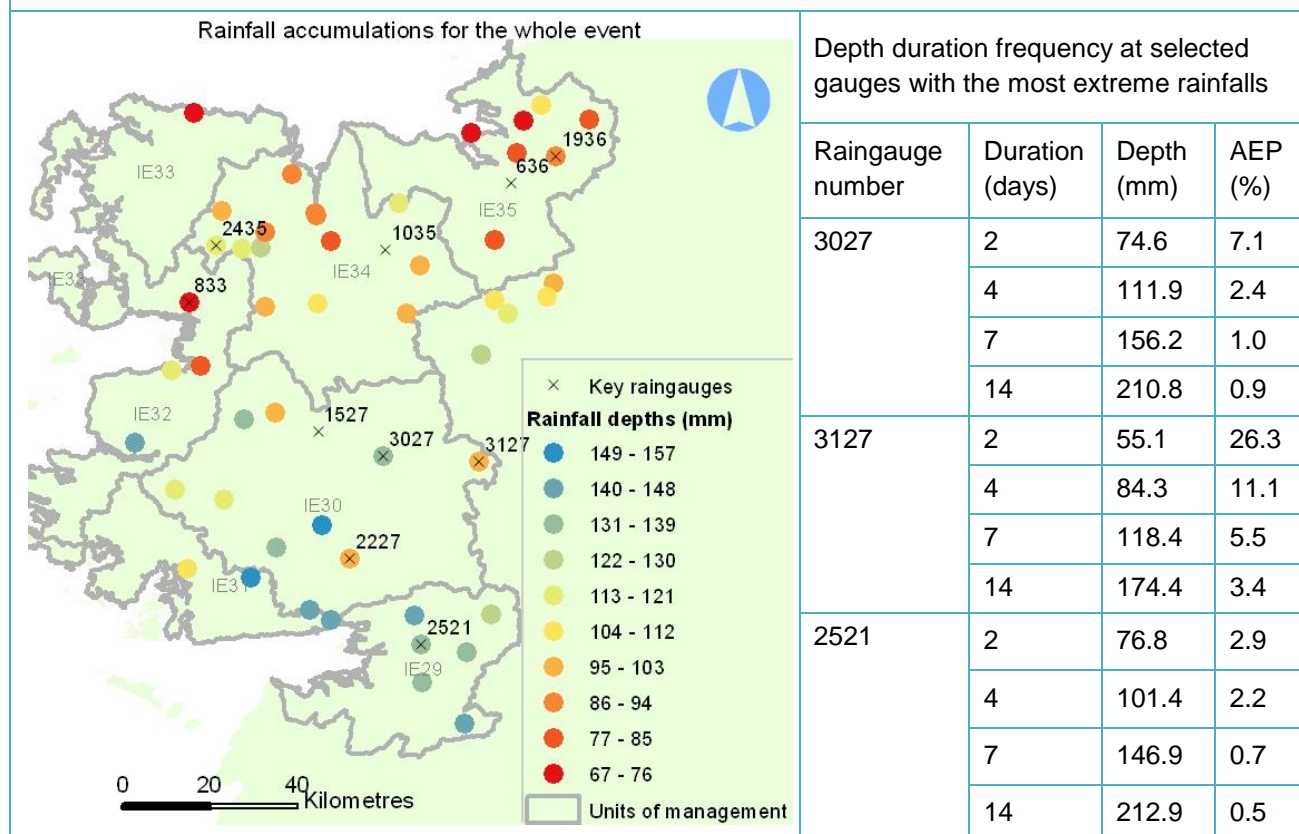
Low pressure close to or over Ireland brought a succession of Atlantic frontal systems across the country, giving some significant falls on 14th and 16th. It was the wettest August in some parts of Ireland. The event affected all of the Western RBD. It was not particularly severe, with an AEP exceeding 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



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.

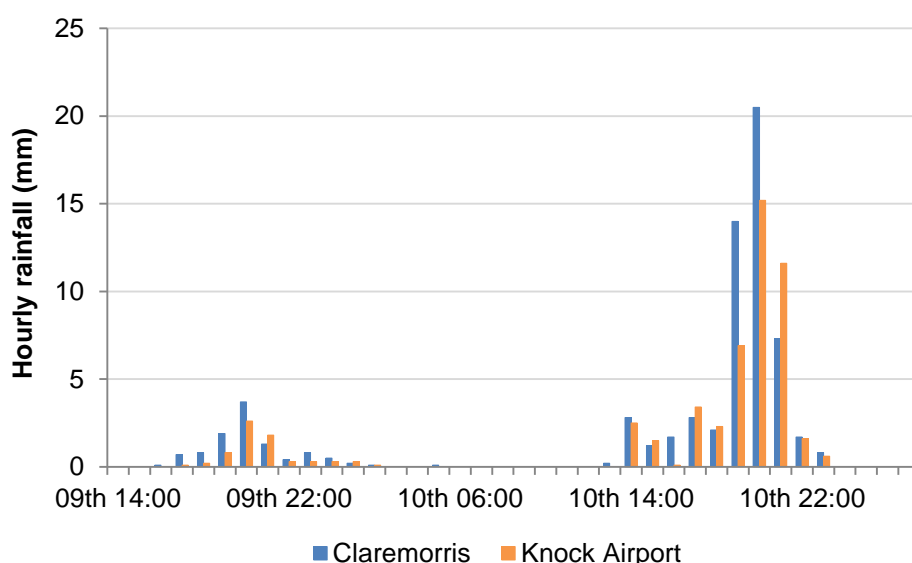
Sub-daily rainfall event summary sheet

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°C 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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