Castleconnell Flood Relief Scheme – Hydrology Report

2019s0927

Final Report

August 2023

www.jbaconsulting.ie

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Revision History

Revision Ref/Date	Amendments	Issued to
7th July 2023 / C01	Published Report	Steering Group
30th August 2023 / C02	Minor amendments	Published publicly

Contract

This report describes work commissioned by Limerick City & County Council as part of the Castleconnell Flood Relief Scheme. Tom Sampson, Orla Hannon and Anastasiya Ilyasova of JBA Consulting carried out this work.

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Purpose

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Abbreviations

Abbreviation	Meaning
AEP	Annual Exceedance Probability
AFA	Area of Further Assessment
AMAX	Annual Maximum
DDF	Depth Duration Frequency
EPA	Environmental Protection Agency
FRS	Flood Relief Scheme
FSU	Flood Studies Update
GIS	Geographical Information System
HEP	Hydrological Estimation Point
LCCC	Limerick City & County Council
mOD	Metres above Ordnance Datum, Malin by default
MPW	Medium Priority Watercourse
OPW	Office of Public Works
PFRA	Preliminary Flood Risk Assessment
CFRAM	Catchment Flood Risk Assessment and Management Study
SFRA	Strategic Flood Risk Assessment

Glossary

Terminology	Meaning
Operational conditions	Set of conditions for operating Ardnacrusha Power Station
Standard operational conditions	Ardnacrusha Power Station functions as standard, namely with the Parteen spillway and four turbines in operation, 345m³/s being the operational maximum intake flow to the turbines from Parteen Basin during flood conditions due to hydraulic constraints in the canal capacity.
Limitations in operational conditions	Two turbines or the spillway and one turbine are not in operation at the Ardnacrusha Power Station, 258m³/s being regulated to the turbine to suit the operational capacity at the station.
"504" Event	AEP Event on Shannon River when Ardnacrusha Power Station functions in standard operational conditions (345m³/s inflow to the turbines), resulting a peak flow of 504m³/s downstream of Parteen Weir during the 1% AEP event. This is the scale of flow experienced in the 2009 event.
Baseline Design Event	Event on Shannon River downstream of Parteen Weir when Ardnacrusha Power Station functions with limitations in operational conditions (258 m³/s regulated from Parteen Basin), resulting a peak flow of 591m³/s downstream of Parteen Weir during the 1% AEP event

1 Introduction

This report is the Final Hydrology Report. The final hydraulic modelling report shall ensure that the documentation covers any feedback from the hydraulic models which influences hydrological assessment.

1.1 Overview

Development of the Castleconnell Flood Relief Scheme requires hydraulic modelling of flood risk to confirm the baseline risk, appraise options and determine the feasibility of a proposed scheme. This hydraulic model requires hydrological analysis to determine the most appropriate inflows to the hydraulic model and consideration of whether flood hydrology influences model downstream boundaries.

The hydrological analysis needs to review the available data that could be used to calibrate the hydraulic modelling.

The level of detail must be appropriate to develop and present a robust scheme for Castleconnell scheme area as identified in Figure 1-1.

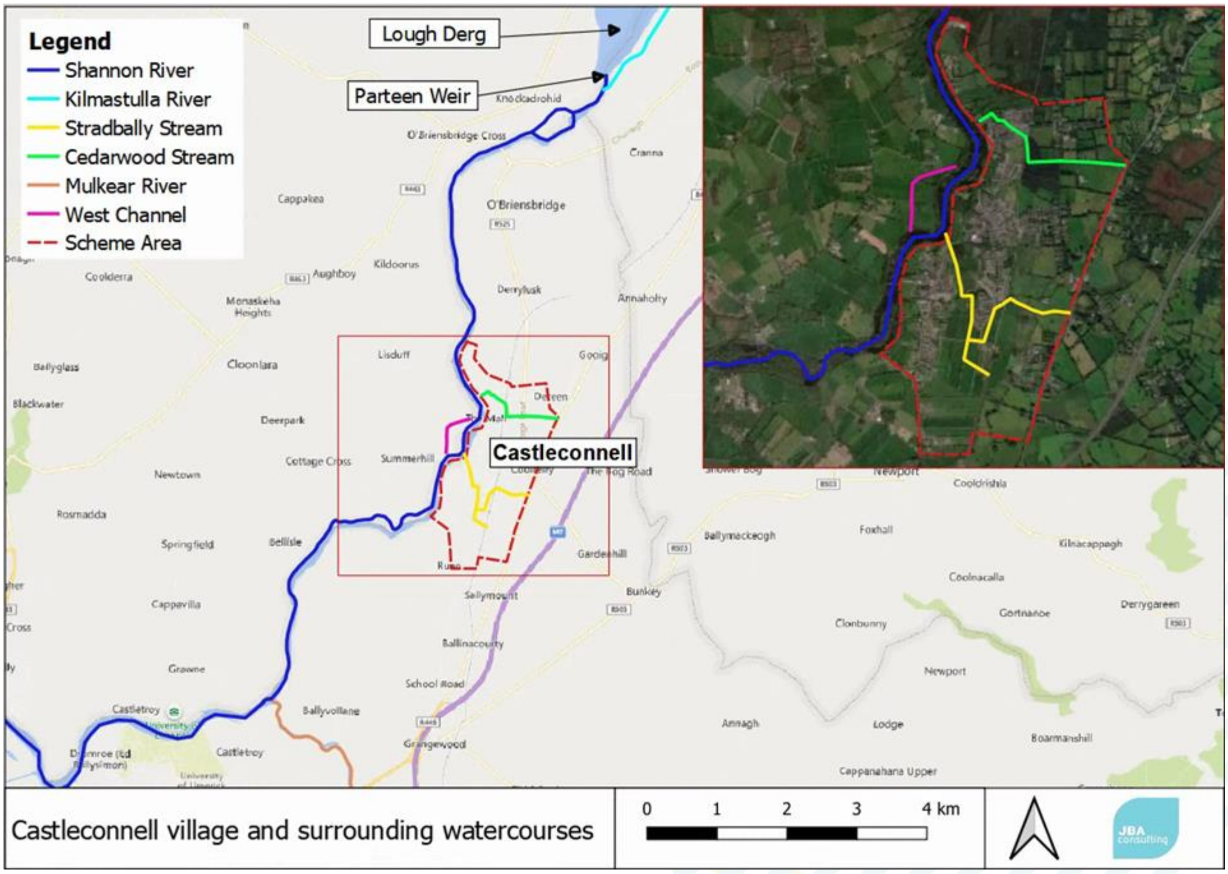

Figure 1-1. Scheme area map with watercourses modelled in the Shannon CFRAM study

1.2 Purpose of the Project

The overall purpose of the Castleconnell FRS project is to reduce risk to properties and critical infrastructure in future flood events.

A core part of the assessment will involve reviewing and, where necessary, updating the CFRAM Study outputs, including the hydrological and hydraulic analysis and options assessment.

As part of this, testing of proposed defences will be done using a hydraulic model of the watercourses and area. This hydrology report describes the proposed approach to reviewing the CFRAM Study and the steps needed to develop design flows for the hydraulic model to be used in assessment.

1.3 General catchment overview

The River Shannon is the dominant source of flood flows at Castleconnell and heavily influenced by Parteen Weir and Lough Derg. Other sources of flooding are the Kilmastulla tributary and potentially also the smaller tributaries that flow into the Shannon at Castleconnell. The Black River which enters immediately downstream of Parteen contributes a small amount of flow in relation to the total Shannon. It is included in the analysis.

The analysis must cover the River Shannon, the influence of the Parteen Weir offtake to Ardnacrusha, the Kilmastulla and smaller tributaries that flow into the River Shannon at Castleconnell, and any which may influence flood flows. The Stradbally Stream is the most southern watercourse and to the north is the Cedarwood Stream. There are no records of flood water flowing in the Limerick-Killaloe Canal in recent flood events. The canal is included in the hydraulic model and a flow of less than 10 m3/s is accounted for in the routing of flow downstream from Parteen Weir to Castleconnell. The different sources are described in detail in the sections below.

The Shannon CFRAM preliminary hydrology report (appendix B to the inception report) and the River Shannon Flood of Winter 1999/2000 report for ESB both describe the upstream catchment response. The ESB (2000) report describes a lag of 2 days for flow from Athlone to Lough Derg. All of the upstream contributing catchments are low lying, with wide natural floodplains and notable Arterial Drainage Schemes. The contributing catchment into Lough Derg (of 1,602 km^2 excluding the Shannon and Suck) and the Shannon downstream of Killaloe is much more responsive, with smaller catchments with steeper slopes and less floodplain attenuation. The FSU catchment descriptors in Table 1-1 give an indication of the response to runoff and hydrological regime of upstream catchments. There is a significant proportion of upstream catchments artificially drained and lake attenuation on the Shannon.

Downstream of Lough Derg is a 3.5km long channel from Lough Derg through Killaloe and Ballina to the upstream end of the Parteen Basin, which is an artificial reservoir before the Parteen Weir regulation of flow to Ardnacrusha and the spill to the Old Shannon.

Table 1-1. Indicative FSU catchment descriptors for upstream catchment locations

Location	AREA (km^2)	BFIsoil	FARL	ARTDRAIN2
Castleconnell	10,824	0.730	0.692	0.202
Parteen Weir	10,803	0.732	0.692	0.203
Portumna (Shannon upstream of L. Derg)	8,807	0.677	0.804	0.191
Suck (discharge at upstream end of L, Derg)	394	0.570	0.996	0.662
Banagher (Shannon between Athlone and Portumna)	7,978	0.654	0.786	0.209
Brosna (discharge into Shannon upstream of Banagher)	1,244	0.706	0.958	0.517
Athlone (downstream of L. Ree)	4,601	0.692	0.670	0.228
Nenagh (discharge into L. Derg)	320	0.599	0.997	0.415

The geometry of the Parteen Weir gates that control the Old Shannon flow are described in the ESB 1999/2000 flood event report. There are six gates numbered 1 to 6 from left to right. Their geometry is described in Table 1-2.

Table 1-2. Parteen Weir Shannon gates

Gate	Width (m)	Crest when closed (mOD Poolbeg)	Sill when open (mOD Poolbeg)
1	18	33.55	30.85
2	18	33.55	30.05
3 to 6	10	35.70	24.80
Note: Levels are to Poolbeg as the ESB gauge datums are to Poolbeg.			

2 Data available

The following tables summarise the hydrometric gauge and other relevant data available to the study.

Table 2-1. Sources of flood peak data

Source	
ESB	Parteen Weir gauge (ref: 25075)
OPW	No gauges relevant to study
EPA	Coole Gauge (ref: 25044)

Table 2-2. Relevant gauging stations

Water-course	Station name	Gauging authority number	Gauging Authority	Catch-ment area (km^2)	Type (rated / ultrasonic / level...)	Start of record and end if station closed
Shannon	Parteen Weir	25075	ESB	10,782	Level with flow rating	1933
Kilmastulla	Coole	25044	EPA	92.54	Water Level and Flow	1961

Table 2-3. Data available at gauging stations

Station name	Start of record	Update for this study?	OK for QMED?	Data quality check needed?	Other comments on station and flow data quality
Parteen Weir	1933	Yes	Yes	Yes	Flows derived for total flow through Ardnacrusha by the ESB based upon upstream, downstream levels and weir gate operation. The CFRAM was based on 78 years of data. We now have further years of AMAX data available to extend the record.
Coole	1961	Yes	Yes, subject to review.	Yes	A2 gauge. Use RC4 from 20/01/1984 to date for FSU. CFRAM report states deemed suitable for estimation of flows up to 0.8 times QMED based on a review of the check gaugings and AMAX flow data for the site.

Table 2-4. Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Comments and link to any rating reviews
Parteen Weir	Modelled estimate of flows based on upstream and downstream level and gate openings.	No	ESB have provided AMAX flow data at the outflow of Parteen Basin, which is the full Shannon inflow (i.e. including flow diverted to Ardnacrusha as measured at the Power Station Turbines and flow through Parteen sluices and weirs). The ESB control of water level in Parteen Basin by operation of Parteen Weir gates and the sluices that control flow to Ardnacrusha power station.
Coole	EPA rating is derived from spot flow gaugings.	No	Rating is extrapolated for QMED and above and so is of limited use. QMED in the CFRAM MPW model for the Kilmastulla is based on ungauged node QMED estimates using Coole gauge as pivotal site for adjustment where appropriate. Pooling groups are used to derive growth curve. Gauge is unlikely to be suitable for single site analysis as heavily influenced by bridge structures and floodplain attenuation. It was considered appropriate to use in CFRAM hydrology. This has been reviewed in this study.

Table 2-5. Other data available and sources

Type of data	Data available?	Source of data	Relevant to this study?	Details
Check flow gaugings	Yes	EPA	Yes	Rating curve and tables for Coole gauge were received
Historic flood data	Yes	OPW/LCC C	Yes	Historic flood records
		LCCC		2009 flood footprint
Flow or river level data for events	Yes	ESB	Yes	As discussed above
		EPA		As discussed above
Rainfall data for events	Yes	Met Éireann	Yes – only for validation of tributary response	Daily raingauges: Castleconnell (5919) Killaloe Docks (6019) Newport Coole (6919) Ardnacrusha (Gen.Stn.No.2)

Type of data	Data available?	Source of data	Relevant to this study?	Details
				(4011) Synoptic gauge at Shannon Airport (518)
	Yes	Met Éireann	No	Radar data for storm events may not have sufficient spatial resolution for the small urban catchments.
Potential evaporation data	n/a	n/a	No	Not necessary for this study.
CFRAM study method & outputs	Yes	OPW	Yes	
Results from other previous studies	Yes	ESB	Yes	River Shannon Inundation Study, Parteen Weir to Limerick City, (ESBI 1993).
	No (however data used is available)	n/a	Yes	Tony Cawley 2009 study
Other data or information	Yes	GSI, EPA	Yes	Groundwater spatial datasets

2.1 Groundwater flood sources

The PFRA does not indicate that groundwater flooding is a problem in most of the study area, refer to Figure 2-2. The GSI groundwater vulnerability for the area is primarily classified as "Moderate", which suggests groundwater levels are not close to the surface. There is a section of "Low" vulnerability in the northern sections of Castleconnell while there are sections of "High" to "Extreme" vulnerability in the south west sections of the study area. There are no karst features in the area. Detailed data and lack of historic records confirms the PFRA conclusions.

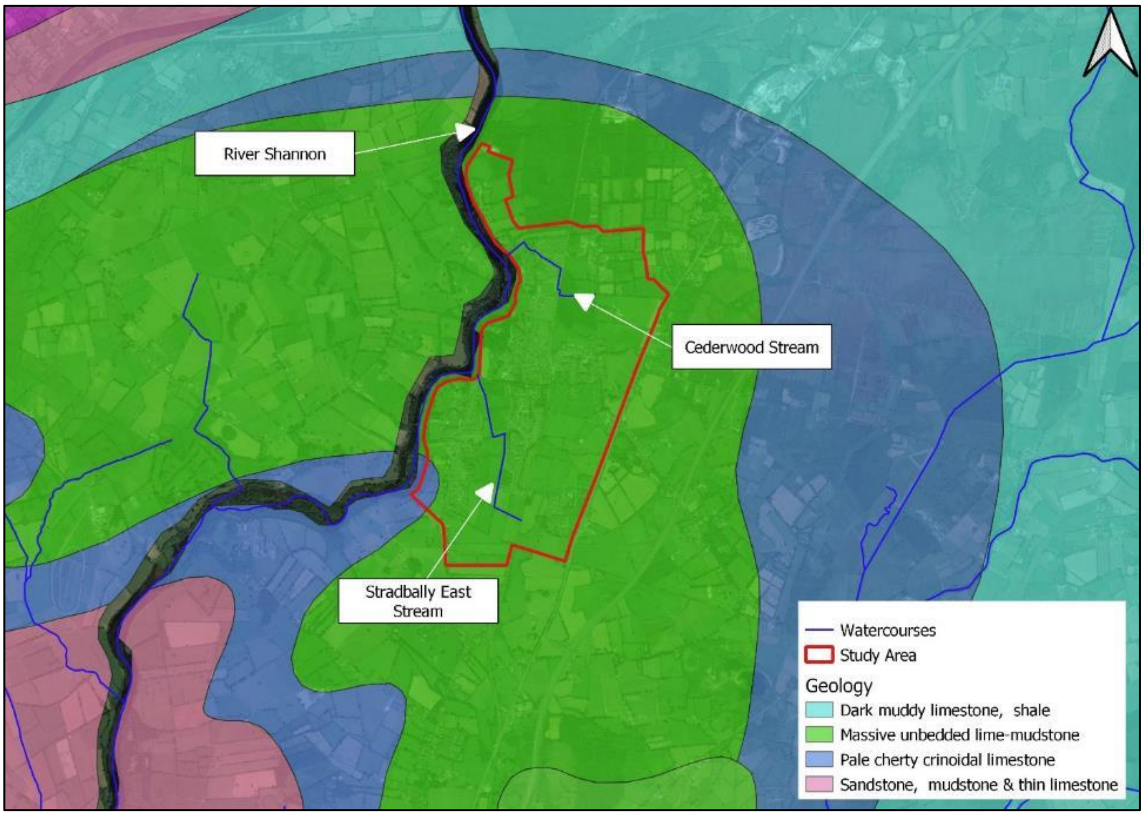

Figure 2-1: Geology map

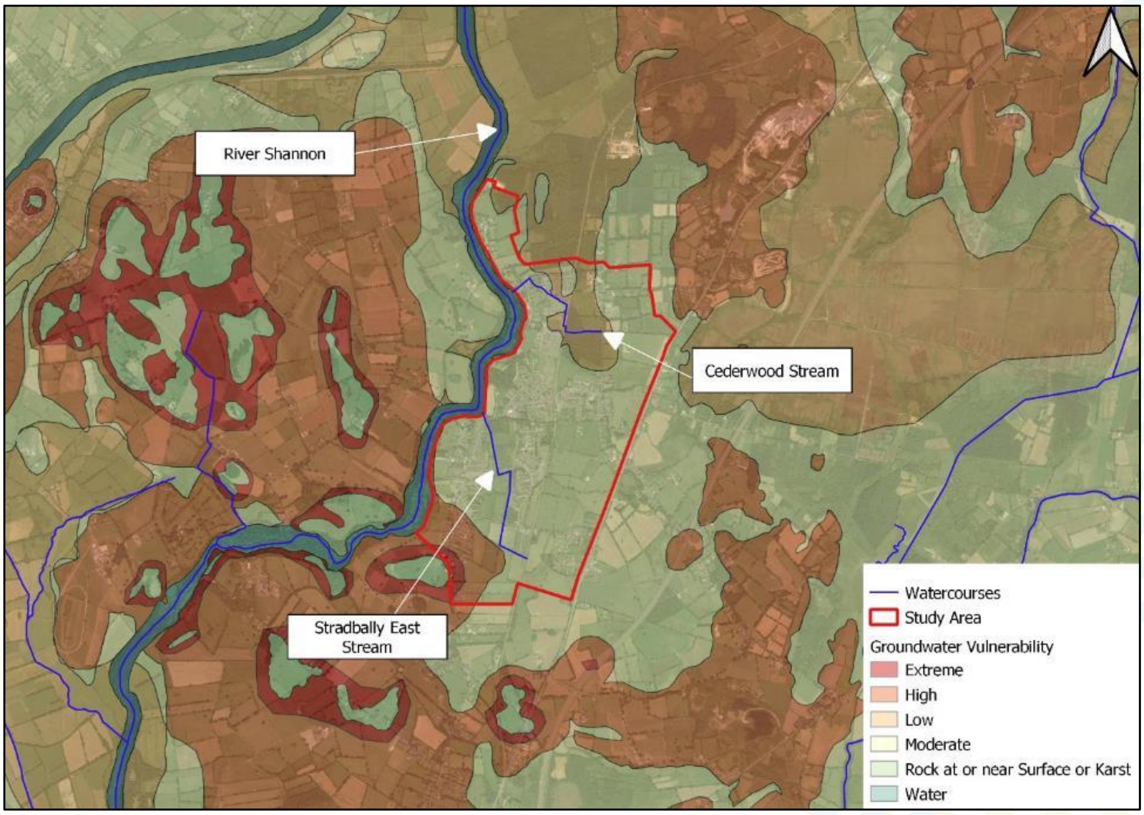

Figure 2-2: Groundwater vulnerability map

3 Flood History

This section of the hydrology report describes the observed data and records of flooding. Estimates of historic event probabilities are detailed in section 4.1 of this hydrology report.

3.1.1 ESB Parteen Weir (Station 25075)

Recorded flows on the Shannon at Parteen Weir are collected by the ESB and the annual maximum dataset has been provided to JBA for analysis (see Table 3-1). The dataset runs from 1933 to 2018 and the dataset reveals that the highest ranked floods are the events of Nov 2009 and Jan 2016 (2015 hydrometric year). 2009 was circa 20m^3/s higher than the Jan 2016 peak.

It should be noted that the flows downstream of the weir are split between the headrace canal for Ardnacrusha and the main channel of the Shannon through Castleconnell.

Only the 2009 and 2015 events have resulted in records of property flooding in Castleconnell. There are suggestions that the 2006, 1990, 1959, 1946 flooding affected Castleconnell, but no records are available. The full 2019 hydrometric year which includes the February 2020 flows has not been made available at the time of undertaking the analysis.

Table 3-1: 10 highest ranked flows at Parteen (1933 - 2019)

Rank	Hydrometric Year	AMAX (m^3/s)
1	2009	842.31
2	2015	822.22
3	1959	749.8
4	1994	741.7
5	2006	730.84
6	1989	704.7
7	1954	701.1
8	1999	700.84
9	1945	681.8
10	2001	667.17

3.1.2 Floodmaps.ie

The OPW National Flood hazard mapping website, www.floodmaps.ie, highlights areas at risk of flooding through the collection of recorded data and observed flood events. Refer to Figure 3-1 for location of historic flooding in the area. A summary of flood events is detailed as follows:

- Belmont Rd, Castleconnell, 19-24th November 2009. Overtopping of the River Shannon following unprecedented rainfall resulting in inundation of the R525 Castleconnell to Montpelier roadway the following areas within Castleconnell village, from Charco's to Scanlan Park and town car park towards the village;
- Gardenhill, Castleconnell, Recurring. Inundation of residential garden and against boundary wall but not the residential dwelling.

Additional flooding has occurred within Castleconnell during 1954, 1990, 2009, and 2015/16. It should be noted that significant flooding occurred throughout Ireland during the Winter 2015/2016 flood event and particularly along the River Shannon.

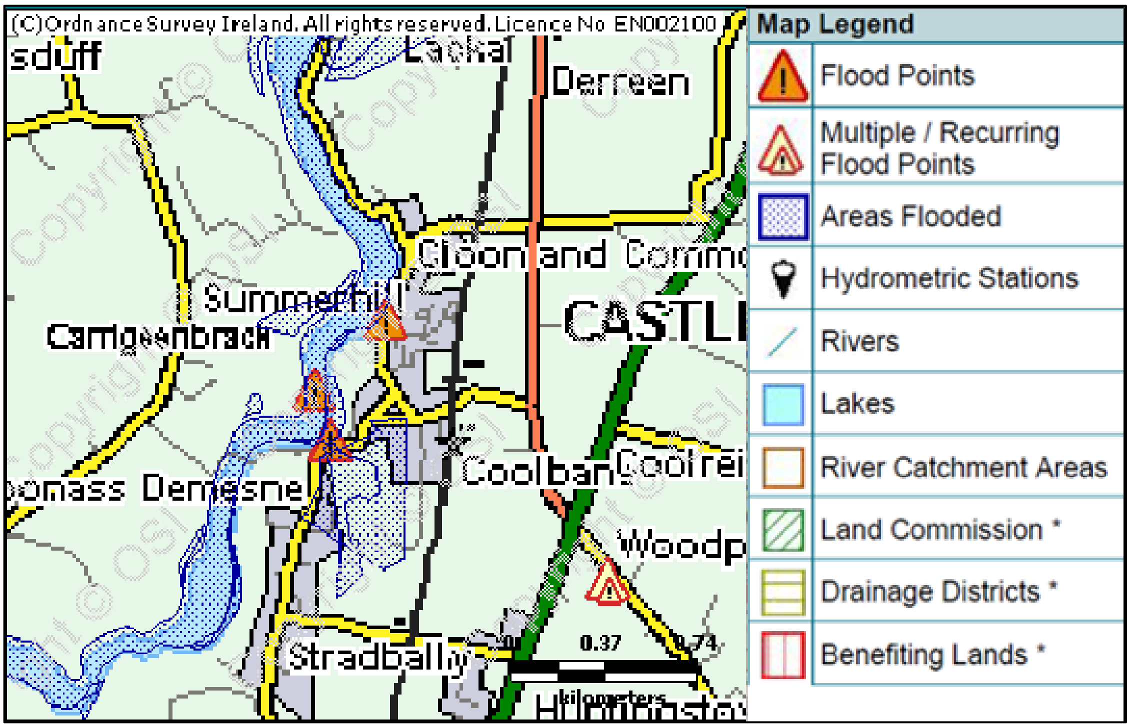

Figure 3-1: Floodmaps.ie

3.1.3 Limerick County Council Flood Report November/December 2009

A report was commissioned by Limerick County Council to review the flood event of November 2009 and the impact on the Lower Shannon area of County Limerick. Details on the recorded rainfall levels show that the rainfall totals for November 2009 were the highest on record at most monitoring stations.

Due to the large catchment of the Shannon River a substantial volume of the record rainfall was ultimately conveyed through the Parteen weir. The resulting impact was inundation of areas within Montpelier Village and through Castleconnell, Mountshannon and Plassey. Several roadways were closed during the flood event and includes the R525 Castleconnell to Montpelier roadway, and several roadways within Castleconnell village.

It is generally accepted that the November 2009 flood events has a return period of approximately 1 in 100 year (1% AEP), as suggested in the Shannon CFRAM Hydrology Report for UoM 25/26[1]. Cawley & Cunnane[2] also suggest a return period for the 2009 event to be between 1% to 0.5% AEP.

3.1.4 Winter 2015/2016 Flood Event

During November/December 2015 Ireland experienced significant flooding particularly along the River Shannon. Review of the OPW report[3] detailing the response to flood event suggests that the Long-Term Average (LTA) rainfall volumes for December were c. 240% higher for certain areas. Six storms affected Ireland during November and December, most notably Abigail, Desmond and Frank.

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1 CFRAM Hydrology Report Unit of Management 25/26 Final Report, OPW, 2016

2 Cawley, A. and Cunnane, C., 2010: Comment on the November 2009 Flooding in the Shannon and Corrib Systems. Irish National Hydrological Conference 2010

3 OPW Response to the Winter 2015/2016 Flooding in Ireland, O. Nicholson & Dr F. Gebre, OPW, 2017

A review of 75 No. gauging stations was undertaken as part of the report to determine the severity of the flood event. It concluded that 37 No. stations registered their highest flood level on record, 23 No. stations recording their second highest levels and 6 No. stations recorded their third highest. Whilst the OPW review did not undertake any single site analysis of the stations the 2015/16 event is likely to be a similar return period to the 2009 event.

The rapid mapping service provided by Copernicus EMS was activated with assistance from the OPW and provides further information regarding the winter 2015/2016 flood event. Flood extent mapping was provided by this service and is presented in Figure 3-3, which does flood parts of Castleconnell. Review of the available data confirms that within the Castleconnell area, the severity of the Winter 2015/2016 flood event was secondary to the November 2009 event.

3.1.5 9th-28th February 2020

The flooding occurred in Castleconnell Village after a prolonged period of heavy rainfall and three storms in the space of four weeks due to increased flows down the Shannon and therefore over Parteen Weir. The maximum flow over Parteen Weir directed down the old Shannon was 410 cumecs on 28th February 2020. Similar magnitude floods to 2009 and 2015. The following roads were affected – Belmont road, the road to Worlds End/Boat Club and The Elvers/The Mall Road. The road from Rivergrove B&B to Worlds end was closed for a length of c. 450m as it was flooded. The boathouse at the end of the cul de sac flooded, and equipment had to be moved or raised to prevent damage. Sandbags prevented internal property flooding of one house at the north of the Mall Road (Eircode V94X9C3). Large sandbags at the entrance prevented flooding of the Mall Road in this location. The pathway from Doonass Bridge to Castleoaks Hotel was impassable due to flood waters. The water was supercritical at Doonass Bridge and almost at the soffit of the openings.

Figure 3-2: Clareville cul de sac

Figure 3-3: Copernicus EMS Flood Extent Map

3.1.6 Limerick County Council Flood Report December 2015/January 2016

The Limerick report confirms that the maximum discharge was 470m^3/s from Parteen Weir on 1st January 2016. Levels on Lough Derg (Pier Head Killaloe) were 34.24mOD in Jan 2016 and 34.33mOD in November 2009, highlighting that the 2009 event was of slightly greater severity than the 2015/16 event.

The risk of flooding was communicated to residents by Limerick City & County Council by way of press release, and the Local Authority managed the impacts of flooding through their Severe Weather Crisis Management Team. Whilst the Major Emergency Plan was not activated the Crisis Management Centre was set up and managed the on-site operational response. Engineers from Limerick City & County Council's Operations and Maintenance Services Section (Service Operations Directorate) were constantly reviewing the situation on-site and reporting back to the Crisis Management Team. The Housing section of Limerick City & County Council was also providing regular feedback to the Crisis Management Team.

An Accommodation and Welfare team was established by the Local Authority. The Winter 2015/16 event progressed slowly which allowed time to advise vulnerable households as to what options were available to them should they need to evacuate.

3.1.7 Rainfall design storms

The design storm for the Shannon catchment upstream of Parteen Weir is quoted in the EBS 1999/2000 flood report as the 25-day duration for the 1,000 and 10,000 year events. The relative rainfall depths for these design storms and selected historic events are described in Table 3-2. Aerial rainfall for the full Shannon catchment has not been calculated for the 2009 and 2015 events, but selected raingauge totals gives an indication of the rainfall amounts.

The CFRAM preliminary hydrology report (Appendix B of the Inception report) has calculated flow volumes of historic events in sub-catchments, but not for any of the main Shannon gauges. Direct comparison of flow response to these events is not possible because operating procedures to control Shannon levels have changed over time.

This shows that potentially the 2009 and 2015 floods are in response to extreme 25-day duration rainfall depths. The 2015 total rainfall at Mount Russell is greater than the 2009 rainfall. On the basis the 2009 peak flow is greater, this suggests that there are other factors that influence the flow response to rainfall. This is discussed further in section 4.1.6.

Table 3-2. Rainfall design storms and event rainfall information for Shannon catchment upstream of Parteen Weir

Data source	Event	Storm Duration	Total Rainfall	Comments
ESB 1999/2000 flood event report	1,000 yr design storm	25 days	297 mm	
	10,000 yr design storm	25 days	384 mm	
	1999/2000 event	25 days	195 mm	Taken as 25 consecutive days of highest mean areal rainfall during Nov to Jan period
		92 days	414 mm	Total period mean areal rainfall over 3 months
	1994/95 event	25 days	180 mm	
PFRA ESB dams and embankments	1994/95 event	30 days	268 mm	Killaloe monthly rainfall for Dec 1994.
Met Eireann report on rainfall of November 2009	2009 event	25 days	258 mm	Castleconnell raingauge
		25 days	271 mm	Ballinasloe raingauge
		25 days	255 mm	Mount Russell raingauge
		25 days	216 mm	Athlone raingauge
Met Eireann Dec 2015 monthly weather bulletin	2015 event	30 days	374 mm	Rainfall at Mount Russell. Areal rainfall has not been calculated.

3.1.8 Review of River Shannon Inundation Study Parteen Weir to Limerick City (ESB, 1993)

This report assessed the residual risk from breach or other failure of Parteen Weir or embankments along the Ardnacrusha head race canal, in terms of the impacts downstream including Limerick City.

In terms of hydrology, three flood events are assessed. The 1,000 year, 10,000 year flows and a 50m wide breach at Parteen Weir and at various locations along the head race canal. The models used are calibrated to the February 1990 flow event.

The flood flows are quoted as 1050 m3/s for the 1,000 year flood and 1,330 m3/s for the 10,000 year flood. The breach scenario is modelled with an inflow of 175 m3/s into Lough

Derg (an average long-term inflow) and with Ardnacrusha at full load between 360 to 370 m3/s.

There is no detail on how the extreme flood flows have been derived in this study. The study includes flood maps which show no risk to Castleconnell in any scenario. These are two reasons why this report cannot be used in the Castleconnell FRS study 1) the omission in documenting the methods used to estimate flood flows and 2) recent flood events show risk to Castleconnell.

3.1.9 Review of Comment on the November 2009 flooding in the Shannon and Corrib Systems (Cawley and Cunnane, 2010)

This review compared hydrometric data for the 2009 flooding with extended AMAX series at gauges up to and including 2009. This study concluded an extreme flood frequency of 1 in 172 for the 2009 event at Parteen Weir and a 1 in 300 probability at Banagher, the two flow gauges on the Shannon. These are extreme estimates which the CFRAM flood flows and extents do not collaborate. We now have longer record series, including the 2015 flood event to further refine flood frequency estimates of these extreme historic events.

4 Data Analysis and Conceptual hydrological understanding of catchments

4.1 River Shannon

This section of the report focuses on the flood flow estimation of the total Shannon discharge at Parten Weir. The effect of the Ardnacrusha offtake and operation is covered in later sections of this report (Section 4.7.1 and Section 5).

Lough Derg upstream of Parteen is a significant break in the conveyance of river flow from upstream of Portumna. The location of Castleconnell and Parteen Weir is shown in Figure 4-1.

The Ardnacrusha Power Station scheme completed in 1922, included a significant canal that can divert a maximum of 400 m^3/s from the Shannon at Parteen Weir. During flood conditions this is limited to 380 m^3/s and in November 2009 345 m^3/s was diverted to the turbines. In normal flow conditions a minimum of 10 m3/s flow continues along the old route of the Shannon downstream to Castleconnell.

The Parteen Weir gauge (ref 25075) is the best source of data and information on the flow conditions and flood frequency for Castleconnell. The AMAX flow series with data up to and including 2018 is shown in Figure 4-2. The Parteen Weir gauge has the longest record length of all gauges on the Irish hydrometric register. There are no gauges with any data for pre-1932, specifically no gauge data for the Shannon before the construction of Ardnacrusha. Banagher (25017) is the gauge with the second longest record series for the Lower and Middle Shannon, with data from 1951. Drainage works splits the record at Banagher into pre-drainage for data before 1972 and post-drainage for more recent records.

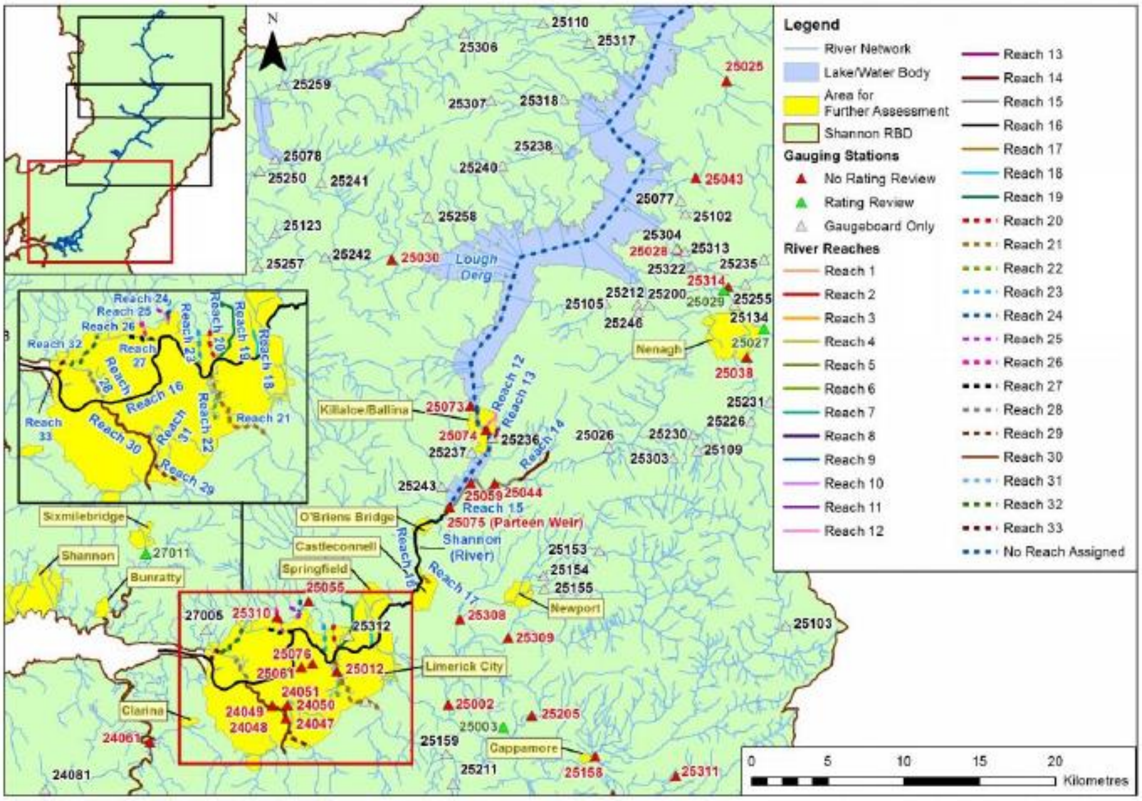

Figure 4-1. Map of Lower and Middle CFRAM River Reaches and Hydrometric Gauge Network (from tender brief)

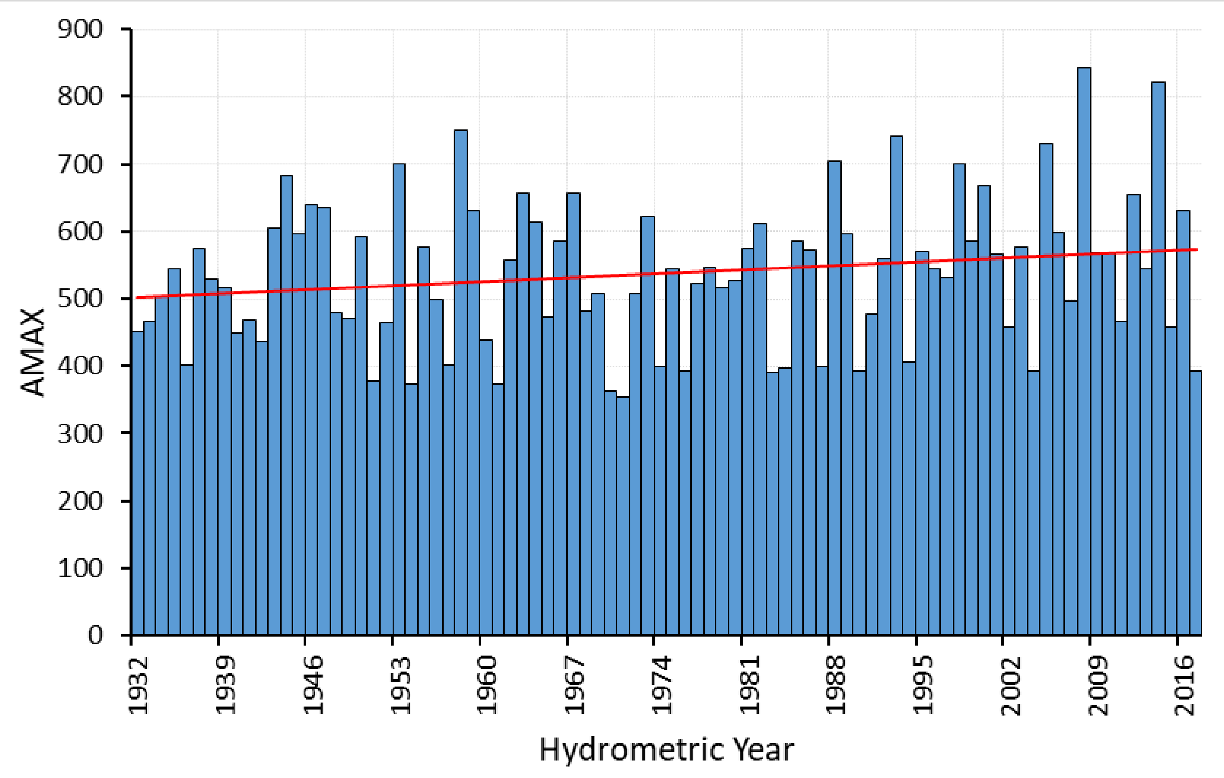

Figure 4-2: AMAX data at Parteen Weir (1933 – 2018) with trend line

4.1.1 Stationarity, trends and partial record series

Statistical analysis of the Parteen Weir AMAX record series has found that there is no statistically significant trend at a 5% confidence level (see trendline in Figure 4-2. The Mann-Kendal p-value is 14.06% and z-score is 1.47.

The effects of excluding historic or recent AMAX records from the Parteen Weir AMAX series, and comparing upstream Banagher and Athlone gauges has been analysed along with a five-year moving average (median) analysis. This information is plotted in Figure 4-3. At all three gauges the 2009 and 2015 peak flows are significantly higher than the 3rd highest AMAX flow. There is no distinct difference in the general trend, nor any step changes between the gauges to suggest exclusion of any period of record, or to use Banagher or Athlone as a pivotal or transfer site to Parteen Weir. Further analysis of the sensitivity to AMAX data series on the distributions to derive the growth curve is carried out below in section 4.1.5.

It is important to note that peak flows recorded at each of these gauges is influenced by the operation of structures and control of lough levels.

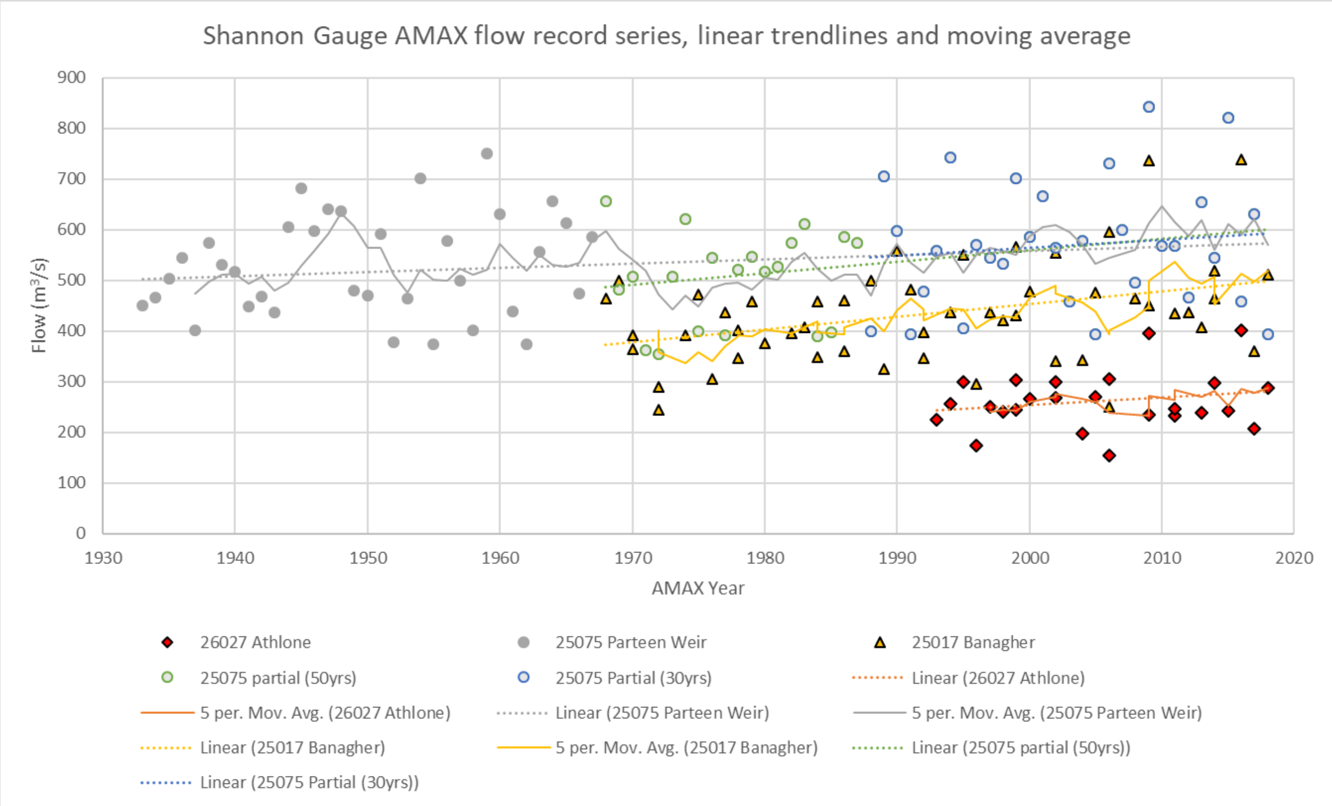

Figure 4-3. Full and partial record series and 5 period year moving average (median) window test for Parteen Weir (25075), Banagher (25017) and Athlone (26027) AMAX series

4.1.2 Influence of operation and level controls

The PFRA report on ESB Dams and Embankments (June 2011) is principally concerned with the risk from dam failure. The report includes a summary of the operating requirements for the Shannon system, which is to ensure the 10,000 year flood can be passed without any overtopping of dams, in this case the Parteen Weir structure and Ardnacrusha head race canal embankments. The Ballintra Sluices control discharge from Lough Allen to lower levels prior to flooding. Lough Ree level and outflow discharge is controlled by Athlone Weir sluices. The level in the Parteen Basin immediately upstream of Parteen Weir is controlled by Parteen Weir operation. In the past a turbine was in operation at Parteen Weir, however this is no longer operational due to a mechanical failure that occurred c.20 years ago. This was confirmed by ESB.

The Parteen Weir AMAX series is therefore influenced by operations and level controls by ESB.

The PFRA report (2011) states that the Parteen Basin water level can be drawn down to 30.0 mOD Malin (converted from 32.70m Poolbeg) prior to significant floods. This is to optimise discharge from Lough Derg through Killaloe. However, drawing water levels down in Parteen Basin is undertaken carefully due to the stability concerns associated with the Fort Henry embankments, and the 30.0mOD Malin level would not be exceeded. Normal maximum operating level is 30.86mOD Malin, when only the compensation flow is released downstream of Parteen weir.

The Shannon CFRAM River Shannon Level Operation Review report (July 2012) assesses the operating regulations and procedures of control structures on the River Shannon. The report concluded that the operation of the Athlone sluices effects flooding and levels around the Shannon Callows, with operations at Parteen having no effect above Meelick Weir. In relation to Parteen Weir flows in the Feb 1995 and Nov 2009 events a hydraulic gradient of 0.21m and 0.43m respectively was generated between Pier Head in Killaloe and Parteen Weir and that allows a discharge of between 750 to 850 m^3/s through the natural channel at Killaloe. This is a good reality check on the derived peak flows provided by ESB.

4.1.3 Index flood estimation

Given the unique nature of the Shannon at this location, there are no appropriate donor sites for data transfer and the use of pooling groups will not improve confidence in the fitting of extreme value distributions to the single site AMAX record. The use of other gauge records on the Shannon would not introduce any representative additional information and may in fact reduce confidence in flood flow estimates at Parteen Weir, because they do not share the same flow regime.

Single site Standard Error (SE) of QMED for an 86yr record series is roughly 6.7 giving a Factorial Standard Error (FSE) of around 1.04.

4.1.4 Growth curve estimation

A distribution is required to be fitted to determine the flood frequency estimates for given events. The decision to base the analysis on a single site, pooled or regional growth curve should consider the uncertainty and confidence intervals of the specific dataset, with reference to the length of the record series.

The Shannon CFRAM hydrology report also determined confirms that pooled analysis is inappropriate for the site (section A6.5 of the hydrology report). A single site EV1 flood growth curve is applied for all design events in the CFRAM study.

The LN2, GEV and EV1 (Gumbel) distributions have been considered for the Parteen Weir AMAX series as shown in Figure 4-4.

The 2 parameter LN2 and EV1 distributions have narrower error bounds for the extreme events but are potentially more susceptible to bias. The 3 parameter GEV distribution has better goodness of fit scores, but wider error bounds for the extreme events. Table 4-1 presents the goodness of fit scores.

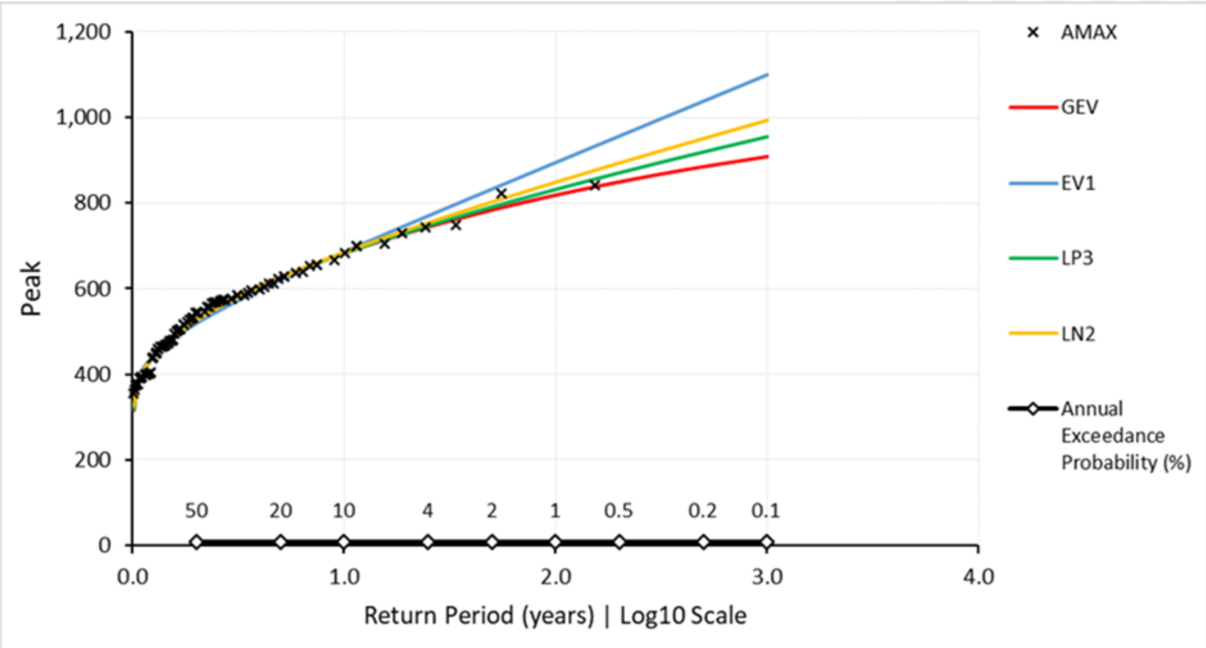

Figure 4-4. Comparison of statistical distributions

Table 4-1. Goodness of fit scores (better fit has lower scores)

Goodness-of-fit test	Distribution		
	LN2	GEV	EV1
Kolmogorov-Smirnov (standard moments)	0.09	0.085	0.118
Kolmogorov-Smirnov (L-moments)	0.086	0.081	0.105
Chi-squared (standard moments)	20.744	16.791	29.047
Chi-squared (L-moments)	18.767	18.372	18.372

Table 4-2 presents the peak flow estimates and ratio of the 1% AEP to 50% AEP for the three distributions and the 95%ile confidence interval error bound for each. The EV1 distribution has a significantly steeper growth curve (1.726 1% AEP growth factor) and so a higher 1% AEP peak flow estimate. The GEV growth curve has the shallowest 1% AEP growth factor (1.547), with the LN2 in the middle (1.612). The LN2 1% AEP peak flow has the narrowest error bound with the 95%ile confidence interval flow 1.088 times the best estimate. The distributions and confidence intervals are presented graphically in Figure 4-5.

Table 4-2. Peak flow estimates in m3/s and ratios

%AEP / Descriptor	Distribution					
	LN2	LN2 95%ile	GEV	GEV 95%ile	EV1	EV1 95%ile
50%	526.9	550.6	529.8	555.4	518.9	541.7
20%	626.3	659.5	628.5	658.5	619.8	657.3
10%	685.5	727.4	684.2	720.6	686.5	736.7
5%	738.5	790.5	731.4	778.7	750.6	813.2
4%	754.7	809.6	745.2	796.7	770.9	837.5
2.5%	787.9	849.3	772.5	834.5	813.5	888.2
2%	803.2	868.1	784.6	852.0	833.5	912.3
1.3%	830.4	901.0	805.5	884.2	869.9	955.8
1%	**849.4**	**924.3**	**819.4**	**907.3**	**895.7**	**986.7**
0.5%	894.0	979.5	850.2	962.2	957.6	1061.2
0.2%	951.2	1050.7	885.8	1034.8	1039.3	1159.8
0.1%	993.5	1103.7	909.2	1087.5	1101.0	1234.1
Ratios						
1% / 50%	1.612	1.679	1.547	1.634	1.726	1.822
1% 95%ile / 1%	1.088		1.107		1.102	

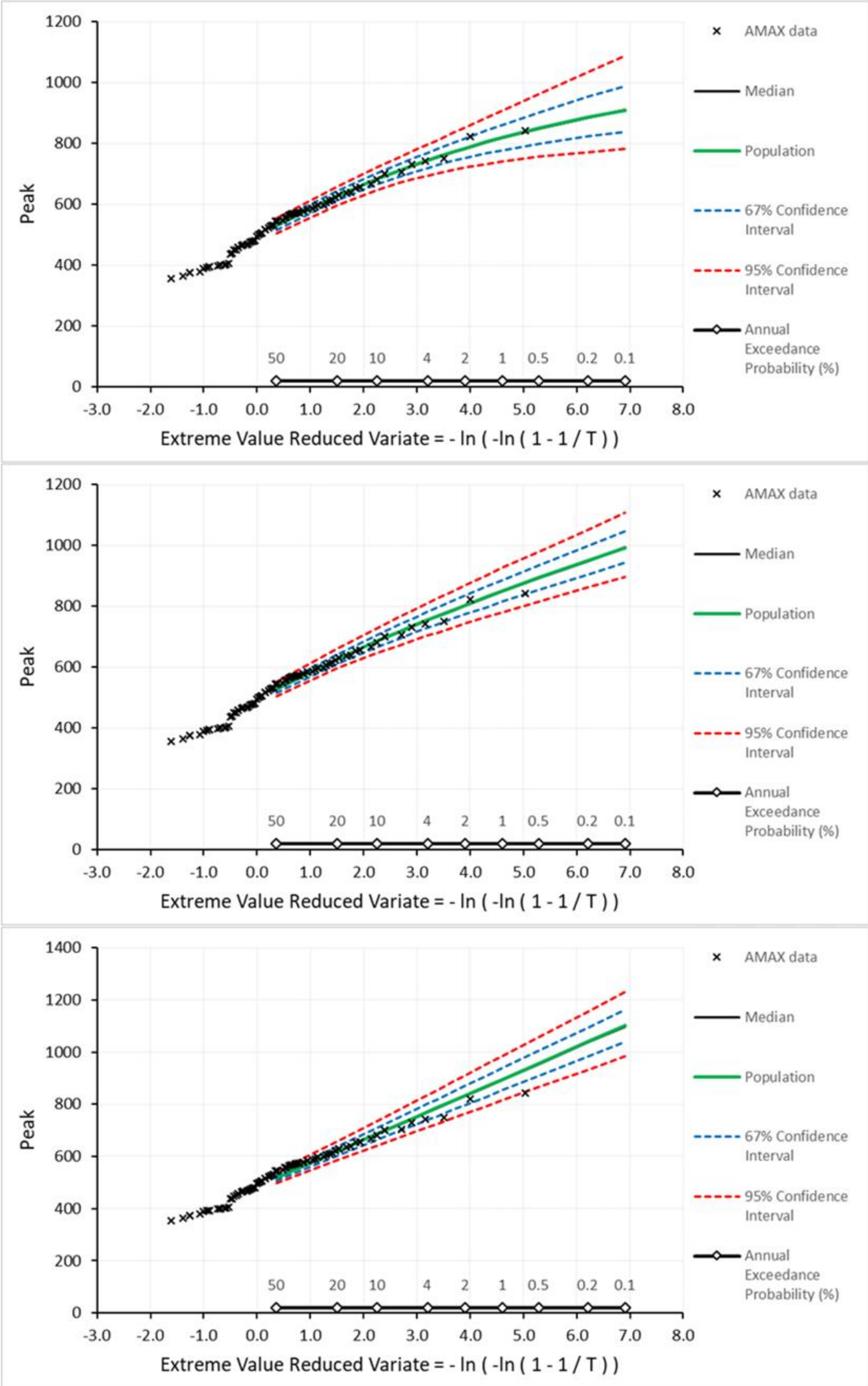

Figure 4-5. GEV (top), LN2 (middle) and EV1 (bottom) distributions and confidence intervals

4.1.5 Sensitivity and uncertainty of statistical distributions to AMAX series

The 3 parameter GEV and 2 parameter EV1 distributions are sensitive to the extreme events. This is demonstrated in the comparison of the growth curves for the AMAX series with only the latest 30 years or record and excluding the latest 10 years of record against the full AMAX series. The results of the sensitivity tests are presented in Table 4-3 and Table 4-4. The 2 parameter LN2 distribution is least sensitive to changes in the AMAX series.

Table 4-3. Peak flow estimates and ratios excluding the last 10 years AMAX data

%AEP / Descriptor	Distribution					
	LN2	LN2 95%ile	GEV	GEV 95%ile	EV1	EV1 95%ile
50%	520.5	544.1	526.5	553.0	512.4	535.9
20%	615.0	648.6	618.2	647.7	607.8	646.1
10%	671.1	714.4	666.9	700.6	670.9	721.5
5%	721.2	774.6	706.4	747.1	731.5	793.8
4%	736.5	793.1	717.6	762.0	750.7	817.0
2.5%	767.7	831.8	739.3	792.5	790.9	865.3
2%	782.1	849.4	748.7	806.8	809.9	888.1
1.3%	807.8	881.0	764.6	832.0	844.3	929.8
1%	**825.6**	**903.1**	**775.0**	**849.8**	**868.7**	**959.1**
0.5%	867.4	955.9	797.2	891.9	927.2	1029.9
0.2%	921.1	1024.0	821.6	944.5	1004.5	1123.8
0.1%	960.6	1074.9	837.0	981.0	1062.9	1195.5
Ratios						
1% / 50%	1.586	1.660	1.472	1.537	1.695	1.790
1% 95%ile / 1%	1.094		1.097		1.104	
1% AEP change (%)	97.2%		94.6%		97.0%	
Growth curve change (%)	98.4%		95.2%		98.2%	
1% AEPO 95%ile change (%)	100.5%		99.0%		100.2%	

Table 4-4. Peak flow estimates and ratios most recent 30 years AMAX data

%AEP / Descriptor	Distribution					
	LN2	LN2 95%ile	GEV	GEV 95%ile	EV1	EV1 95%ile
50%	562.9	607.4	563.9	613.6	554.0	598.4
20%	674.1	737.4	674.9	734.8	666.7	740.2
10%	740.7	823.8	739.5	813.9	741.3	839.0
5%	800.7	904.8	795.6	895.7	812.8	935.3
4%	819.0	930.1	812.3	923.0	835.5	966.2
2.5%	856.6	983.1	845.5	982.4	883.0	1029.9
2%	874.0	1008.2	860.5	1013.2	905.4	1059.8
1.3%	904.9	1052.4	886.5	1069.2	946.1	1115.3
1%	**926.5**	**1083.8**	**904.0**	**1112.4**	**974.9**	**1154.9**
0.5%	977.4	1157.8	943.5	1221.3	1044.0	1247.8
0.2%	1042.8	1255.0	990.2	1375.0	1135.2	1370.0
0.1%	1091.2	1328.0	1021.8	1505.1	1204.2	1463.5
Ratios						
1% / 50%	1.646	1.784	1.603	1.813	1.760	1.930
1% 95%ile / 1%	1.170		1.230		1.185	
1% AEP change (%)	109.1%		110.3%		108.8%	
Growth curve change (%)	102.1%		103.7%		101.9%	
1% AEPO 95%ile change (%)	107.5%		111.1%		107.5%	

4.1.6 Operational uncertainty and influence on historic event probabilities

The selection of distribution makes a significant difference to the estimation of the 2009 and 2015 flood peak annual exceedance probability as shown in Table 4-5. Figure 4-6 shows the daily flows for the total Shannon, Ardnacrusha and the Old Shannon as recorded by the ESB. It is worth noting that there is a greater proportion of the total peak flow in the Old Shannon in the 2009 event than the 2015 event. The peak flow during each event (for the period when the Total Shannon flow is in excess of 500 m^3/s) for the total Shannon flow, the Ardnacrusha flow and Old Shannon flow is shown in Table 4-6. This shows that the peak Old Shannon flow does not always occur on the same date as the peak total Shannon flow.

Table 4-5. Approximate estimate of 2009 and 2015 event % AEP

Event (peak flow)	Approximate estimate of event % AEP		
	LN2	GEV	EV1
2009 (842 m^3/s)	1%	0.5%	2%
2015 (822 m^3/s)	1.3%	1%	2.5%

Table 4-6. Comparison of peak flows and date of peak flows as recorded by ESB for 2009 and 2015/16 events

Event	Flow (m^3/s) and date			
	Total Shannon flow	Ardnacrusha	Ardnacrusha (on date of peak Old Shannon Flow)	Old Shannon Flow
2009	842.31 26/11/2009	366.83 15/11/2009	341.70	497.00 28/11/2009
2015	822.22 05/01/2016	391.20 02/12/2015	335.95	462.01 06/01/2016

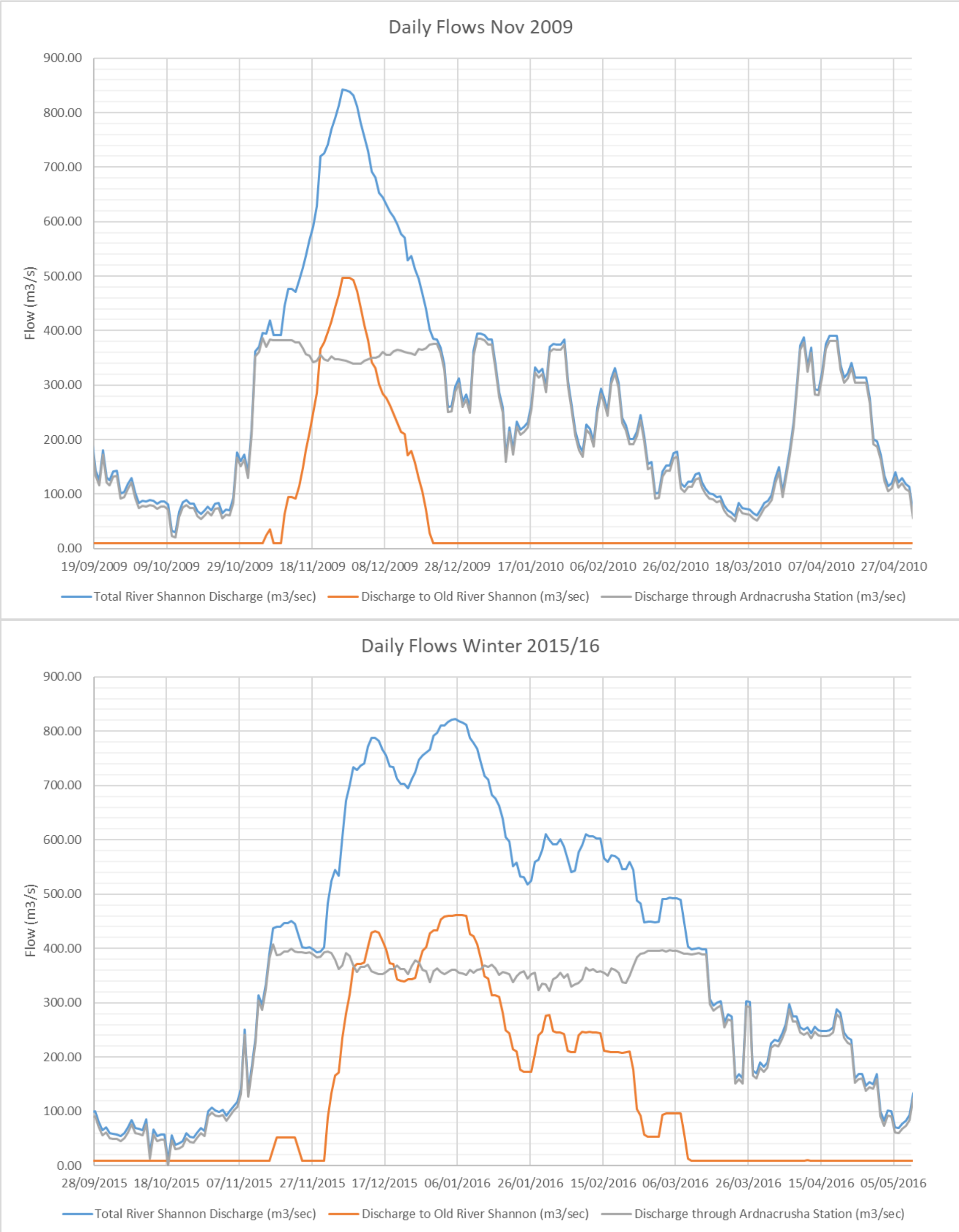

Figure 4-6. Daily flows as recorded by ESB for 2009 (top) and 2015 (bottom)

There is therefore notable uncertainty in the decision to removing a uniform 345 m^3/s from the flood hydrograph to account for the regulated flow down the headrace to Ardnacrusha. With a 345 m^3/s reduction in flood peak flow the resulting 1% AEP flow in the Old Shannon would be 504, 474 and 551 m^3/s (respectively for LN2, GEV and EV1). The headrace canal flow will not impact upon AMAX values, QMED or flood frequency estimates as these are based on the total Parteen Flow. The remaining flow in the Old Shannon for the LN2 and GEV distribution is less than the total Parteen QMED flow of 544.5 m^3/s. As no significant trend has been detected in the Parteen Weir AMAX series, it could be assumed that the natural “pre-Ardnacrusha” flow is the total Parteen Weir flow without any inflow to the headrace. A flood of QMED magnitude would not be expected to result in flood damages such as seen in 2009, which suggests there are other hydraulic factors influence the change in flood risk over time in Castleconnell. The possible causes will be explored in the hydraulic modelling analysis:

- Changes to the channel form and conveyance capacity in the Old Shannon since the completion of Ardnacrusha.
- Natural constriction of discharge from Lough Derg from the channel at Killaloe.
- Changes in operation of water levels and structures.
- Uncertainty in the flow estimation at Parteen Weir.
- Variation in sub-daily flow at Parteen and Ardnacrusha operation, which is not reflected in the daily flow data.

4.1.7 Alternative indicators to AMAX series

The ESB calculate the total Shannon daily inflow (Parteen Weir total flow + change in storage in Lough Derg, Ree and Allen) and daily inflow to Lough Derg. The uncertainty in this flow estimate is unknown. These flow calculations remove some degree of operational influence and are shown in Table 4-7 for significant past floods up to 2011 when the PFRA report was published. Use of the total Shannon daily inflow will not reduce uncertainty in the Parteen Weir AMAX series because there is a temporal effect from the influence of initial lough levels and upstream controls on attenuating the peak flow. There is also the natural attenuating effect of the River Shannon channel at Killaloe, which constrains the outflow from Lough Derg.

Table 4-7. Parteen Weir comparison of Peak Lough Derg Inflow, Peak Total Catchment Inflow and Maximum Discharge to Shannon (old channel) for Significant Past Floods (from PFRA report)

	Winter 1994/1995	Winter 1999/2000	Winter 2006/2007	Nov/Dec 2009
Peak Lough Derg Inflow	809 m^3/s	757 m^3/s	771 m^3/s	929 m^3/s
Peak Catchment Inflow	1,035 m^3/s	1,019 m^3/s	925 m^3/s	1,243 m^3/s
Discharge to River Shannon	385 m^3/s	376 m^3/s	370 m^3/s	497 m^3/s

The conveyance capacity of the Shannon through Killaloe, as represented in the CFRAM models, is in excess of 1,000 m3/s and does not present a constriction that could limit present day or climate change 1% AEP peak flow estimates for Parteen Weir.

Flood event volume analysis is also a function of lake level and Ardnacrusha operation. This would not improve the uncertainty in the peak flow estimates and so has not been carried out.

4.1.8 Best estimate of the hydrological design flows

The best estimate of the Total Shannon upstream of Parteen Weir design event flow is from the Parteen Weir AMAX single site LN2 distribution growth curve. The design flows are presented in Table 4-8 and the total flow growth curve and confidence intervals plotted in Figure 4-5.

Table 4-8. Best estimate of design event flows for the Total Shannon upstream of Parteen Weir

% AEP	Total Parteen Weir Flow (m^3/s)
50%	526.9
20%	626.3
10%	685.5
5%	738.5
2%	803.2
1%	**849.4**
0.5%	894.0
0.1%	993.5

4.1.9 Hydrograph shape

The hydrograph shape used in the CFRAM study is based on the 2009 flood event as recorded at Parteen Weir (Figure 4-7). Application of FSU hydrograph shape methods is not valid as the hydrograph shape is a function of operating rules for Ardnacrusha. The 2015 hydrograph shape of Old Shannon flows (Figure 4-6) shall be used for the scheme model. The shape of the hydrograph is strongly influenced by the operation of water levels in the Parteen Basin, Ardnacrusha turbine operation and gate operations at Parteen Weir.

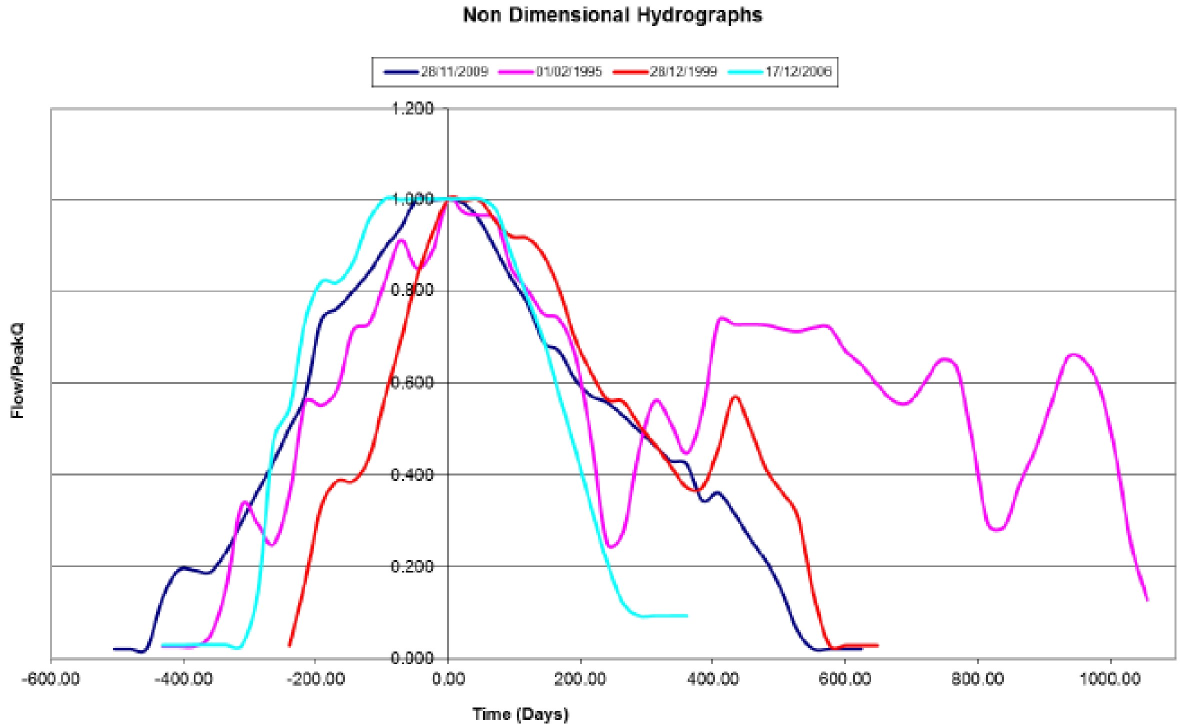

Figure 4-7. CFRAM hydrograph shape (model N12, reach 16)

4.1.10 Climate change flows

Climate change projections specified in the project specification are for an increase in peak flow by 20% for the MRFS and 30% for the HEFS. Table 4-9 presents the climate change flows.

Table 4-9. Climate change flows for the Total Shannon upstream of Parteen Weir (1% AEP)

Climate change projection	1% AEP current Total Flow (m^3/s)	1% AEP future Total Flow (m^3/s)
MRFS (+20%)	849.4	1,019.3
HEFS (+30%)	849.4	1,104.2

4.2 Kilmastulla River

The Kilmastulla River is now diverted to flow alongside Parteen Basin and joins the Old Shannon downstream of Parteen Weir. As the catchment contributes additional flow to the flow that passes over Parteen Weir, it needs to be considered in this study. The most downstream ungauged FSU node for the Kilmastulla river is at its original discharge location into Lough Derg (see Figure 4-8).

The contributing catchment area (103km²) is similar to the FSU node (102.23km²), with a difference of only 0.77km². It is therefore comparable to the total contributing catchment to the current outlet to the Old Shannon. The difference in catchment area between the Coole Gauge, using the FSU gauged node (92.55 km^2), and the confluence of the Kilmastulla River and Shannon (103km^2) is around 10.55 km^2.

The EPA Blueline River Network does not need to be reviewed as the route used by the CFRAM MPW model is correct.

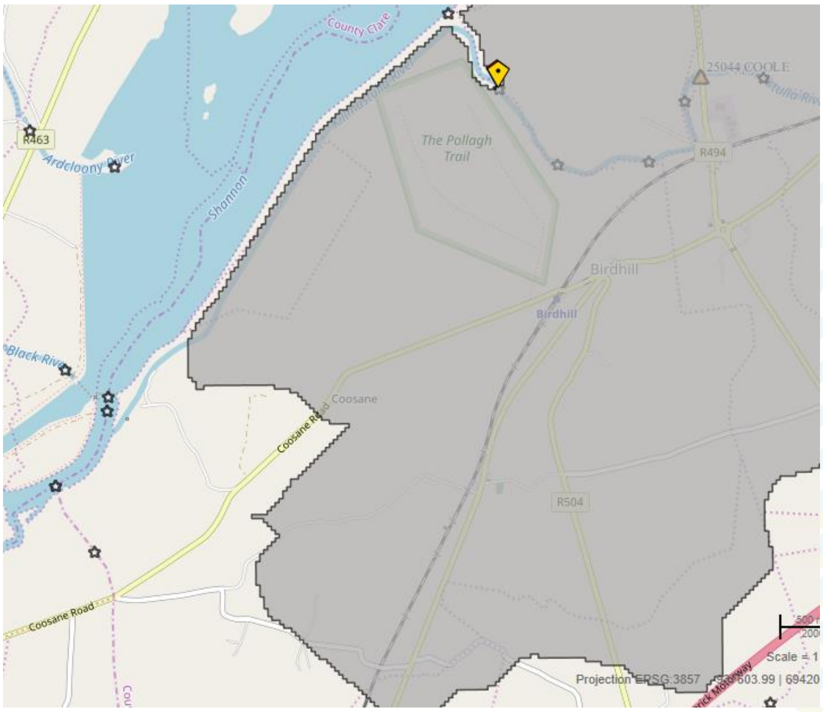

Figure 4-8. FSU portal catchment boundary of most downstream ungauged node on the Kilmastulla River

For Coole gauge it is important that only flow AMAX records for 1983 onwards after completion of drainage district works are used. The 1994 AMAX flow is a low outlier and has been removed from the AMAX series. This is because it is an incomplete year of flow data and no substantial flow was observed in this year. This does not alter the QMED estimate but could influence the fitting of a distribution to the AMAX single site series. Our analysis has used the full AMAX series as supplied by the EPA which includes extra years beyond the record length on the FSU portal.

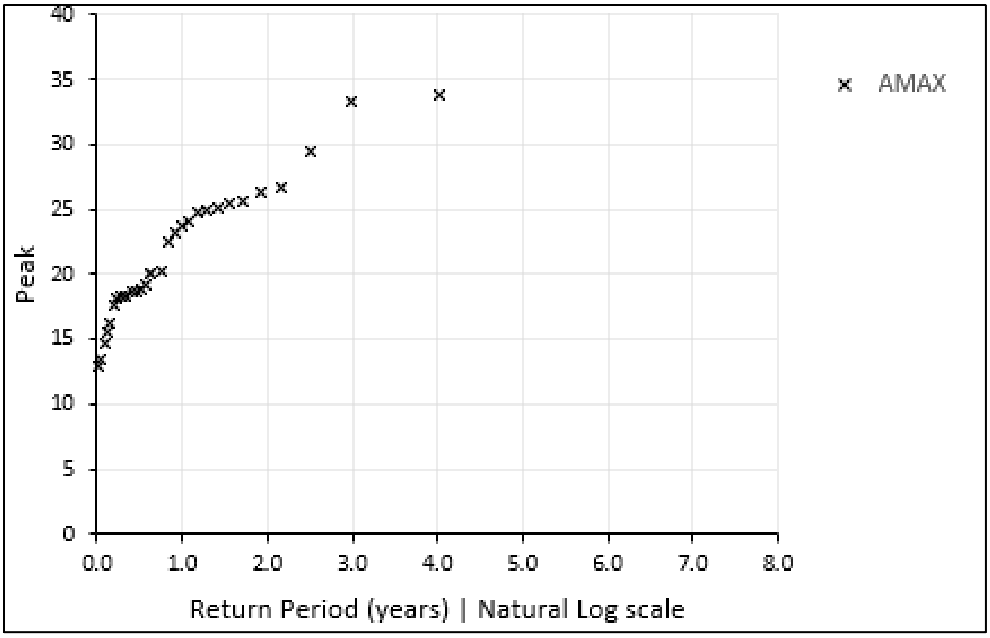

Figure 4-9: Coole Gauge AMAX series plotting position

The QMED adjustment factor for Coole gauge (25044) derived in the CFRAM study is 0.98 from the ratio of the gauged QMED of 20.1 to the FSU QMED catchment descriptor estimate of 20.6. The CFRAM study uses a pooled growth curve and the EV1 distribution for flood frequency estimates on the Kilmastulla River. Given the uncertainty in the inflow boundary conditions to the CFRAM routing model we have compared these to ungauged unadjusted FSU QMED estimates. The catchment area size is suitable for FSU analysis and so other methods are not required.

Appendix A contains details of different approaches to flow estimation. Table 4-10 presents the CFRAM peak flow estimates. Table 4-11 presents relevant catchment descriptors and Table 4-12 presents the comparison of peak flow estimates. Given the similarity between the FSU ungauged node peak flow estimates and the CFRAM model inflows, we can be sufficiently confident in the MPW model inflows based on the gauge adjusted QMED and the pooling group growth curve. The pooling group growth curve is shallower than the single site growth curve. Given the uncertainty in single site peak flow estimates above QMED (the gauge rating is highly suspect for flows greater than 0.8 times QMED), the GS is considered unreliable so has not been used in the analysis, and use of the pooling group growth curve is appropriate.

Table 4-10. CFRAM growth factors and peak flow estimates for Kilmastulla River (model N12 reach 15) at Coole Gauge

% AEP	Growth factor	Peak flow (m³/s)
50% (2yr)	1.00	20.20
20% (5yr)	1.22	24.64
10% (10yr)	1.35	27.27
5% (20yr)	1.47	29.69
2% (50yr)	1.62	32.72
1% (100yr)	1.73	34.95
0.5% (200yr)	1.84	37.17
0.1% (1000yr)	2.08	42.02

Table 4-11: Catchment characteristics for Kilmastulla at Coole Gauge and at downstream ungauged FSU node

Descriptor	Coole Gauge	FSU node downstream of Coole Gauge 25_3881_5	FSU node at outlet to Shannon 25_3881_9
Area	92.55	93.37	102.26
SAAR	1187	1187	1185
FARL	0.997	0.997	0.997
URBEXT	0	0	0
ArtDrain2	0	0	0
S1085	2.666	2.696	2.666
DRAIND	1.371	1.338	1.241
BFIsoil	0.583	0.582	0.582
QMED (catchment descriptors unadjusted)	19.44 m³/s	19.51 m³/s	20.62 m³/s

Table 4-12: Comparison of Kilmastulla flow estimates (with CFRAM growth curve)

Annual Exceedance Probability (%)	CFRAM Pooling Group Growth Factor	Coole Gauge (CFRAM estimate)	FSU node downstream of Coole Gauge 25_3881_5	FSU node at outlet to Shannon 25_3881_9
50% (2yr)	1	20.20	19.52	20.62
20% (5yr)	1.22	24.64	23.81	25.16
10% (10yr)	1.35	27.27	26.35	27.84
5% (20yr)	1.47	29.69	28.69	30.31
2% (50yr)	1.62	32.72	31.62	33.41
1% (100yr)	1.73	34.95	33.77	35.68
0.5% (200yr)	1.84	37.17	35.92	37.94
0.1% (1000yr)	2.08	42.02	40.60	42.89

4.2.1 Hydrograph shape

Hydrograph shapes were derived based on the 2005 event gauged hydrograph shape which is the highest recorded flood at Coole gauge(Figure 4-10). The 2005 event peak is 32.7 m3/s which is higher than the recent 2015 flood event peak. It is worth noting the 2015 event has a similar hydrograph shape to the 2009 flood (Figure 4-11) and also similar peak flows. Potentially due to topping out of the gauge.

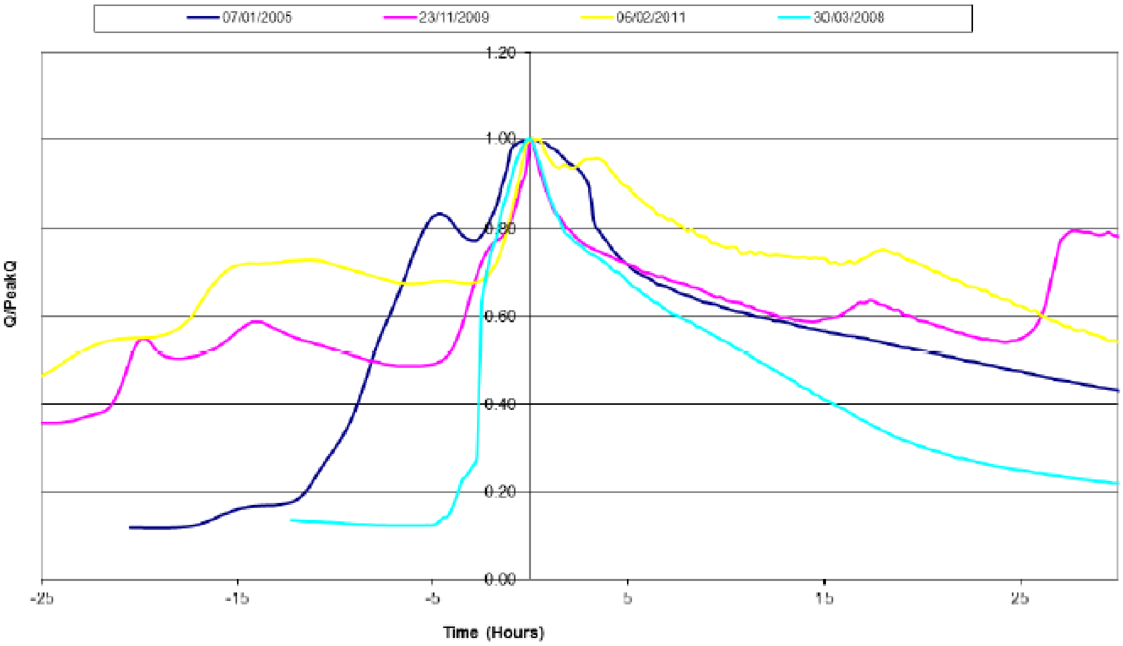

Figure 4-10. CFRAM hydrograph shape for Coole gauge (25044)

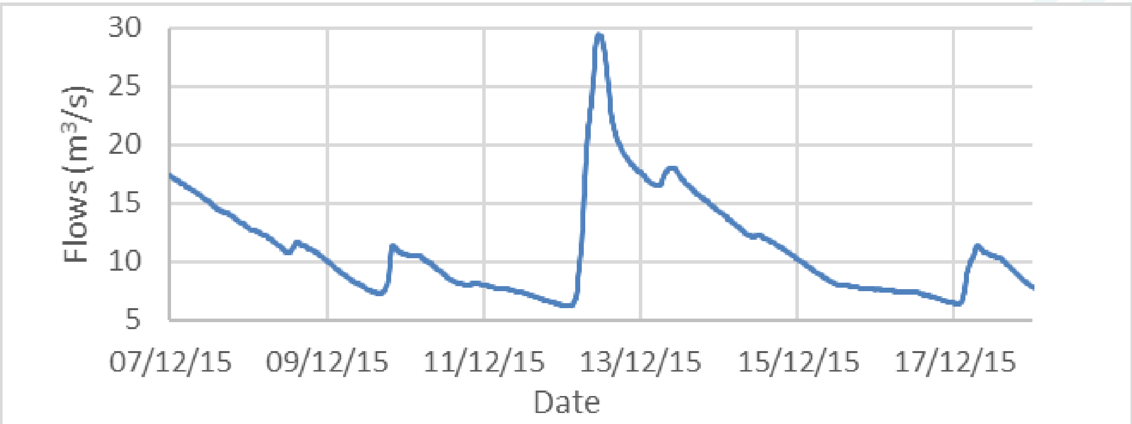

Figure 4-11. December 2015 Kilmastulla flood event

The applicability of the Coole Gauge hydrograph shape and CFRAM hydraulic model outputs has been reviewed and this confirms that there are routing effects to the hydrograph (see Figure 4-12). A number of lateral inflows are included in the CFRAM model, which is a 1D ISIS model with reservoir units to represent the floodplain. A review of the CFRAM model has found that many of these spills are not correctly schematised with the lateral inflows incorrectly assigned to upstream node levels and not interpolated node water levels. For determining the contribution of the Kilmastulla River to the Old Shannon flow, these lateral spills have been inactivated so that the Kilmastulla River inflow is only a function of attenuation and routing of flow within the embankments of the Kilmastulla River. The inflows for the HEP from the Kilmastulla River need to be derived from the routing of flow estimates at Coole gauge through the MPW model.

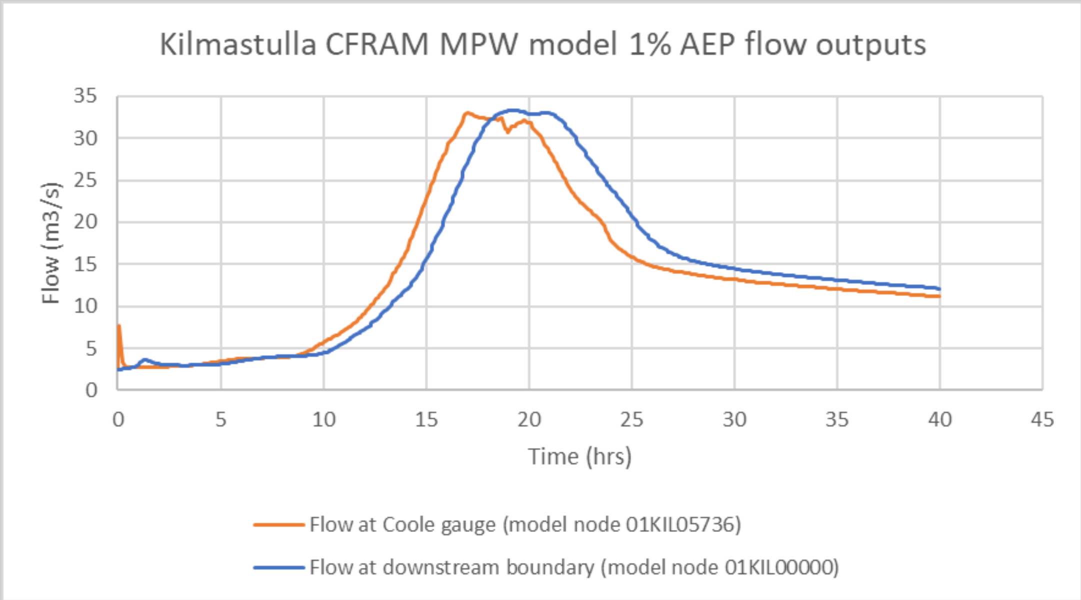

Figure 4-12. Routing of the Kilmastulla flood hydrograph from the Coole gauge to tributary with the Shannon

4.2.2 Sensitivity to downstream Boundary Conditions (Shannon flow and level)

The CFRAM MPW model for the Kilmastulla has a downstream boundary condition based on a high Shannon flow, a constant 23.6 mOD, which is lower than the 10% AEP peak level as shown on the CFRAM flood maps.

A further downstream boundary sensitivity test has been run by JBA for the CFRAM MPW model for the Kilmastulla, with a downstream boundary based on the 1% AEP level on the Shannon (as published on the final CFRAM flood extent maps). There was no CFRAM node on the Shannon at the junction with the Kilmastulla so the nearest downstream node had to be used (upstream node is upstream of Parten Weir). This was Node 15LSH02861, 1.3km downstream. To account for the difference in water height upstream, the 0.1% water level at the node (26.07mOD) was used as the 1% water level in the Shannon at the junction with the Kilmastulla. The increase downstream boundary condition was used with three design inflows: 10, 2% and 1% AEP events. The increase in downstream boundary level has no impact on flows along the Kilmastulla, as seen in Figure 4-13 below.

The outflows for the Kilmastulla are therefore not sensitive to levels in the Shannon. This means that the hydraulic model outflow takes account of the backwater effect from a high flow on the Shannon, and therefore can be used to inform the contribution of the Kilmastulla to the Shannon, during a Shannon flood. We are confident in the application of the Kilmastulla model to determine the scale of attenuation for the estimation of HEP flows at the downstream of the Kilmastulla River.

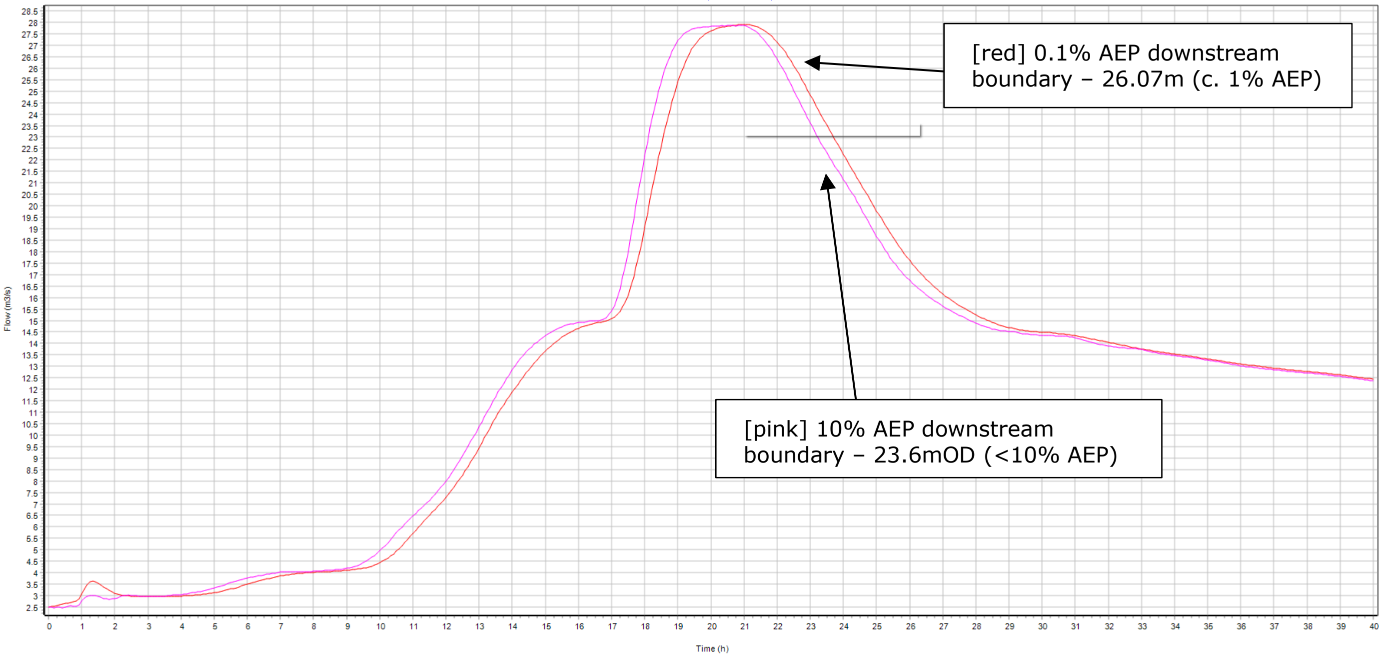

Figure 4-13: Kilmastulla downstream boundary conditions – Q100 Flow hydrograph

4.2.3 Kilmastulla design event inflows

The CFRAM Shannon models do not include a designated inflow for the Kilmastulla catchment. The Kilmastulla inflow most likely forms part of the 'Lough Derg (Upper)' inflow in the CFRAM model, which represents the lateral tributaries entering the lake. This inflow is applied upstream of Parteen Weir whereas the Kilmastulla enters the Shannon downstream of Parteen Weir. The scheme modelling shall ensure Kilmastulla inflows are applied to the correct location.

Upon review of the Coole GS records and its AMAX there are clear issues with out of bank flow and the record is considered unreliable. It is also has been impacted by drainage works in the catchment. The CFRAM model is incorrectly representing the hydraulic processes in this area. For the purposes of defining a representative inflow that is routed from the top of the Kilmustalla model to the Shannon an FSU based estimate of extreme flood flows was undertaken as the gauge was unreliable. The downstream model outputs for the Kilmastulla River are be used as inflows to the Castleconnell FRS hydraulic model.

It is likely that a Kilmastulla high flow event could occur independent of the Shannon flows. A flood on the Kilmastulla alone (Kilmastulla flow plus 10m^3/s Shannon baseflow) is well below the threshold flow that would cause any flood problems in Castleconnell. It is likely that a Kilmastulla flood response could occur at the same time as when the Shannon is in high flow conditions. The probability of such an event would be less than the Shannon flow probability (e.g. a 1% AEP Kilmastulla at the same time as a Shannon 1% AEP event would in total be less likely than the 1% AEP). The joint probability (JP) of flow events on the Kilmastulla and the Shannon was assessed using the FSU Guidance for River Basin Modelling work package, Table 4-13. The table indicates that during a 1% AEP event on the Shannon a 14% AEP event is estimated for the Kilmastulla (red outline). The conservative approach was taken where the 5% AEP Kilmastulla flow hydrograph will be applied to the old Shannon flows for the 1% AEP design event. This is due to the magnitude of Shannon flows is significantly higher than Kilmastulla's, and the difference between 5% and 20% on Kilmastulla is less than 5cumecs – than means only 10mm water level difference in the model.

The timing of the Kilmastulla inflow shall be set so that peak flow on the Kilmastulla and Shannon coincide.

Table 4-13: Dependence model results for 1% AEP event for different classes of pairwise catchment descriptors

AEP at conditioning site: 0.01 Equivalent return period: 100 years			Table C-1: Dependence model results for different classes of pairwise catchment descriptors								
Connected	Difference of BFI within 0.3	Centroids within 25 km	Ratio of AREA within a factor of 2.7	Difference of FARL within 0.07	5%ile AEP	25%ile AEP	**Median AEP period at dependent site**	75%ile AEP	95%ile AEP	No. of pairs in data	Interpretation of site configuration
TRUE	FALSE	FALSE	FALSE	FALSE	1.000	1.000	0.989	0.732	0.102	4	Not enough data
TRUE	FALSE	FALSE	FALSE	TRUE	NA	NA	NA	NA	NA	0	
TRUE	FALSE	FALSE	TRUE	FALSE	0.972	0.480	0.188	0.052	0.002	4	Not enough data
TRUE	FALSE	FALSE	TRUE	TRUE	NA	NA	NA	NA	NA	0	
TRUE	FALSE	TRUE	FALSE	FALSE	NA	NA	NA	NA	NA	0	
TRUE	FALSE	TRUE	FALSE	TRUE	NA	NA	NA	NA	NA	0	
TRUE	FALSE	TRUE	TRUE	FALSE	NA	NA	NA	NA	NA	0	
TRUE	FALSE	TRUE	TRUE	TRUE	NA	NA	NA	NA	NA	0	
TRUE	TRUE	FALSE	FALSE	FALSE	0.596	0.295	0.140	0.054	0.006	2	Not enough data
TRUE	TRUE	FALSE	FALSE	TRUE	0.999	0.610	0.247	0.073	0.009	8	Connected, far, AREA different, BFI and FARL similar
TRUE	TRUE	FALSE	TRUE	FALSE	0.888	0.505	0.193	0.079	0.007	5	Not enough data
TRUE	TRUE	FALSE	TRUE	TRUE	0.100	0.027	0.016	0.008	0.004	9	Connected, far, similar
TRUE	TRUE	TRUE	FALSE	FALSE	0.225	0.079	0.039	0.028	0.015	8	Connected, near, BFI similar, others different
TRUE	TRUE	TRUE	FALSE	TRUE	0.464	0.199	0.061	0.028	0.008	51	Connected, near, AREA different, others similar
TRUE	TRUE	TRUE	TRUE	FALSE	0.398	0.106	0.045	0.020	0.004	13	Connected, near, FARL different, others similar
TRUE	TRUE	TRUE	TRUE	TRUE	0.100	0.045	0.023	0.013	0.004	102	Connected, near and all similar
FALSE	FALSE	FALSE	FALSE	FALSE	1.000	1.000	1.000	0.995	0.396	40	Disconnected, far, all different
FALSE	FALSE	FALSE	FALSE	TRUE	1.000	1.000	0.991	0.724	0.141	86	Disconnected, far, FARL similar, others different
FALSE	FALSE	FALSE	TRUE	FALSE	1.000	0.997	0.824	0.319	0.028	63	Disconnected, far, AREA similar, others different
FALSE	FALSE	FALSE	TRUE	TRUE	1.000	0.998	0.823	0.363	0.027	159	Disconnected, far, AREA and FARL similar, others different
FALSE	FALSE	TRUE	FALSE	FALSE	1.000	1.000	0.996	0.871	0.337	3	Not enough data
FALSE	FALSE	TRUE	FALSE	TRUE	NA	NA	NA	NA	NA	1	Not enough data
FALSE	FALSE	TRUE	TRUE	FALSE	0.432	0.122	0.059	0.011	0.001	3	Not enough data
FALSE	FALSE	TRUE	TRUE	TRUE	NA	NA	NA	NA	NA	1	Not enough data
FALSE	TRUE	FALSE	FALSE	FALSE	1.000	0.962	0.712	0.354	0.060	258	Disconnected, far, BFI similar, others different
FALSE	TRUE	FALSE	FALSE	TRUE	1.000	0.951	0.550	0.183	0.019	981	Disconnected, far, BFI and FARL similar, AREA different
FALSE	TRUE	FALSE	TRUE	FALSE	0.997	0.772	0.400	0.122	0.013	681	Disconnected, far, BFI similar, AREA and FARL different
FALSE	TRUE	FALSE	TRUE	TRUE	0.997	0.723	0.317	0.095	0.009	2255	Disconnected, far, all similar
FALSE	TRUE	TRUE	FALSE	FALSE	0.949	0.437	0.164	0.061	0.009	6	Not enough data
FALSE	TRUE	TRUE	FALSE	TRUE	0.810	0.310	0.122	0.035	0.007	78	Disconnected, near, BFI and FARL similar, AREA different
FALSE	TRUE	TRUE	TRUE	FALSE	0.707	0.224	0.081	0.034	0.006	36	Disconnected, near, AREA and BFI similar, FARL different
FALSE	TRUE	TRUE	TRUE	TRUE	0.341	0.123	0.049	0.021	0.005	245	Disconnected, near, all similar

For the design events other than the 1% AEP, we have reversed the dependency ratio and assumed a 1% AEP inflow from the Kilmastulla to occur during Shannon design events more frequent than the 5% AEP event. For the less frequent Shannon events we retain the 5% AEP Kilmastulla peak inflow. The 2% AEP Shannon event takes a mid point and assumes also a 2% AEP Kilmastulla inflow. Table 4-14 below shows the peak flows for the MPW model for the Kilmastulla outflow for design events, and the equivalent % AEP of the Kilmastulla peak flow. The MRFS peak flow is increased by 20% AEP, and HEFS an increase of 30%.

Table 4-14: Peak flows for Kilmastulla HEP inflow after routing and dependence analysis

Shannon Design Event % AEP	Equivalent % AEP of Kilmastulla flood	Present Day (m^3/s)	MRFS (m^3/s)	HEFS (m^3/s)
50%	1% AEP	33.29	39.95	43.28
20%	1% AEP	33.29	39.95	43.28
10%	1% AEP	33.29	39.95	43.28
5%	1% AEP	33.29	39.95	43.28
2%	2% AEP	31.40	37.68	40.82
1%	5% AEP	28.71	34.45	37.32
0.5%	5% AEP	28.71	34.45	37.32
0.1%	5% AEP	28.71	34.45	37.32

4.3 Black River

The Black River is a tributary of River Shannon with a catchment area of 21.68km^2 and main stream length of 7.67km. The catchment area has been derived from the FSU database. Black river discharges into the Old Shannon just downstream of Parteen Weir. The FSU Qmed calculated for the river is 6.22m^3/s. The catchment area is shown in Figure 4-14 and Table 4-15 lists the key catchment descriptors. The catchment area has been reviewed through site visits and it is possible that some flow through or under the Ardnacrusha Head Race from the Black River catchment enters the Old Shannon at O'Briensbridge. The source of this flow is highly uncertain and so we have derived a single inflow from the Black River catchment to enter the model in the design events immediately downstream of Parteen Weir.

The Black River is an ungauged catchment and we do not apply any pivotal adjustment factor as the nature of the nearby Coole Gauge on the Kilmastulla catchment is not comparable to the flow conditions of the Black River. The catchment area is just below the threshold for which FSU QMED estimates are recommended. Given that there are no particularly unique features of the catchment, it is appropriate to apply the standard FSU methods.

A pooling group has been derived from FSU portal as presented in Appendix A.3. With the exception of the Catchment Area and S1085 the subject site is representative of the catchment descriptors the pooling group. The variance in S1085 is also likely to be a function of the smaller catchment area, and the typical nature of smaller catchments having steeper S1085 values. No adjustments to the pooling group are necessary. The 1%AEP flow is 12.07m^3/s using the Extreme Value Type 1 growth curve, which has been selected to be consistent with the Kilmastulla analysis. The Black River inflows will be input into the Castleconnell model downstream of the Parteen Weir along with the Kilmastulla outflows.

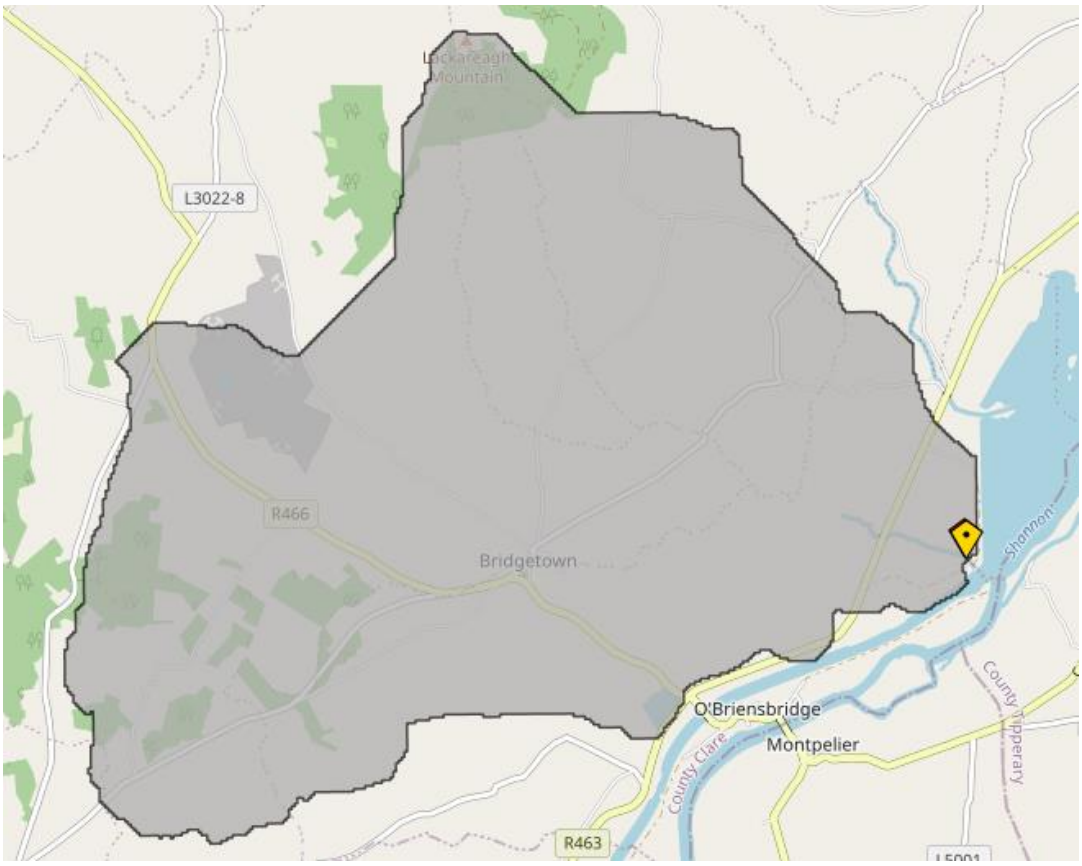

Figure 4-14: Black River FSU catchment area

Table 4-15: Catchment characteristics for the Black River

Descriptor	Black River outlet FSU node 25_3838_4
Area	21.68
SAAR	1343
FARL	1
URBEXT	0
ArtDrain2	0
S1085	13.727
DRAIND	0.919
BFIsoil	0.660
QMED (catchment descriptors unadjusted)	6.22 m^3/s

Table 4-16: Black River peak flow estimates at FSU node 25_3838_4 (EV1)

Annual Exceedance Probability (%)	Growth factor	Peak Flow (m^3/s)
50% (2yr)	1.00	6.22
20% (5yr)	1.25	7.79
10% (10yr)	1.42	8.82
5% (20yr)	1.58	9.82
2% (50yr)	1.79	11.11
1% (100yr)	1.94	12.07
0.5% (200yr)	2.09	13.03
0.1% (1000yr)	2.45	15.26

4.3.1 Hydrograph shape

The volume of the Shannon flow is significantly greater than the Black River inflow contribution and so volume and hydrograph shape is not considered to be critical. For this reason the unadjusted FSU design hydrograph shall be adopted as presented in Figure 4-15.

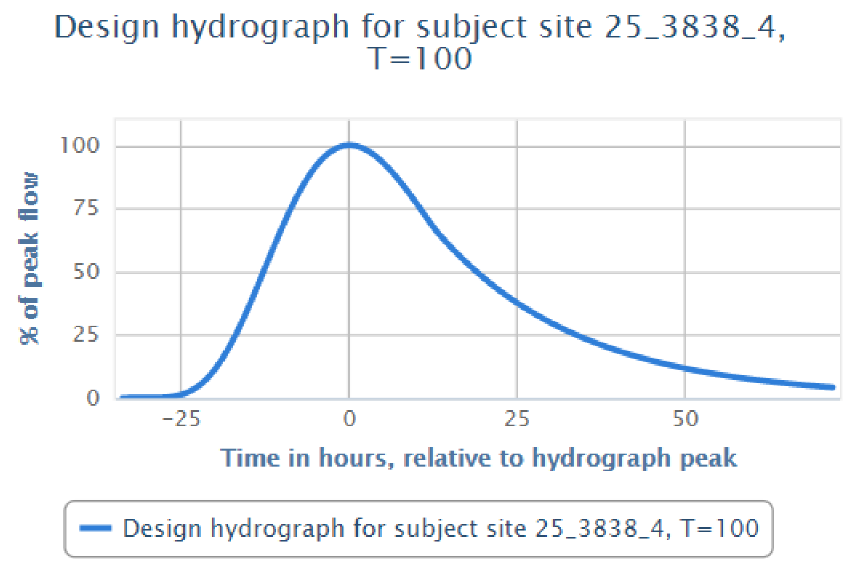

Figure 4-15. Black River FSU hydrograph (unadjusted)

4.3.2 Sensitivity to downstream Boundary Conditions (Shannon flow and level)

The outlet of the Black River is through pipe under the Ardnacrusha Head Race. The effect of the Shannon water level is not considered to have any influence on the rate of discharge from the Black River.

4.3.3 Black River design event inflows

Like the Kilmastulla, the same joint probability combinations apply to the Black River (see Table 4-13). The design event inflows to the Old Shannon are presented in Table 4-17.

The timing of the Kilmastulla inflow shall be set so that peak flow on the Kilmastulla and Shannon coincide. The Black River inflow HEP is located immediately downstream of Parteen Weir on the Old Shannon, at the same location as the Kilmastulla HEP.

Table 4-17: Peak flows for Black River HEP inflow after routing and dependence analysis

Shannon Design Event % AEP	Equivalent % AEP of Black River flood	Present Day (m^3/s)	MRFS (m^3/s)	HEFS (m^3/s)
50%	1% AEP	12.07	14.48	15.69
20%	1% AEP	12.07	14.48	15.69
10%	1% AEP	12.07	14.48	15.69
5%	1% AEP	12.07	14.48	15.69
2%	2% AEP	11.11	13.33	14.44
1%	5% AEP	9.82	11.78	12.77
0.5%	5% AEP	9.82	11.78	12.77
0.1%	5% AEP	9.82	11.78	12.77

4.4 Cedarwood and Stradbally Streams

Two tributaries flow through Castleconnell and enter the Shannon (Figure 4-16). The Cedarwood Stream has a catchment area of 1.28km^2 and a main stream length of 2.58km. The catchment has a complex network of drains and it has been split following a GIS analysis of topography and historical map review (Figure 4-17). The stream starts at the R445 and flows through The Commons and Castlecourt housing estates before entering the River Shannon. There are a number of structures along the length of the stream (6 culverts, 4 bridges and 1 weir surveyed as part of the CFRAM). The drainage maps received from LCCC do not identify any surface water drainage entering the stream.

A review of the catchment boundaries and channel networks has been carried out through site visits.

The Stradbally stream has a catchment area of 3.91km^2 and a main stream length of 2.37km. There are two upstream branches to the Stradbally Stream. The upper branch starts downstream of the R445 and flows west to Castlerock before turning south/west to join with the second branch by Belmont Road. The lower branch starts upstream of the railway line in the Stradbally area and flows north adjacent to the Castlerock estates before entering the Shannon by The Ferry Playground. There are two structures identified along the reach in the CFRAM study, one at the upper reaches under Belmont Hill and one at the downstream end under Chapel Hill. An additional two structures are located under the rail bridge. Stormwater drainage from the Stradbally area, Coolbane Woods discharge to the stream.

The Castleconnell surface water sewer network diverts some runoff out of the Stradbally catchment and discharges directly into the Shannon. The surface water network has been taken into consideration when defining the Stradbally sub-catchments.

Both streams have been modelled in the CFRAM study with inflows to each presented in Table 4-18. The CFRAM hydraulic modelling report found that the Shannon flows are not affected by any inflow from these tributaries.

The hydrological assessment will need to understand if there is any flood risk from these tributaries that needs to be managed by the proposed scheme and if the scheme would cause any additional flooding that needs to be mitigated.

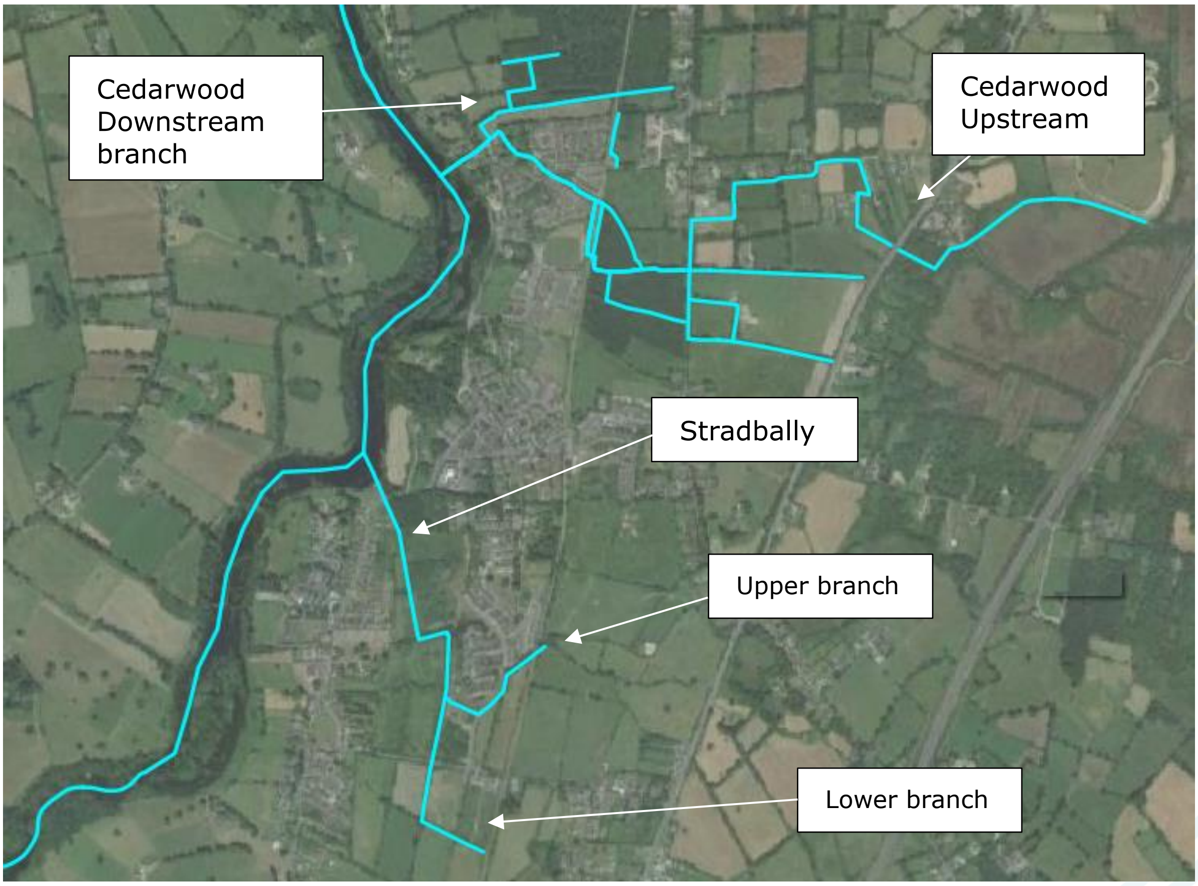

Figure 4-16. Tributaries that flow through Castleconnell village and discharge to the River Shannon. Cedarwood is the Northern stream and Stradbally is the Southern stream.

Table 4-18. CFRAM model inflows for Cedarwood (01CED00870) and Stradbally (01STR01236) Streams

		Design event inflows (m3/s)							
HEP	**Node**	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.1%**
25_3823_6a	01CED00870	0.59	0.77	0.89	1.00	1.15	1.27	1.38	1.64
25_3823_8a	01STR01236	0.74	0.97	1.12	1.27	1.45	1.60	1.74	2.06

4.4.1 Hydrology Calculations

Figure 4-17 and Figure 4-18 show the catchment boundaries for the two watercourses. Three hydrological estimation points (HEP) were calculated for the Cedarwood Stream; one for upstream catchment (Cedarwood Upstream), one lumped (Cedarwood Downstream) and for the branches entering downstream from north (Cedarwood North).

Four HEPs were calculated for the Stradbally; one point inflow for the lower branch (Stradbally South), one point inflow for the upper branch (Stradbally East), one lumped estimate upstream of Belmont Road (Stradbally West), and one of the lumped estimate of the entire catchment the junction with the Shannon (Stradbally). The lumped estimates are used to calculate lateral inflows between the point inflow and the downstream of the

Stradbally. Table 4-19 and Table 4-20 show the catchment characteristics used in the flow estimations. There were no FSU ungauged nodes on either of the tributaries.

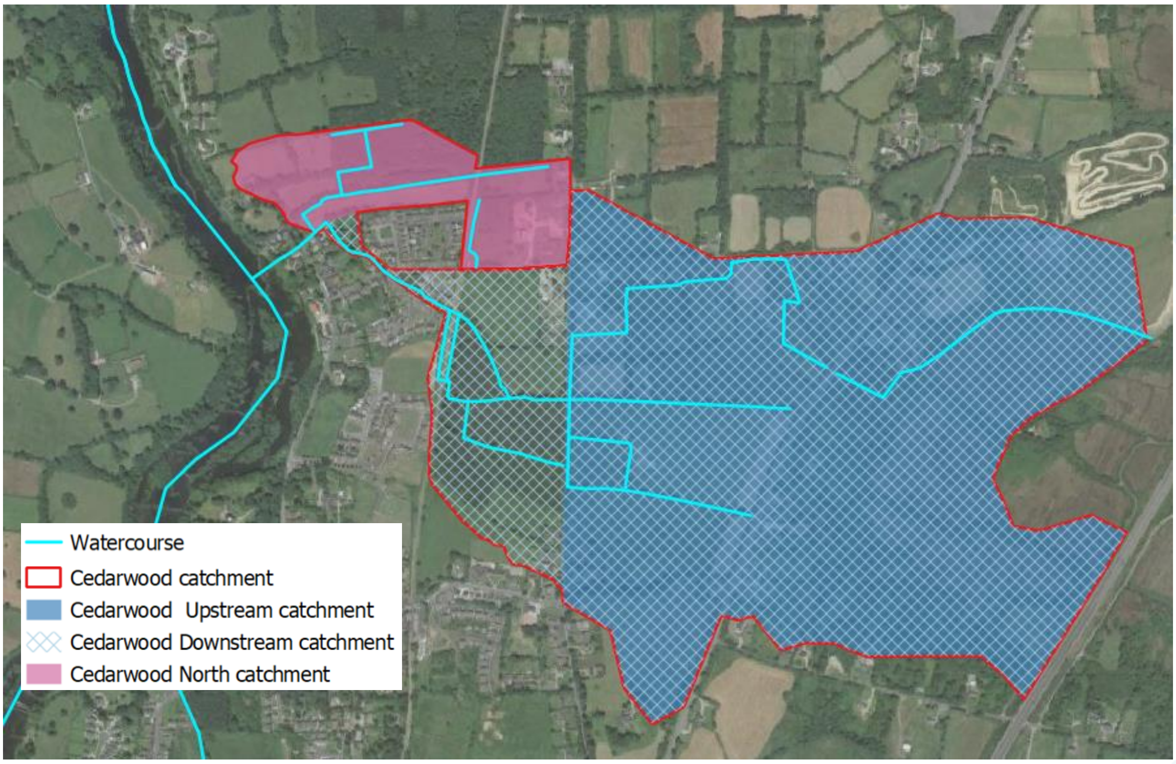

Figure 4-17: Cedarwood catchments

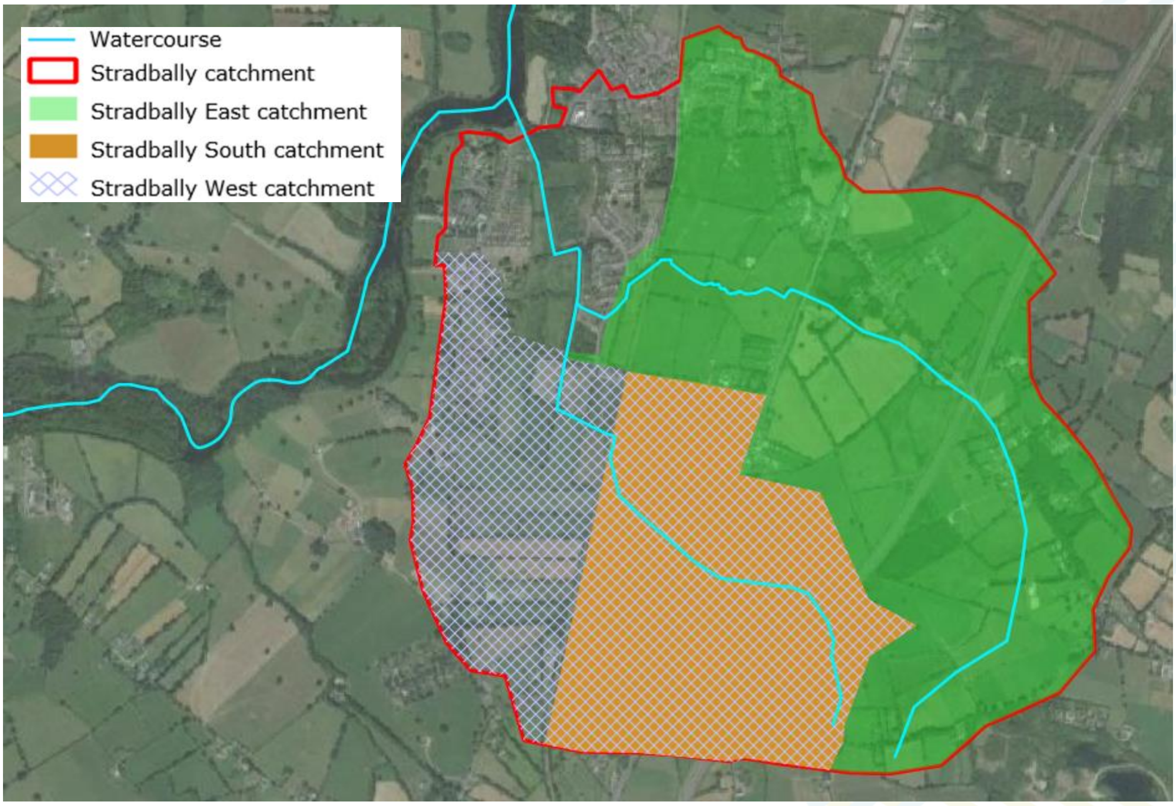

Figure 4-18: Stradbally catchments

Table 4-19: Catchment characteristics for Cedarwood

Descriptor	Cedarwood	Cedarwood Upstream	Cedarwood Downstream	Cedarwood North
Area	1.28	0.98	1.15	0.13
SAAR	1114	1115	1111	1123
FARL	1	1	1	1
URBEXT	0. 26	0.012	0.033	0
MSL	2.583	1.79	2.574	0.58
ArtDrain2	0	0	0	0
S1085	4.252	3.409	4.46	6.788
Soil (number)	2(73%) 3(27%)	2(64%) 3(36%)	2(70%) 3(30%)	2(100%)
SOIL	0.034	0.034	0.034	0.034
M5-2day	50	50	50	50
r	0.3	0.3	0.3	0.3
Runoff Coefficient	0.30	0.37	0.38	0.36
Time of Concentration (hrs)	113min	60mins	57mins	15mins

Table 4-20: Catchment characteristics for Stradbally

Descriptor	Stradbally	Stradbally East	Stradbally West	Stradbally South
Area	3.91	1.92	1.561	0.928
SAAR	1092	1095	1083	1084
FARL	1	1	1	1
URBEXT	0.33	0.06	0.120	0.010
MSL	2.367	1.088	0.234	0.077
ArtDrain2	0	0	0	0
S1085	1.449	2.95	11.84	8.311
Soil (number)	2 (88%) 3 (12%)	2(83%) 3(17%)	2: 1	2:1
SOIL	0.312	0.313	0.3	0.3
M5-2day	50	50	50	50
r	0.3	0.3	0.3	0.3
Runoff Coefficient	0.30	0.37	0.38	0.36
Time of Concentration (hrs)	110mins	57mins	21mins	8mins

4.4.1.1 Hydrological Estimation Points

The estimate peak design flows a variety of methods were considered for the watercourse; Flood Studies Update Small Catchments Methods (FSU SC), Flood Studies Report Rainfall Runoff Method (FSR RR), IH 124 Method and the Rational Method.

The acceptable range of catchment sizes for application of the FSU SC Method is between 1-25km². The catchments in this study are at the lower end of the recommended size, with some below 1km². Therefore we do not apply this method for the Stradbally and Cedarwood catchments. The FSR RR Method generates steep growth curves which are generally suited to smaller upland catchments where the flow regime is very flashy. The catchments in this study would not be typical of those used in the derivation of the FSR RR method. However, the convolution of rainfall with an appropriate Unit Hydrograph would be a suitable means to determine flood estimates. As an alternative check, a more crude assessment can be provided by the Rational Method. This method is very sensitive to Time of Concentration (Tc), which for a rural catchment is difficult to estimate. This method resulted in very different flows for each of the catchments. As a result of these uncertainties, the Rational method is not recommended for calculating final flows.

The IH124 Method should only be considered for very small catchments, such as those below 5km². Generally best suited to small rural or urban catchments, which fits well with the catchments in this study. This is the preferred method for flow calculations for Stradbally and FSR RR for Cedarwood in this study. Additional descriptions for each method can be found in Appendix A.4.

The IH 124 Method was selected as the most suitable method for calculating flows. The peak flows for the methods are shown in Table 4-22 below. Flow validation is also included in the table to ensure the flows are sensible. Sensitivity testing could also be carried out with the 95% Confidence Interval, refer to Table 4-23.

Table 4-21: Cedarwood HEP values (present day)

AEP (%)	Cedarwood	Cedarwood Upstream	Cedarwood Downstream	Cedarwood North
50% (2yr)	0.73	0.35	0.42	0.05
20% (5yr)	0.93	0.44	0.52	0.06
10% (10yr)	1.06	0.50	0.60	0.07
5% (20yr)	1.19	0.56	0.68	0.07
2% (50yr)	1.37	0.65	0.78	0.08
1% (100yr)	1.51	0.72	0.86	0.09
0.5%(200yr)	1.66	0.78	0.94	0.09
0.1% (1000yr)	2.01	0.95	1.14	0.12

Table 4-22: Stradbally HEP values (present day)

AEP (%)	Stradbally	Stradbally East	Stradbally West	Stradbally South
50% (2yr)	2.03	0.63	0.53	0.30
20% (5yr)	2.57	0.80	0.68	0.38
10% (10yr)	2.94	0.91	0.77	0.44
5% (20yr)	3.31	1.03	0.87	0.49
2% (50yr)	3.80	1.19	1.00	0.57
1% (100yr)	4.19	1.31	1.10	0.63
0.5%(200yr)	4.59	1.43	1.21	0.69
0.1% (1000yr)	5.58	1.74	1.47	0.84

4.4.1.2 Hydrograph Shape

The hydrograph shape used in conjunction with the IH 124 peak flows is based on the FSR RR hydrograph, refer to Figure 4-19. Comparisons were carried out of the volumes for the IH124 peak flow (which used the FSR RR hydrograph shape) and the peak flow critical storm duration (using the FSR RR method) for the 1% AEP event. As seen in Table 4-23 and Table 4-24 below, there are some notable differences between the volumes for the catchments, particularly the Cedarwood and Stradbally East catchments. The possible range in hydrograph volumes with different storm durations will be taken into account in the design of flood risk management measures and assessment of the performance of existing structures. higher volumes will have to be taken into consideration when designing any flood defences. The IH124 peak flow estimates are not sensitive to storm duration. The critical duration for the peak flow will be used for flood mapping, damage assessment and to understand the performance of structures on each watercourse. Two additional storm durations were also assessed as part of the study.

The first is the critical storm duration to determine if structures along the watercourse are sensitive to volumes. This was calculated based on the maximum volume for the peak flows within the catchment. The 9-hour storm is the critical storm duration for the peak volume for the Cedarwood and Stradbally catchments. The second storm duration is the total storm duration which represents the maximum volume contained within the Stradbally catchment if there was a high Shannon flow preventing it from discharging. The volumes for each of these methods in the Stradbally catchment is shown in Table 4-25 below. Refer to Appendix A.4.5 for volume calculations.

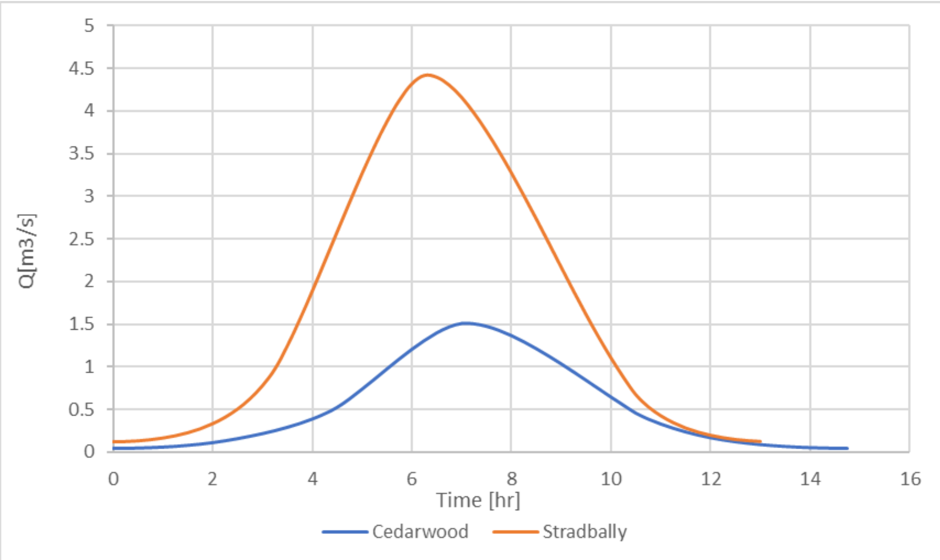

Figure 4-19: Hydrograph shape

Table 4-23: Volume comparison for Cedarwood catchments (9hr storm duration)

VOLUME (m³)	**Cedarwood**	**Cedarwood Upstream**	**Cedarwood Downstream**	**Cedarwood North**
FSSR RR Peak Flow and hydrograph	29,643	23,570	27,480	2,873
IH 124 peak with scaled FSSR RR hydrograph shape	28,556	21,631	24,546	2,051

Table 4-24: Volume comparison for Stradbally catchments (9hr storm duration)

VOLUME (m³)	**Stradbally**	**Stradbally East**	**Stradbally West**	**Stradbally South**
FSRR Peak Flow and hydrograph	87,820	44,769	24,976	12,969
IH 124 peak with scaled FSSR RR hydrograph shape	76,046	34,065	19,964	11,434

Table 4-25: Critical and Total storm duration volumes

Storm Duration	Stradbally		Cedarwood	
	10%	100%	10%	100%
9hr	-	96,954	-	29,643
48hr	114,034	167,260	61,366	82,412
240hr	327,665	424,325	114,426	148,689

4.4.1.3 Design flows

From the flow estimation points, the inflows for the model are calculated. The point inflows will be applied directly to the upstream of the appropriate channels. The lumped estimates are used to calculate the lateral inflows between the point inflows and the downstream of the catchments. Figure 4-20 and Figure 4-21 show where the inflows will be applied. Table 4-28 and Table 4-26 shows the design flows for the model. HEP names are labelled in the maps and tables.

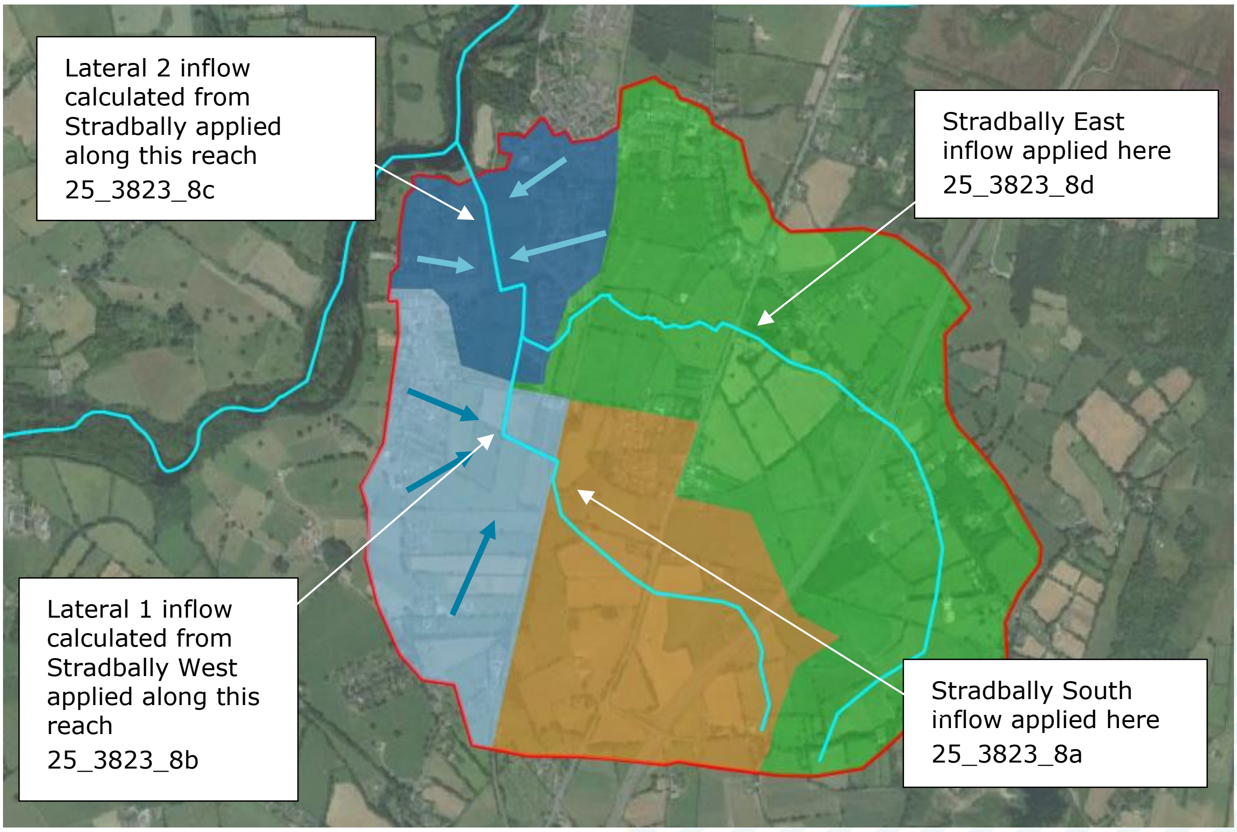

Figure 4-20: Application of flows to model for Stradbally

Table 4-26: Stradbally design event flows

AEP (%)	Stradbally East	Stradbally South	Stradbally Lateral 1	Stradbally Lateral 2
	25_3823_8d	25_3823_8a	25_3823_8b	25_3823_8c
50% (2yr)	0.63	0.30	0.23	0.14
20% (5yr)	0.80	0.38	0.30	0.18
10% (10yr)	0.91	0.44	0.33	0.20

AEP (%)	Stradbally East	Stradbally South	Stradbally Lateral 1	Stradbally Lateral 2
	25_3823_8d	25_3823_8a	25_3823_8b	25_3823_8c
5% (20yr)	1.03	0.49	0.38	0.23
2% (50yr)	1.19	0.57	0.43	0.26
1% (100yr)	1.31	0.63	0.47	0.29
0.5% (200yr)	1.43	0.69	0.52	0.32
0.1% (1000yr)	1.74	0.84	0.63	0.39

Table 4-27: Stradbally 95% confidence interval flows

AEP (%)	Stradbally East	Stradbally South	Stradbally Lateral 1	Stradbally Lateral 2
	25_3823_8d	25_3823_8a	25_3823_8b	25_3823_8c
50% (2yr)	1.72	0.82	0.63	0.38
20% (5yr)	2.18	1.04	0.82	0.49
10% (10yr)	2.48	1.20	0.90	0.55
5% (20yr)	2.81	1.34	1.04	0.63
2% (50yr)	3.25	1.56	1.17	0.71
1% (100yr)	3.58	1.72	1.28	0.79
0.5% (200yr)	3.90	1.88	1.42	0.87
0.1% (1000yr)	4.75	2.29	1.72	1.06

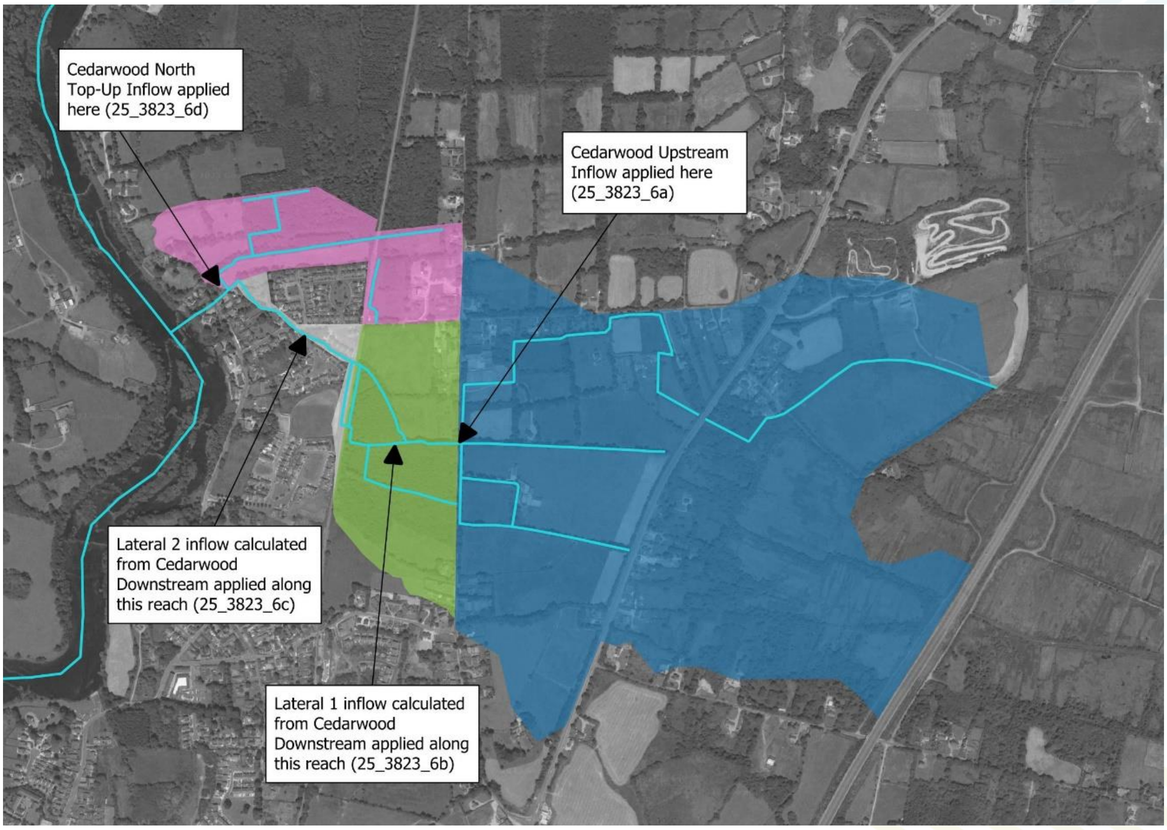

Figure 4-21: Application of flows to model for Cedarwood

Table 4-28: Cedarwood design event flows

AEP (%)	Cedarwood Upstream	Cedarwood Lateral 1	Cedarwood Lateral 2	Cedarwood North
	25_3823_6a	25_3823_6b	25_3823_6c	25_3823_6d
50% (2yr)	0.30	0.05	0.01	0.05
20% (5yr)	0.41	0.07	0.01	0.07
10% (10yr)	0.48	0.08	0.01	0.08
5% (20yr)	0.57	0.09	0.01	0.09
2% (50yr)	0.69	0.11	0.01	0.11
1% (100yr)	0.79	0.13	0.02	0.13
0.5% (200yr)	0.91	0.15	0.02	0.15
0.1% (1000yr)	1.22	0.21	0.03	0.20

Table 4-29: Cedarwood 95% confidence interval flows

AEP (%)	Cedarwood Upstream	Cedarwood Lateral 1	Cedarwood Lateral 2	Cedarwood North
	25_3823_6a	25_3823_6b	25_3823_6c	25_3823_6d
50% (2yr)	0.81	0.14	0.03	0.14
20% (5yr)	1.11	0.19	0.03	0.19
10% (10yr)	1.30	0.22	0.03	0.22
5% (20yr)	1.54	0.24	0.03	0.24
2% (50yr)	1.86	0.30	0.03	0.30
1% (100yr)	2.13	0.35	0.05	0.35
0.5% (200yr)	2.46	0.41	0.05	0.41
0.1% (1000yr)	3.29	0.57	0.08	0.54

4.4.2 Climate change

Climate change projections specified in the project specification are for an increase in peak flows by 20% for the MRFS and 30% for the HEFS. The peak flow is the only parameter relevant to Stradbally and Cedarwood catchment, e.g. sea level not relevant, no significant forestry in the area. Climate change runs are only required for the 1% AEP event. Table 4-30 presents the climate change flows for the 1% AEP short duration event.

Table 4-30: Climate change flows for the 1% AEP event for Cedarwood

Climate change projection	Cedarwood Upstream	Cedarwood Lateral 1	Cedarwood Lateral 2	Cedarwood North
MRFS (+20%)	1.94	0.16	0.02	0.16
HEFS (+30%)	2.11	0.17	0.03	0.17

Table 4-31: Climate change flows for the 1% AEP event for Stradbally

Climate change projection	Stradbally East	Stradbally South	Stradbally Lateral 1	Stradbally Lateral 2

MRFS (+20%)	1.57	0.76	0.56	0.35
HEFS (+30%)	1.70	0.82	0.61	0.38

4.5 Intermediate flow contributions

The intermediate catchment area between Parteen Weir and Castleconnell upstream of the Cedarwood Stream (excluding the Kilmastulla River and Black River) increases by a total of 18.4km^2. This extra area is 0.2% of the total upstream contributing catchment area to Castleconnell (10,820 km^2). This is insignificant additional inflow on top of the total Shannon flow to Parteen Weir.

The best and most reliable method of routing flow to intermediate HEP locations along the Shannon is through the use of the detailed hydraulic model. Forcing the model to calibrate to HEP peak flow estimates based on some form of FSU catchment descriptor adjustment would increase uncertainty in the model and so should not be carried out.

4.6 Downstream boundary

The downstream boundary will be a function of hydraulic conveyance effects only because there are no downstream hydrometric gauges with reliable flow estimates.

The Mulkear River is not expected to have any influence on water levels at Castleconnell. This was confirmed in the CFRAM study and will be validated with sensitivity testing on the hydraulic model. If the sensitivity tests suggest there could be an influence, then an assessment of Mulkear flow contributions will be determined.

4.7 Hydrological Estimation Points (HEPs)

The hydrological analysis above has estimated peak flow, hydrograph shape and coincidence for the Total Shannon flow upstream of Parteen Weir (section 4.1), the Kilmastulla which discharges into the Old Shannon downstream of Parteen Weir (section 4.2) and the Cedarwood and Stradbally Streams which discharge into the Shannon in Castleconnell (section 4.4). Each of these points form Hydrological Estimation Points (HEPs).

There is no requirement in the analysis for catchment descriptors for the Shannon and so these do not need to be reviewed, because Parteen Basin and Lough Derg are significant hydrological breaks in the Shannon system. Catchment descriptors relevant to rational and rainfall-runoff methods will be reviewed for the tributaries.

The catchment boundary for the Kilmastulla River has been reviewed and does not require any update. There is no requirement for intermediate flow contributions (section 4.5) between the Shannon HEPs. Given the small intermediate catchment areas, lateral inflows are less than 1% of the Parteen Weir Shannon peak flow, there is no requirement for lateral inflows. There are no offtakes downstream of Parteen Weir that need to be considered.

The downstream boundary (section 4.6) does not require an HEP as this is set by hydraulic controls.

An additional HEP is required on the Old Shannon immediately downstream of Parteen Weir to account for the effect of Ardnacrusha operation on the flow that passes over Parteen Weir. The HEP peak flow estimates at this HEP will be significantly influenced by different operating conditions at Ardnacrusha and associated assumptions. This HEP will be used as the model inflow boundary, with different peak flow estimates for different operating conditions.

Section 5 of the hydraulics report details the application of hydrology.

The influence of Ardnacrusha operating conditions on discharge upstream of Parteen Weir and on the Old Shannon downstream of Parteen Weir is discussed in section 4.1.6.

The discharge along the Shannon downstream of Parteen Weir at the upstream and downstream extent of the 2D model domain (as documented in the Hydraulics Report) are derived from modelled hydraulic routing of the flood wave on the Old Shannon. There is no additional benefit to a separate routing model for deriving HEP flow estimates and then forcing the more detailed hydraulic model to fit this.

The following HEP locations are required for the Castleconnell FRS hydraulic modelling and scheme development (Figure 4-22):

- Total Shannon upstream of Parteen Weir (HEP ref: 25075)
- Old Shannon downstream of Parteen Weir – upstream boundary condition for hydraulic model (HEP ref: 25_3886_1)
- Inflows from the Kilmastulla river and Black River, just downstream of Parteen Weir (HEP ref: Kilmastulla - 25_3881_10 and Black River - 25_3838_4)
- The Stradbally Stream and Cedarwood Stream will not be input into the Shannon model. The purpose of these flow estimates is to understand whether there is any risk from these watercourses and to determine potential volume of runoff that needs to be managed in the flood risk scheme options. The location of these HEPs are presented in Figure 4-20 and Figure 4-21.

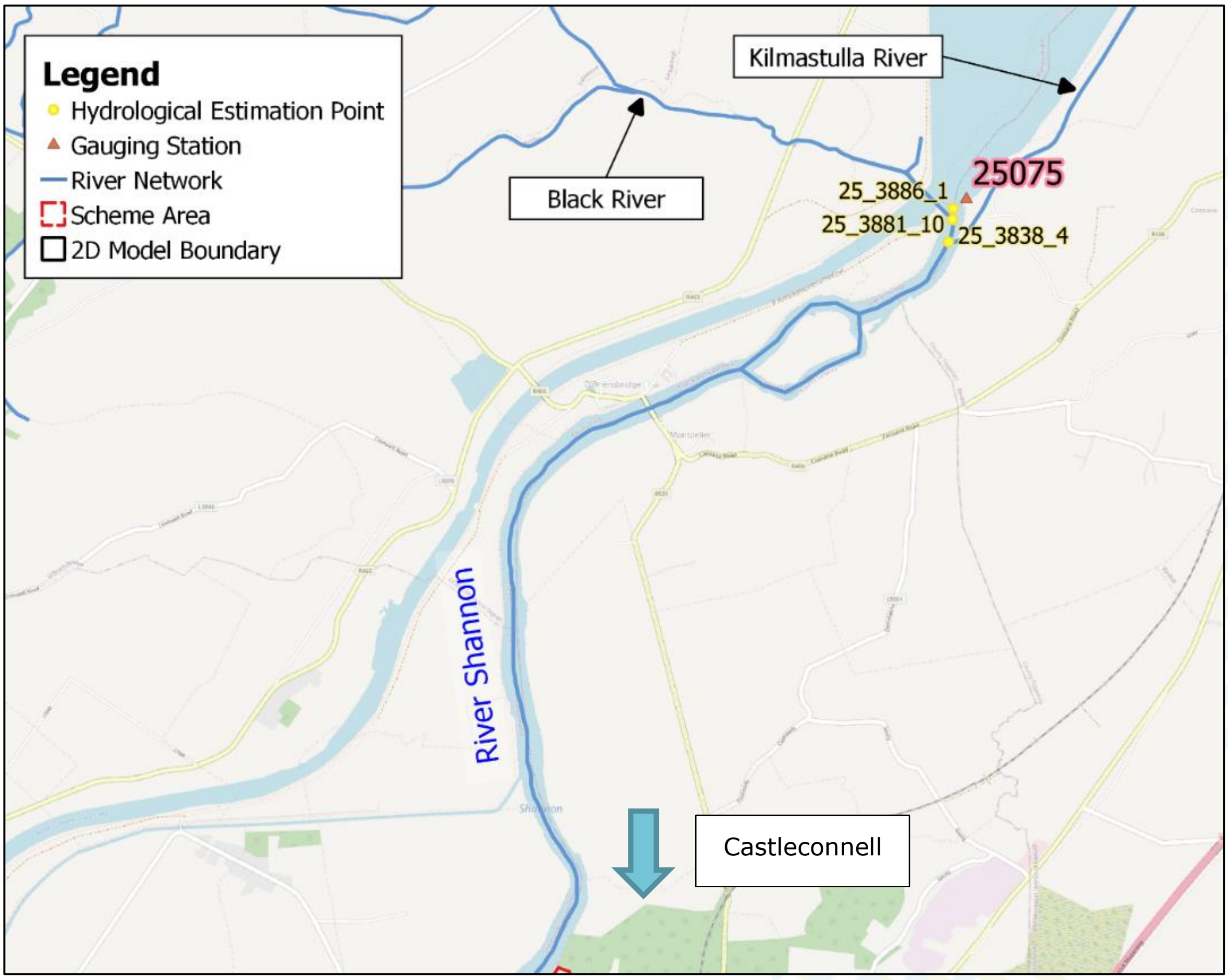

Figure 4-22. HEP locations for the Old Shannon

4.7.1 Flow estimates for the Old Shannon downstream of Parteen Weir (HEP 25_3886_1)

Flow estimates for this HEP 25_3886_1 are strongly influenced by the assumptions relating to the operation of turbines and spillway at Ardnacrusha.

During flood events under "standard operational conditions", we have assumed four turbines are in operation and $345m^3/s$ is regulated to the turbines. The headrace flow assumption of $345m^3/s$ is based upon previous estimates in the Shannon CFRAM studies as informed by the ESB. The operational conditions of the power station were discussed in a meeting held between JBA, ESB, OPW and LCCC on 22/04/20. In this meeting the ESB advised that in high flow conditions, $345m^3/s$ can be delivered down the head race to the power station, but a number of factors should be taken into account and this is not a fixed quantity and could be lower. With this assumed head race flow a **"504" Event** was established for the River Old River Shannon at the HEP downstream of Parteen Weir (HEP ref 25_3886_1), with a 1% AEP peak flow of 504 m3/s. This flow is similar in scale to that experienced in the 2009 flood event.

For the purpose of the design of the Castleconnell FRS, an allowance has been made for operational conditions at Ardnacrusha that could, within reasonable contemplation, occur. In the event of one turbine being out of operation for maintenance or as a result of a mechanical failure, ¾ of the $345m^3/s$ ($258\ m^3/s$) has been assumed to continue down the head race and the rest, ¼ ($87m^3/s$) would pass over Parteen Weir into the Old River Shannon. In a planned situation, a spillway can be opened at Ardnacrusha and the flows along the canal maintained. However, as the spillway is not automatic, in an unplanned situation it cannot pass the full flow immediately. Therefore, a reduced flow down the head race must be considered in the design of the scheme. This scenario was discussed with ESB and based on their past operational experience the design team adopted a suite of operational conditions to define the potential uncertainties within the design flow. Extended turbine maintenance has been necessary during previous flood seasons , in February/March 2020 for example, where one turbine was out of commission during the 2022 winter season. This supports why the design team has had to consider the headrace inflow quantum carefully in selecting the design flow in the Old River Shannon.

These limitations in operational conditions outlined above will result in greater discharge passing over the weir at Parteen into the River Shannon resulting in a 1% AEP peak flow of $591\ m^3/s$. This is adopted as the **Baseline Design Event** for the River Shannon at the HEP downstream of Parteen Weir (HEP ref 25_3886_1).

This approach has been adopted to ensure that appropriate contingency is accommodated in the design of the flood relief scheme to afford a high level of flood protection to Castleconnell Village and the scheme area, allowing for limitations in operational conditions at the power station.

These determine the peak flow estimates for the Old Shannon downstream of Parteen Weir and have been tested in the hydraulic model as presented in Table 4-32. These scenarios are referred to by the amount of flow regulated in the headrace to the Ardnacrusha power station. Table 4-33 presents the design event flows for each of the operating and climate scenarios.

To give context to this, the 2009 event experienced in Castleconnell was approximately the 1% AEP peak ($504m^3/s$) which occurred during "standard operational conditions" at Ardnacrusha.

Table 4-32. 1% AEP Old Shannon flow downstream of Parteen Weir (HEP ref: 25_3886_1) based on operational conditions at Ardnacrusha

Headrace flow to Ardnacrusha (m^3/s)	1% AEP peak flow Old Shannon (m^3/s)	Name and description of operational conditions at Ardnacrusha
345	504	**Standard operational conditions ("504" Event)** The 504 m^3/s flow is comparable to the 2009 flood event peak flow in the Old Shannon. This is the residual flow after ~345 m^3/s is regulated to the headrace from the 1% AEP total Shannon upstream of Parteen Weir. All turbines in operation, previous scale of inflow to Ardnacrusha during flood conditions without any operational limitations.
258	591	**Limitations in operational conditions (Baseline Design Event)** Addresses operational uncertainty and represents possible situations such as: - 1 turbine down with Ardnacrusha spillway not in operation, or - 2 turbines down with spillway in operation - reduced inflow along headrace as a result of wind set up conditions increasing the hydraulic gradient along the canal or reduced throughput at the station due to high tide levels at the outfall
0	849	**Complete Outage** Assumes complete operational failure of Ardnacrusha or head race system with the total Shannon upstream of Parteen Weir (HEP ref: 25075) passing over Parteen Weir

Table 4-33. Old Shannon flows downstream of Parteen Weir (HEP ref 25_3886_1)

	HEP and scenario peak flow estimates (m^3/s)			
HEP	Total Shannon Upstream of Parteen Weir (HEP ref: 25075)	Old Shannon downstream of Parteen Weir HEP ref: 25_3886_1)		
Name of scenario		The "504" event (standard operational conditions)	Baseline design event (limitations in operational conditions)	Complete Outage
Inflow to Ardnacrusha HEP	N/A	345	258	0
50%	526.9	181.9	268.9	526.9
20%	626.3	281.3	368.3	626.3
10%	685.5	340.5	427.5	685.5
5%	738.5	393.5	480.5	738.5
2%	803.2	458.2	545.2	803.2
1%	849.4	**504.4**	**591.4**	**849.4**
0.5%	894	549	636	894

HEP and scenario peak flow estimates (m³/s)				
HEP	Total Shannon Upstream of Parteen Weir (HEP ref: 25075)	Old Shannon downstream of Parteen Weir HEP ref: 25_3886_1)		
Name of scenario		The "504" event (standard operational conditions)	Baseline design event (limitations in operational conditions)	Complete Outage
Inflow to Ardnacrusha HEP	N/A	345	258	0
0.1%	993.5	648.5	735.5	993.5

For the **Climate Change** scenarios, the increase in flow (20% for the MRFS and 30% for the HEFS) is applied to the Total Shannon flow upstream of Parteen Weir (HEP ref: 25075) and then the Ardnacrusha headrace flow of 345 m³/s deducted. This estimate assumes there is no natural or artificial change in the routing or attenuation or alteration of operating procedures throughout the Shannon Basin in response to climate change impacts upstream. This is to avoid compounding uncertainty scenarios in the climate change analysis.

Table 4-34. Shannon HEP Climate change scenario flows

	HEP and scenario peak flow estimates (m³/s)					
HEP	Total Shannon Upstream of Parteen Weir (HEP ref: 25075)			Old Shannon downstream of Parteen Weir HEP ref: 25_3886_1)		
Name of scenario	n/a			"504" event		
Climate scenario	Present day	MRFS	HEFS	Present day	MRFS	HEFS
50%	526.9	632.3	685.0	181.9	287.3	340.0
20%	626.3	751.6	814.2	281.3	406.6	469.2
10%	685.5	822.6	891.2	340.5	477.6	546.2
5%	738.5	886.2	960.1	393.5	541.2	615.1
2%	803.2	963.8	1044.2	458.2	618.8	699.2
1%	**849.4**	**1019.3**	**1104.2**	**504.4**	**674.3**	**759.2**
0.5%	894	1072.8	1162.2	549	727.8	817.2
0.1%	993.5	1192.2	1291.6	648.5	847.2	946.6

4.8 Summary of changes in data since completion of previous studies

The following summarises the changes since previous CFRAM studies were completed:

- Further gauge data is now available (updated AMAX, etc.) – all AMAX series are based on data collected up to and including 2018 water year.
- The 2015 flood event is now be fully included in the analysis. A full review of statistical distributions has been carried out to derive the Parteen Weir growth curve.
- Access to urban drainage system data (outfall locations into watercourses) – this is used to refine flood estimates for the Cedarwood and Stradbally streams.

- New CORINE 2018 landcover dataset is available. This does not alter inflow estimates and so will not be reviewed.
- Inclusion of the Kilmastulla and Black River inflows to the Old Shannon.

4.9 Uncertainties

The following uncertainties have been considered in the hydrological assessment.

- Appropriate sensitivity and uncertainty parameters will be derived to inform the Castleconnell FRS design.
- Parteen Weir AMAX flow record is not purely a function of natural runoff and flow response to rainfall and antecedent catchment conditions. Artificial management and control of Parteen Basin levels influences flow rate. No sub-daily flow data or turbine operation data has been provided.
- Uncertainty relating to the operation of Ardnacrusha has been considered in the selection of the baseline design event. Previous inflows assumed along the headrace are based on conditions unique to the 2009 event. It should be noted that there is uncertainty in this flow arising from the wind conditions along the canal for an event, the influence of the sluices at Parteen drawing levels down further and reducing the flow to the turbine. Therefore, these uncertainties are of significance when approaching the calculation of the design flow, in combination with the operational uncertainties associated with on-status of mechanical equipment.
- Given the unique nature of the Shannon at this location, there are no appropriate donor sites for data transfer and the use of pooling groups will not improve confidence in the fitting of extreme value distributions to the single site AMAX record. The use of other gauge records on the Shannon would not introduce any additional information, and may in fact reduce confidence in flood flow estimates at Parteen Weir, because they do not share the same flow regime. The growth curve is sensitive to the selection of the statistical distribution.
- It is expected that the Kilmastulla, Black River and Shannon respond independently. There is a low likelihood that a coincident Kilmastulla and/or Black River 1% AEP event could occur at the same time as a Shannon 1% AEP event. Such an occurrence will have a probability less than the 1% AEP, but will be modelled as a sensitivity test to understand residual risks. The 1% AEP design event shall use a 1% AEP Old Shannon flow with 5% AEP Kilmastulla and Black River hydrographs. The inverse relationship shall apply to more frequent Shannon floods.
- Coole gauge does not give a reliable flow estimate, and so has been treated with caution. The Black River flow estimation is based on unadjusted ungauged catchment descriptor peak flow estimates.
- To help determine overall uncertainty in flood levels to determine appropriate freeboard for proposed flood defences, an understanding of the uncertainty in the hydrological analysis will be required. This will be in the form of confidence intervals for peak flow for a range of different flood probabilities.
- Uncertainty in relation to morphological change of the Shannon through Castleconnell will be assessed in the hydraulic modelling.

It should be noted that summing all the uncertainties in the hydrology would lead to a significantly larger design flow, greater than experienced in 2009. This has a nominal return period of 1 in 100 years or a 1% AEP event. However, management of these uncertainties by operational foresight, accepting that failure of mechanical equipment may occur and the hydraulics at Parteen Weir have not been tested above the 2009 event does lead to a balanced and proportionate inclusion of flow uncertainty in the selection of design flow. The bounds of this uncertainty are described further in the Options Report, as they impact on defence design levels, and how they are included in the freeboard or factor of safety for the defences.

5 Design event flow estimates

5.1 River Shannon

For the Total Shannon upstream of Parteen Weir HEP peak flow estimates we propose a single site statistical method of the Parteen Weir gauge data, using the full 86 years of AMAX series. The ESB rating is not available for review.

The Log-Normal 2 extreme value distribution is appropriate to use to derive single site growth curve from the Parteen Weir AMAX record series. This decision has been subject to comprehensive analysis and review.

The Old Shannon downstream of Parteen Weir HEP peak flow estimates are based on adjustments to reflect a range of possible operating conditions and the associated flow regulated to Ardnacrusha during a flood event. The headrace flow was estimated to be 345 m^3/s in the CFRAM and used in the Castleconnell scheme analysis. However, in any system where river regulation is the subject of operational controls by a third party and involves the reliability of hydro-mechanical equipment, a fixed flow along the canal is not appropriate. Hydrological scenarios have been introduced whereby the design flow reflects all possible operational outcomes at Parteen Weir. The effect of these operational conditions at Ardnacrusha are considered in detail in the hydraulic modelling report.

Hydrograph shapes are to be based upon the daily flow hydrograph with a uniform rate of flow regulation to Ardnacrusha, for all timesteps where flow down the old Shannon is greater than $10m^3/s$.

Climate change shall be represented by a 20% increase in flood flows for the MRFS and 30% for the HEFS. The actual change in Old Shannon flows is greater than the total 20% increase in the inflows due to the adopted finite capacity of the headrace and the turbines operated by the ESB.

A possible worst-case flow scenario where the Ardnacrusha Power Station headrace does not convey any flow has been derived to inform the residual risk that will need to be managed outside of the engineering components of the proposed scheme. Appropriate residual risk scenarios will be considered in the hydraulic modelling report.

The hydraulic model will consider the potential for debris blockage or sedimentation during an extreme event. There is no need to address this in the hydrological analysis.

Table 4-33 and Table 4-34 Table 4-34. Shannon HEP Climate change scenario flows present the Shannon HEP flow estimates for the range of operational conditions and climate change scenarios.

5.1.1 Available calibration data

There are no level or flow gauges in Castleconnell to calibrate the hydraulic model. Observed flood extents and subsequently surveyed spot levels can be used to validate model performance.

Model validation shall be based on matching the observed flood extents and indicative depths of flooding in the 2009, 2015/16 and 2020 events. This validation will need to consider the effect of any channel morphological change during or since the 2009 event and how this may influence flood flows and levels. There are number of spot wrack levels along the Shannon past Castleconnell and that will be confirmation of the scale of flows that passed down the Shannon in these flood events.

Inflow hydrographs for 2009, 2015/16 and 2020 validation events should be based upon the daily flow hydrographs for the Old Shannon as presented in Figure 4-6.

5.1.2 Residual risk flows

The hydraulic model report will include an assessment of the residual risk from different operational conditions of Ardnacrusha, Parteen Weir and the Head Race channel.

Parteen Weir has been designed to convey the 10,000-year flood. This extreme event is for dam safety and does not need to be considered as it is in excess of the scheme design.

5.2 Kilmastulla and Black River

The Kilmastulla MPW (model N12 reach 15) CFRAM hydrology and hydraulic model has been refined and used to route the flow from Coole gauge to the confluence with the Shannon.

The CFRAM hydrograph shapes and growth curve remain valid and appropriate for this study. The Coole gauge rating curve cannot be improved any further, without additional spot gaugings by the EPA.

The Black River inflow to the Old Shannon just downstream of Parteen Weir is also included in the inflow boundaries to the hydraulic model.

It is likely that a Kilmastulla or Black River high flow event would occur independent of the Shannon flows. A comparison of the coincident probabilities of Shannon, Kilmastulla and Black Riuver flows has been carried out. The timing of inflows will coincide at the peak of the Shannon flow.

5.2.1 Data for calibration of hydraulic models

The CFRAM MPW model has been calibrated to HEPs and the Coole Gauge, and is comparable to ungauged FSU analysis.

5.2.2 Preliminary design event flows

The preliminary design event flows for the Kilmastulla and Black River are shown in Table 5-1 below. Testing of the downstream boundary conditions on the CFRAM MPW Kilmastulla model showed there was no flow impacts on the Kilmastulla with varying levels in the River Shannon and so these inflows can be during all return periods on the River Shannon. The MRFS 1% AEP climate change scenario flow is also included in the table for reference.

Table 5-1: Kilmastulla and Black River design event flows

Shannon % AEP	Kilmastulla Peak Flow HEP: 25_3881_10 (m^3/s)	Black River Peak Flow HEP: 25_3838_4 (m^3/s)
50%	33.29	12.07
20%	33.29	12.07
10%	33.29	12.07
5%	33.29	12.07
2%	31.40	11.11
1%	28.71	9.82
0.5%	28.71	9.82
0.1%	28.71	9.82

5.3 Cedarwood and Stradbally

Flow estimation methods for the tributaries include the FSR RR for hydrograph shape scaled to IH 124 peak flow estimate and for comparison the Rational methods. The topographic and drainage network catchments are slightly different, with an additional surface water drainage network draining direct to the Shannon. This was taken into account when defining the catchment boundaries for the tributaries for the flow estimates.

5.3.1 Model calibration and validation

There are no gauges present on either water course to calibrate the models. Public consultation, anecdotal and mapped historic event reports has been used as validation.

5.3.2 Preliminary design event flows

The preliminary design event flows for the Cedarwood and Stradbally catchments are shown in Table 5-2 and Table 5-3 below.

Table 5-2: Cedarwood design event flows (m^3/s)

AEP (%)	Cedarwood Upstream	Cedarwood Lateral 1	Cedarwood Lateral 2	Cedarwood North
	25_3823_6a	25_3823_6b	25_3823_6c	25_3823_6d
50% (2yr)	0.30	0.05	0.01	0.05
20% (5yr)	0.41	0.07	0.01	0.07
10% (10yr)	0.48	0.08	0.01	0.08
5% (20yr)	0.57	0.09	0.01	0.09
2% (50yr)	0.69	0.11	0.01	0.11
1% (100yr)	0.79	0.13	0.02	0.13
0.5% (200yr)	0.91	0.15	0.02	0.15
0.1% (1000yr)	1.22	0.21	0.03	0.20

Table 5-3: Stradbally design event flows (m^3/s)

AEP (%)	Stradbally East	Stradbally South	Stradbally Lateral 1	Stradbally Lateral 2
	25_3823_8d	25_3823_8a	25_3823_8b	25_3823_8c
50% (2yr)	0.63	0.30	0.23	0.14
20% (5yr)	0.80	0.38	0.30	0.18
10% (10yr)	0.91	0.44	0.33	0.20
5% (20yr)	1.03	0.49	0.38	0.23
2% (50yr)	1.19	0.57	0.43	0.26
1% (100yr)	1.31	0.63	0.47	0.29
0.5% (200yr)	1.43	0.69	0.52	0.32
0.1% (1000yr)	1.74	0.84	0.63	0.39

5.3.3 Flow volumes

The Cedarwood and Stradbally models will be run with a range of different storm durations to confirm the performance of existing key structures and volume of runoff required to be managed with a proposed scheme in place.

5.3.4 Joint Probability

The Cedarwood and Stradbally streams have significantly smaller catchments than the River Shannon. Flood events in the Shannon can last for prolonged periods so it is possible that during a flood event on the Shannon when the water levels are high, a flood event could also occur on the smaller tributaries. Therefore, it is an acceptable approach to assume high Shannon flows as the downstream boundary to the Cedarwood and Stradbally models.

6 References

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A Flow Calculations

A.1 Coole gauge

Table A- 1: Single-site Analysis

Annual Exceedance Probability (%)	GEV	EV1	LN2	LN3	LO
50% (2yr)	20.93	20.67	20.97	20.94	21.57
20% (5yr)	25.76	25.52	25.78	25.74	25.69
10% (10yr)	28.71	28.74	28.72	28.66	28.10
5% (20yr)	31.37	31.82	31.40	31.31	30.31
2% (50yr)	34.58	35.81	34.72	34.59	33.13
1% (100yr)	36.83	38.81	37.12	36.96	35.22
0.5% (200yr)	38.94	41.79	39.46	39.28	37.29
0.1% (1000yr)	43.38	48.69	44.77	44.51	42.08

Table A- 2: FSU Pooling Group

Annual Exceedance Probability (%)	EV1,Growth Factor	EV1,Peak Flow	GEV,Growth Factor	GEV,Peak Flow
50% (2yr)	1	19.52	1	19.52
20% (5yr)	1.25	24.45	1.25	24.4
10% (10yr)	1.42	27.71	1.41	27.47
5% (20yr)	1.58	30.84	1.55	30.35
2% (50yr)	1.79	34.9	1.74	33.91
1% (100yr)	1.94	37.93	1.87	36.47
0.5% (200yr)	2.1	40.96	1.99	38.94
0.1% (1000yr)	2.46	47.96	2.27	44.36

A.2 FSU Node 25_3881_9 (Kilmastulla Outlet)

Table A- 3: FSU flow estimation, unadjusted

Annual Exceedance Probability (%)	EV1,Growth Factor	EV1,Peak Flow	GEV,Growth Factor	GEV,Peak Flow
50% (2yr)	1	20.62	1	20.62
20% (5yr)	1.25	25.83	1.25	25.78
10% (10yr)	1.42	29.28	1.41	29.04
5% (20yr)	1.58	32.58	1.55	32.06
2% (50yr)	1.79	36.86	1.74	35.82
1% (100yr)	1.94	40.07	1.87	38.53
0.5% (200yr)	2.1	43.27	1.99	41.14
0.1% (1000yr)	2.46	50.67	2.27	46.86

A.3 Black River

Table A- 4: FSU flow estimation

Annual Exceedance Probability (%)	EV1		GEV	
	Growth factor	Peak Flow (m^3/s)	Growth factor	Peak Flow (m^3/s)
50% (2yr)	1.00	6.22	1.00	6.22
20% (5yr)	1.25	7.79	1.25	7.77
10% (10yr)	1.42	8.82	1.40	8.73
5% (20yr)	1.58	9.82	1.55	9.62
2% (50yr)	1.79	11.11	1.72	10.71
1% (100yr)	1.94	12.07	1.85	11.49
0.5% (200yr)	2.09	13.03	1.97	12.23
0.1% (1000yr)	2.45	15.26	2.22	13.84

Table A- 5: FSU portal pooling group members

Station	Euclidean DIST(ij)	# years in FSU database	Cumulative # station-years
30020	0.714	16	16
22009	0.972	24	40
19046	1.108	9	49
20006	1.127	25	74
19020	1.228	28	102
25158	1.33	18	120
36071	1.377	20	140
19016	1.392	11	151
34018	1.446	27	178
25044	1.464	40	218
33070	1.551	28	246
16006	1.553	33	279
25038	1.564	17	296
16013	1.57	33	329
31002	1.573	26	355
16005	1.574	30	385
34011	1.682	30	415
30021	1.694	26	441
29071	1.709	26	467
13002	1.75	19	486
25040	1.78	19	505
27070	1.803	29	534
23012	1.834	18	552
27003	1.849	47	599
12013	1.863	30	629
26010	1.878	35	664
29001	1.882	40	704

Legend: Pooled Auxiliary Selected

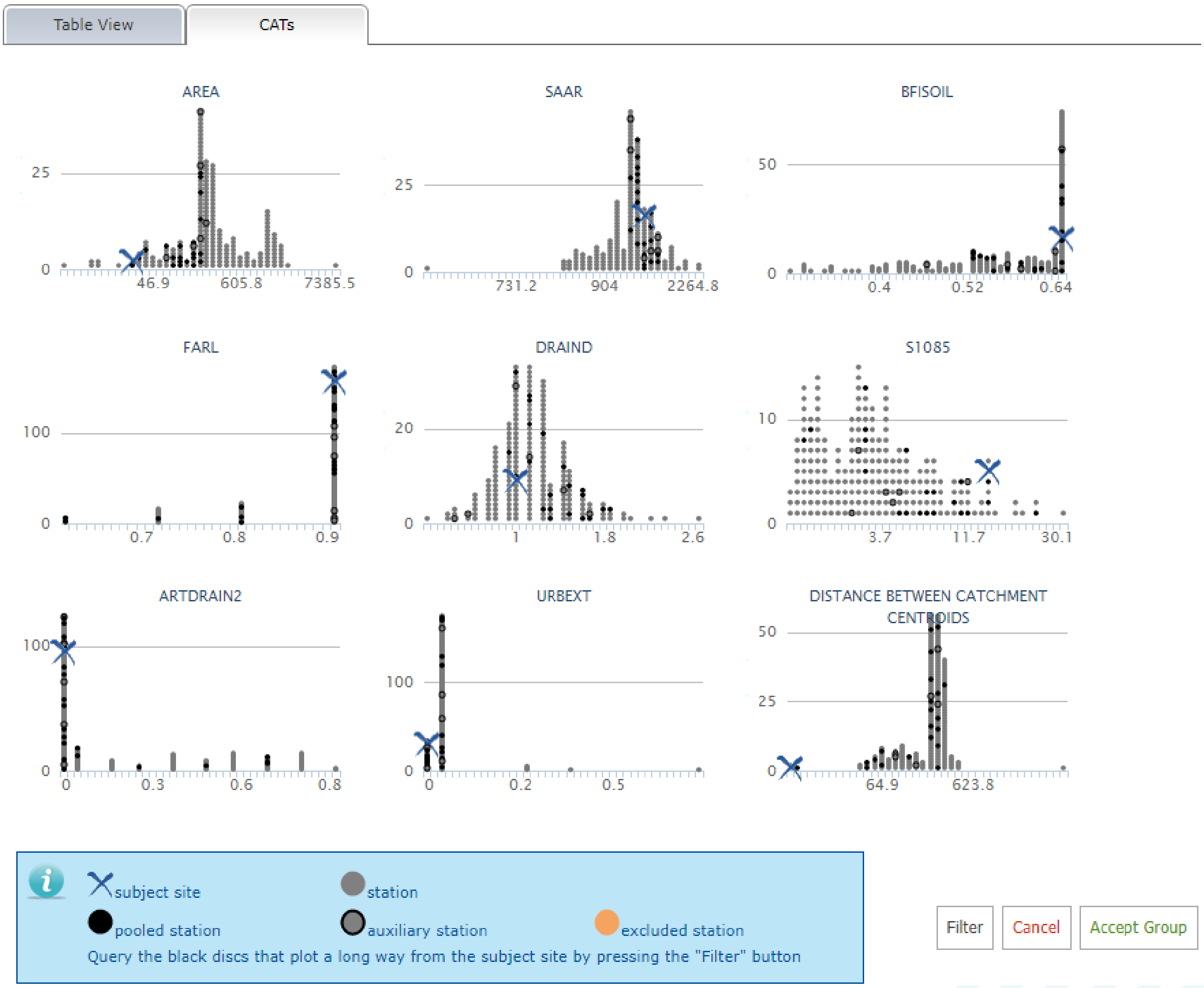

Figure A- 1: Pooling group descriptors

Pooled Group Summary

Number of station-years pooled	509
Number of stations	21
Mean length of A-max records pooled	25
Pooled L-CV	0.147
Pooled L-skewness	0.132
Pooled L-kurtosis	0.142
Return period (y):	100
Distribution:	GEV

Heterogeinity statistics based upon weighted S.D. of the L-CVs

H	30.26
H*	36.2

Heterogeinity statistics based upon average distance from site to the regional average on a graph L-CV VS. L-Skewness

H	1.873
H*	8.304

Heterogeinity statistics based upon average distance from site to the regional average on a graph L-Skewness VS. L-Kurtosis

H	0.942
H*	8.915

Figure A- 2: Pooling Group Summary Statistics

A.4 Flow Calculations - Cedarwood and Stradbally

A.4.1 FSU Small Catchments

The typical FSU method is recommended for use for catchment sizes larger than 25km2. Where catchments are lower than 25km2 it is recommended that the FSU small catchment equation is used, as given below:

$$Qmed = (2.0951*10^{-5})*(AREA^{0.9245})*(SAAR^{1.2695})*(BFI^{0.9030})*(FARL^{2.3163})*(S1085^{0.2513})$$

The above equation provides a Qmed value for the catchment using the values provided in Table 4-20. The growth curve is from the CFRAM Study. FSU SC flows are shown in the table below. This method is recommended for catchments between 1-25km². The catchments in this study are at the lower end of the recommended size, with some below 1km². It is therefore not recommended to use on the catchments.

Table A- 6: Stradbally FSU SC flows

Annual Exceedance Probability (%)	Stradbally (lumped)	Stradbally East (point)	Stradbally South (Point)	Stradbally West (lumped)
50% (2yr)	0.66	0.21	0.11	0.16
20% (5yr)	0.86	0.28	0.15	0.22
10% (10yr)	1.00	0.33	0.17	0.25
5% (20yr)	1.12	0.37	0.19	0.28
2% (50yr)	1.30	0.42	0.22	0.32
1% (100yr)	1.42	0.46	0.24	0.35
0.1% (1000yr)	1.83	0.60	0.31	0.46

A.4.2 FSR RR Method

This method generates steep growth curves which are generally suited to smaller upland catchments where the flow regime is very flashy. The Q100 hydrographs for the FSR RR method are shown in Figure A-1 and A-2 below. The Stradbally hydrograph shape is based on the 6.25hour storm duration as recommended by the method, and the Cedarwood hydrograph is based on the 9.25hour storm duration. The catchments in this study would not be typical of those recommended for the FSR RR method.

Table A- 7: Stradbally FSR RR flows

Annual Exceedance Probability (%)	Stradbally (lumped)	Stradbally East (point)	Stradbally South (Point)	Stradbally West (lumped)
50% (2yr)	1.81	0.43	0.63	1.07
20% (5yr)	2.30	0.59	0.89	1.53
10% (10yr)	2.67	0.69	1.05	1.79
5% (20yr)	3.07	0.82	1.17	1.99
2% (50yr)	3.76	0.99	1.46	2.49
1% (100yr)	4.42	1.14	1.74	2.97
0.1% (1000yr)	7.51	1.78	2.78	4.71

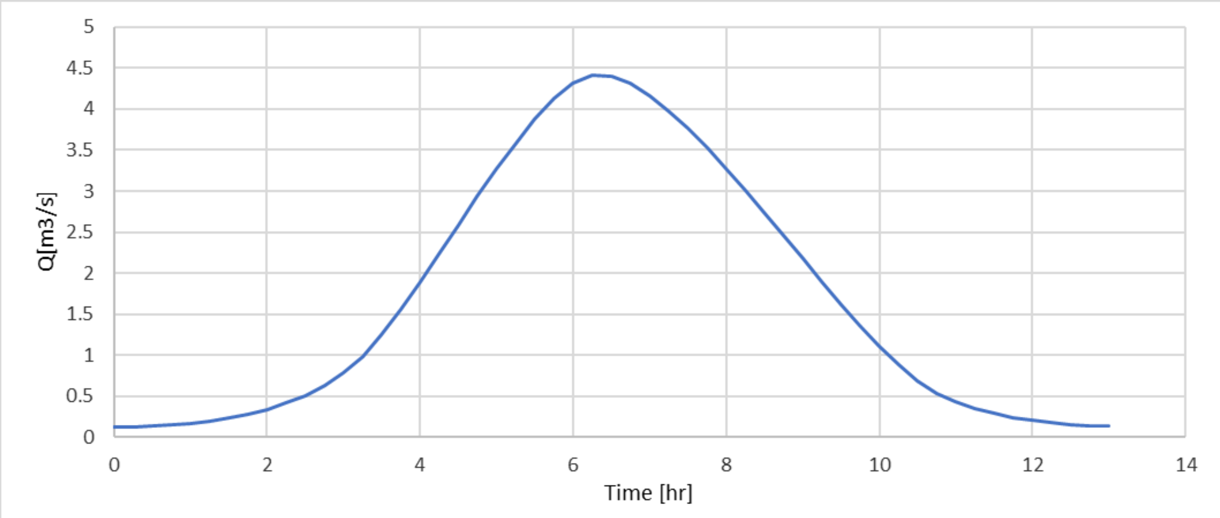

Figure A- 3: Stradbally Q100 FSR RR Hydrograph Shape

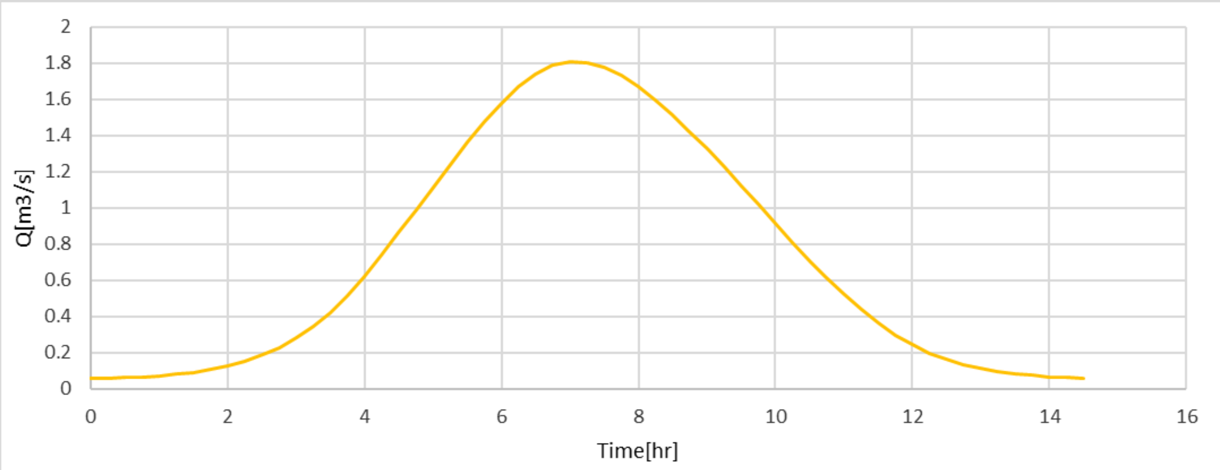

Figure A- 4: Cedarwood Q100 FSR RR Hydrograph Shape

A.4.3 IH 124 Method

This method should only be considered for use in very small catchment, such as those below 5km². Generally best suited to small rural or urban catchments, which fits well with the catchments in this study. This is the preferred method for flow calculations in this study. This method would use the FSR RR hydrograph shape.

Table A- 8: Stradbally IH 124 flows

Annual Exceedance Probability (%)	Stradbally (lumped)	Stradbally East (point)	Stradbally South (Point)	Stradbally West (lumped)
50% (2yr)	2.03	0.40	0.30	0.53
20% (5yr)	2.57	0.51	0.38	0.68
10% (10yr)	2.94	0.58	0.44	0.77
5% (20yr)	3.31	0.65	0.49	0.87
2% (50yr)	3.80	0.75	0.57	1.00
1% (100yr)	4.19	0.83	0.63	1.10
0.1% (1000yr)	5.58	1.10	0.84	1.47

A.4.4 Rational Method

The Rational Method is one of the earliest methods of calculating runoff and uses the following formula:

Q =kCiA

Where k = 2.78 and is the conversion factor from imperial to metric

C = runoff coefficient (dimensionless)

i = Rainfall intensity mm/hr - intensity assumed constant over duration

A = area in ha

The rainfall intensity was calculated from the Time of Concentration and the Depth Duration Frequency (DDF) model from Met Eireann. In natural catchments, the Bransby-Williams equation is used to calculate the Time of concentration (Tc), while the Friends formula is used for overland flows, generally found in upstream sections of catchments where there is no defined channel. The two formulas are shown below and were used in combination over the flow path length to get realistic Tc for the catchment.

Tc natural catchments (Bransby-Williams equation)

$$t_c = \frac{F_c . L}{A^{1/10} S^{1/5}} \qquad (14.6)$$

where,

t_c = the time of concentration (minute)

F_c = a conversion factor, 58.5 when area A is in km^2, or 92.5 when area is in ha

L = length of flow path from catchment divide to outlet (km)

A = catchment area (km^2 or ha)

S = slope of stream flowpath (m/km)

Tc natural catchments (Bransby-Williams equation)

$$t_c = \frac{F_c . L}{A^{1/10} S^{1/5}} \qquad (14.6)$$

where,

t_c = the time of concentration (minute)

F_c = a conversion factor, 58.5 when area A is in km^2, or 92.5 when area is in ha

L = length of flow path from catchment divide to outlet (km)

A = catchment area (km^2 or ha)

S = slope of stream flowpath (m/km)

Table A- 9: Stradbally and Cedarwood Rational Tc

	Stradbally	Stradbally East	Stradbally West	Stradbally South	Stradbally North	Cedarwood
Tc Natural	107	54	18	4	58	52
Tc Overland	3	3	3	4	-	3
Tc Total	**110**	**57**	**21**	**8**	**58**	**55**

Rainfall depths from the estimated Tc were taken for each return period from the DDF models. From these the rainfall intensity was calculated (mm/hr), e.g. for the Stradbally the rainfall depth at 2 hours for the 1% was 37.8mm. This was divided by 2 to get the intensity in mm/hr, 18.9mm/hr.

The runoff coefficient for rural areas and urban areas is based on the table below. For catchments with a mix of land use types the C values (and flow calculations) were applied using area weighting in relation to URBEXT. The C values used in this study are 0.4 for suburban residential and 0.36 for rolling cultivated lands with clay and silt loam.

The table below shows the rational formulas for the 1% AEP event for each catchment. As shown in the table, there is great vary below the flows in the Stradbally catchment. Stradbally is equal to Stradbally East + Stradbally West + remaining catchment area but the Stradbally is estimated as having a much lower flow than the other catchments. This is as a result of differing Tc between the catchments and sub-catchments - Stradbally has a lower Tc and so rainfall intensity and resulting flows are less than smaller catchment with a high Tc. The Stradbally catchment is therefore very sensitive to Tc and the above sub-catchment calculations would not be suitable as final inflows

Table A- 10: Stradbally and Cedarwood Rational flows

	Cedarwood	Stradbally	Stradbally East	Stradbally West	Stradbally South
K	2.78	2.78	2.78	2.78	2.78
C	0.363	0.296	0.368	0.378	0.364
I (mm/hr)	30.4	18.9	30.5	80	102
A (ha)	173.6	390.7	140.3	156.1	92.8
Q 100 (m3/s)	**5.32**	**6.07**	**4.37**	**13.14**	**9.58**

Following the sensitivity of the catchment, the Rational method was used when catchment was divided up into three sub-catchments so there was no overlap in catchments and between the flows calculated: Stradbally South (point inflow, same as previous), Stradbally East (point inflow, same as previous) and the remaining catchment, Stradbally North. Flows were calculated using the same process as described above and results are shown in the table below. However, due to the sensitivity of the Rational method to the catchment characteristics it is not the preferred method to use for inflows to the model.

Table A- 11: Stradbally Updated Rational flows

	Stradbally South	Stradbally East	Stradbally North
K	2.78	2.78	2.78
C	0.364	0.368	0.372
I (mm/hr)	102	30.5	30.5
A (ha)	92.8	140.3	157.4
Q 100 (m3/s)	**9.58**	**4.37**	**4.96**

A.4.5 Volume Estimates

The volume estimates for the two streams were calculated using the FSR RR method. The FRS RR hydrograph shapes were scaled to the IH 124 peak flows (which were selected as the most appropriate). The volume for each IH124 peak flow was compared with the critical duration for the peak flow for each of the sub-catchments (using the FSR RR method). The peak flow critical durations vary between each of the sub-catchments in the Stradbally. The comparison is shown in Table A-7.

Table A- 12: Cedarwood and Stradbally Volume Comparison

VOLUME	Cedarwood	Stradbally (Total)	Stradbally East	Stradbally West	Stradbally South
FSRRR Peak Flow Critical Duration	40651	87820	32739	24976	12969
IH 124	29047	76046	15064	19964	11434

To assess the sensitivity of the structures to volume, the critical storm duration for the total Stradbally and Cedarwood catchments was calculated. Table A-8 below shows the volumes for each of the peak flows for each of the catchments. From this, the 9-hour storm duration estimates the maximum volume and so this is used as the critical storm duration for the catchment. The 9-hour storm is also the critical storm duration for the Cedarwood catchment.

In the event of a high flow on the Shannon and the tributaries could not discharge into the Shannon, the volumes for the total storm duration was also calculated for the tributaries. Table A-9 below shows the volumes for a range of storm durations for the catchments for both the 10% and 1% AEP events. The 600-hour (25day) storm gives the highest volume for the catchments. In a worst-case scenario, these would be the volumes contained in the tributaries if unable to discharge to the Shannon.

Table A- 13: Critical Storm Duration

Peak Critical Storm Duration (hrs)	Stradbally (Total)	Stradbally East	Stradbally West	Stradbally South
9	**95954**	32739	34385	20114
7	87820	29119	31774	18542
4	71415	23053	24976	15655
3	64052	20639	22279	12969

Table A- 14: Total Storm Duration

Strom Duration	Stradbally		Cedarwood	
	10%	**100%**	**10%**	**100%**
9hr (Critical)	-	96,954	-	40,651
48hr (Total)	114,034	167,260	50,778	74,952
240hr (Total)	327,665	424,325	148,673	193,306
600hr (Total)	683,500	841,266	312,552	386,413

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