Lower Lee (Cork City) Drainage Scheme
(Flood Relief Scheme)

Hydrology Report

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JBA Project Manager

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Final June 2016	Incorporates comments made by Mark Hayes by email 8 June 2016 on the Forecasting report (Section 7 of this report was included within it as an appendix)	ARUP, OPW
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Contract

This report describes work commissioned by the Office of Public Works, by a contract signed in September 2013 by Arup, with JBA Consulting operating as sub-contractors under Trading Agreement SC003826. OPW's representative for the contract was John Kelly. Paul Wass, Rosie Hampson, David Forde and Joanne Cullinane of JBA Consulting carried out this work.

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Purpose

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Abbreviations

1D	One Dimensional (modelling)
AEP	Annual Exceedance Probability
AMAX	Annual Maximum
BFI	Base Flow Index
BSM	Broad-Scale Modelling
CEH	Centre for Ecology and Hydrology
CFRAM	Catchment Flood Risk Assessment and Management
CFRAMS	Catchment-Based Flood Risk Assessment and Management Study
CIWEM	The Chartered Institution of Water & Environmental Management
CS	Cross Section
DDF	Depth Duration Frequency
DEFRA	Department of the Environment, Food and Rural Affairs (formerly MAFF)
EA	Environment Agency
EC	European Community
EPA	Environmental Protection Agency
ESB	Electicrity Supply Board
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FEWS	Flood Early Warning System (flood forecasting software developed by Delatres)
FSR	Flood Studies Report
FSU	Flood Studies Update
GEV	General Extreme Value Distribution
GL	General Logistic Distribution
GPD	Generalised Pareto Distribution
HEC-RAS	Hydrologic Engineering Center – River Analysis System (developed by the US Army)
HEP	Hydrological Estimation Point
HR	Hydraulic Research, Wallingford
ICE	The Institution of Civil Engineers
ISIS	Hydrology and hydraulic modelling software
LMAX	Maximum observed level
LMED	Median Annual Level (with return period 2 years)
MAFF	Ministry of Agriculture Food and Fisheries (now part of Defra)
NRA	National Rivers Authority
ODPM	Office of the Deputy Prime Minister
OPW	Office of Public Works
PDM	Probability Distributed Model

PE	Potential Evaporation
PR	Percentage Runoff
Q100	Flow at the 100-year return period
QMED	Median Annual Flood (with return period 2 years)
R&D	Research and Development
SAAR	Standard Average Annual Rainfall (mm)
Tp	Time to Peak
URBEXT	FEH index of fractional urban extent
US	Upstream

1 Introduction

Arup and JBA Consulting were commissioned to develop the Lower Lee Flood Relief Scheme, including works in Blackpool and Ballyvolane. This commission builds upon the findings of the Lee CFRAM, and is in response to the frequent and severe fluvial, tidal and surface water flooding experienced in Cork City, Blackpool and Ballyvolane.

Carrigadrohid and Inniscarra reservoirs, upstream of Cork, alter the natural flow regime of the River Lee. The two dams are operated for hydropower, but also offer some flood protection to Cork. Such a specific intervention means that traditional flood estimation methods are not applicable to the Lee. For this study, a robust method of flood estimation is needed to derive design flows and to appraise the options that use the reservoirs for storage to help attenuate floods. Any method used to calculate design flows must incorporate the effect of reservoirs and therefore the flood hydrology of the river Lee catchment is unusual as it cannot be adequately represented by conventional methods such as single site, FSR and FSU. Instead a routing model must be used to simulate the reservoir. Continuous simulation has been chosen as the preferred method of calculating design flows.

This report is structured as follows:

Section 1: Introduction

Section 2: Details a review of the flood history

Section 3: A review of the work completed during the Lower Lee CFRAM study.

Section 4: An introduction to the catchment characteristics

Section 5: Provides a discussion of key considerations for flood estimation in the Lower Lee catchment, an introduction to continuous simulation and the general approach is outlined.

Section 6: Discusses the development of a stochastic rainfall model and subsequent post processing of outputs are outlined, followed by an assessment of the results compared to FSU rainfall depths,

Section 7: Catchment modelling is described in this section and covers rainfall runoff model calibration and implementing control rules in an ISIS river model of the reservoirs. The section also provides evidence that the model works well and is reliable.

Section 8: The stochastic rainfall series and the catchment model are combined in Section 8 to give design flows for scenarios with no reservoirs (these are compared to the gauged flow estimates derived in Appendix B), the Baseline scenario and the Design scenario. This section also covers climate change and the context of the November 2009 event.

Section 9: In this section, design flows at ungauged catchments are developed from methodologies discussed in Appendix B and will provide inflow values to the hydraulic model for the ungauged tributaries downstream of Waterworks Weir in Cork City.

Section 10: Conclusion

Appendix A: Includes a review of the quality and availability of the hydrometric data for the area and is supported by individual gauge hydrometric data analysis.

Appendix B: Discusses deriving Qmed and flood frequency curves through conventional methods to give estimated flows at the gauged tributaries. These flows will be compared against results obtained from continuous simulation in Section 8, to ensure the continuous simulation model is representative of what is expected in reality and the catchment model is accurate. Flood Peak analysis for each of the individual gauging stations analysed accompanies this Appendix. Also detailed is the 1% AEP calculation of Q100 flow at the Waterworks Weir prior to the construction of the reservoirs by simulating a scenario that removes the influence of the reservoirs using ungauged techniques (FSU). The calculated flow was compared to un-reservoired design flows obtained in Section 8 where a 100-year flood was generated using continuous simulation to ensure the method is representative of the statistical understanding of floods in the region.

Appendix C: PDM rainfall runoff models are calibrated directly for gauged catchments and therefore require observed rainfall and flow. Although flow records at gauged locations were very patchy, there was sufficient data to attempt a calibration at several locations. The rating curves used to calculate flow where available and their parameters are given in Appendix C.

Appendix D: PDM parameters (for all catchments) and calibration performance (for gauged catchments) are documented in model evaluation sheets in Appendix D.

Appendix E: Simulated reservoir levels verses observed reservoir level at Inniscarra and Carrigadrohid for high flow events is outlined in Appendix E

Appendix F: The gates are controlled by a complex set of logical rules which aim to replicate the 'real' flood operation of the reservoirs and these rules are outlined in Appendix F.

Appendix G: Outlines the Proposed Operational Procedures.

Appendix H: Describes the observations from the December 2015 event.

2 Flood History

There is a long history of flooding in Cork City. Severe floods affected the city in January 1789, November 1853, November 1916, August 1986 and November 2009. Since the construction of the dams in the 1950s, floods in Cork have generally been less severe, although there has been fairly frequent flooding of land, roads and small number of properties. The event of November 2009 was an exception, with major damage caused to commercial and residential properties in Cork City. Though it was a fluvial event that caused significant damage in November 2009, the city also suffered from tidal flooding city in 1945, 1962, 1994,1996, 2004 and 2014. Details of flood history are included in Figure 2-1.

2.1 Fluvial Events

2.1.1 January 1789 Event

The 1789 event is the earliest account of significant flooding. It occurred after a period of continuous heavy rainfall and flood waters are believed to have thundered down like a mountain torrent and broke every boundary and overflowed the entire city between the gates. It resulted in the inundation of large parts of the city from Mansion house to Cork Harbour to depths of between 5 and 7 feet.

2.1.2 November 1853 Event

This flood is believed to be close in magnitude to the 2009 Event. Accounts of the flood state that one building collapsed and 40 feet of the quay wall was washed away due to the force of the flood waters. St. Patrick's Bridge also collapsed and was washed away after the foundation was undermined and the North Gate was closed due to concerns of its stability. Reports suggest that 12 people lost their lives during the event and that the lower levels of most buildings were under water.

2.1.3 November 1916

This event has been compared to the 1853 event, with local media stating it was the worst in event in 70 years. Flood waters resulted in a headwater of 7 feet above the crest of Waterworks Weir. Peak flow was estimated here as to be between 523 and 530 cumecs and the bridge at UCC collapsed due to the undermining of its foundation.

2.1.4 August 1986

The largest peak inflow to the dam on record occurred on the 6th of August 1986. This flood was caused by a severe rainstorm. It resulted in large degree of damages on the headwater tributaries with the towns of Ballyvourney and Macroom badly affected.

2.1.5 November 2009

The November 2009 flood is well documented and it caused unprecedented levels of flooding and property damage in living memory. Though no loss of life occurred during the event, 18,000 households were without drinking water and over 100 people were evacuated. Many of the areas affected had not been flooded previously in living memory. It is believed that the November 2009 was similar in magnitude to the 1853 and therefore is likely to be the worst flood in 150 years in the city.

Lower Lee Flood History

Indication of the rank of the most severe fluvial events provided, based on available flood history.

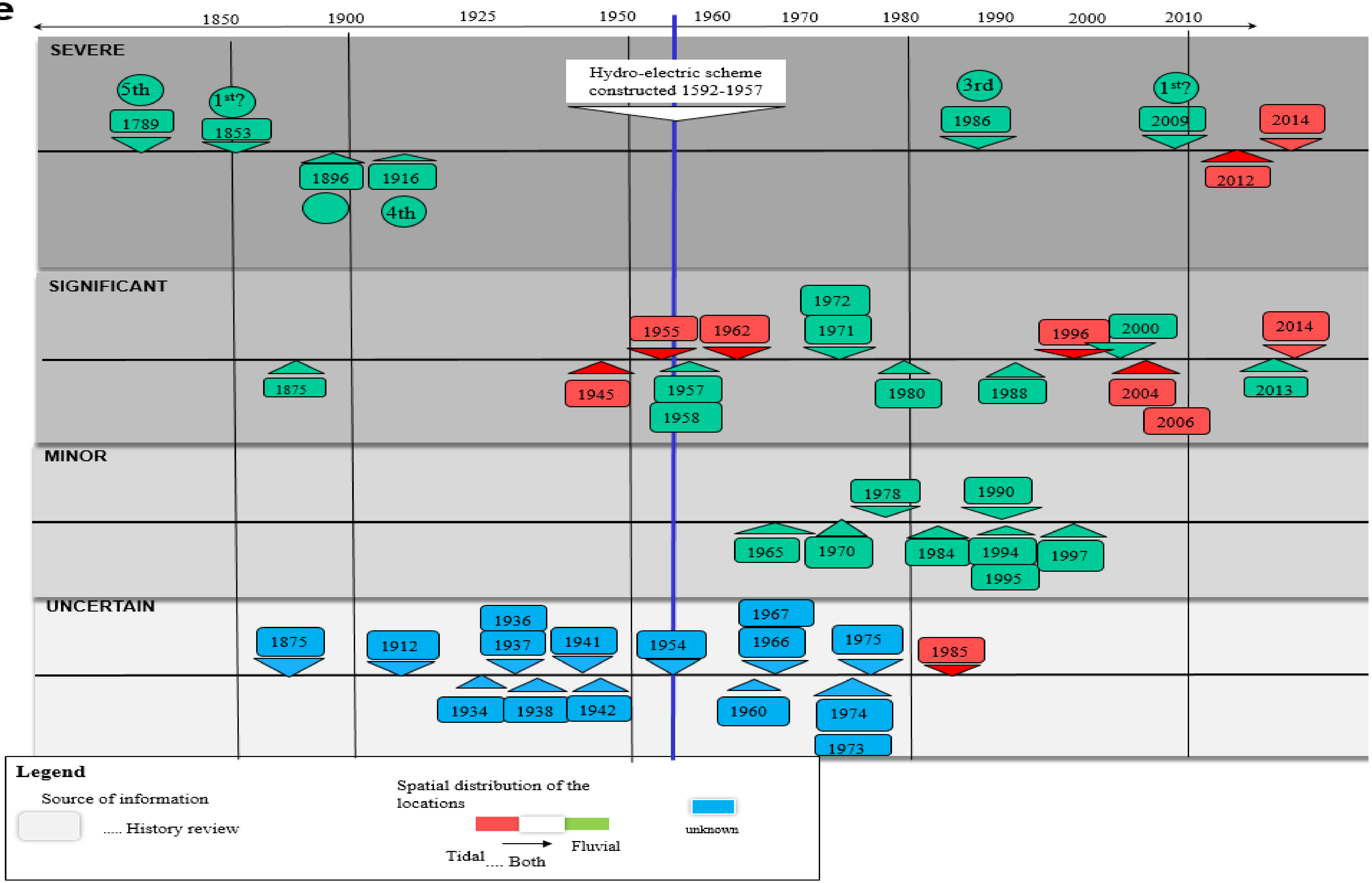


Figure 2-1: Lower Lee Flood History

3 Review of Lee CFRAMS

Before undertaking the hydrology assessment for this study, a review of the hydrological analysis completed during the Lee CFRAMS was undertaken. The following is a summary of how the study derived design flows for their hydraulic models:

Qmed Estimates for the Lee CFRAMS were completed in 2009 using Flood Estimation Handbook (FEH) methodology which predated the Flood Studies Update (FSU). All Qmed estimates were either derived directly from hydrometric station records (gauged catchments) based on single site analysis, or for ungauged catchments inferred from nearby hydrometric station records.

Qmed estimates were calculated using single site analysis for gauged catchments.

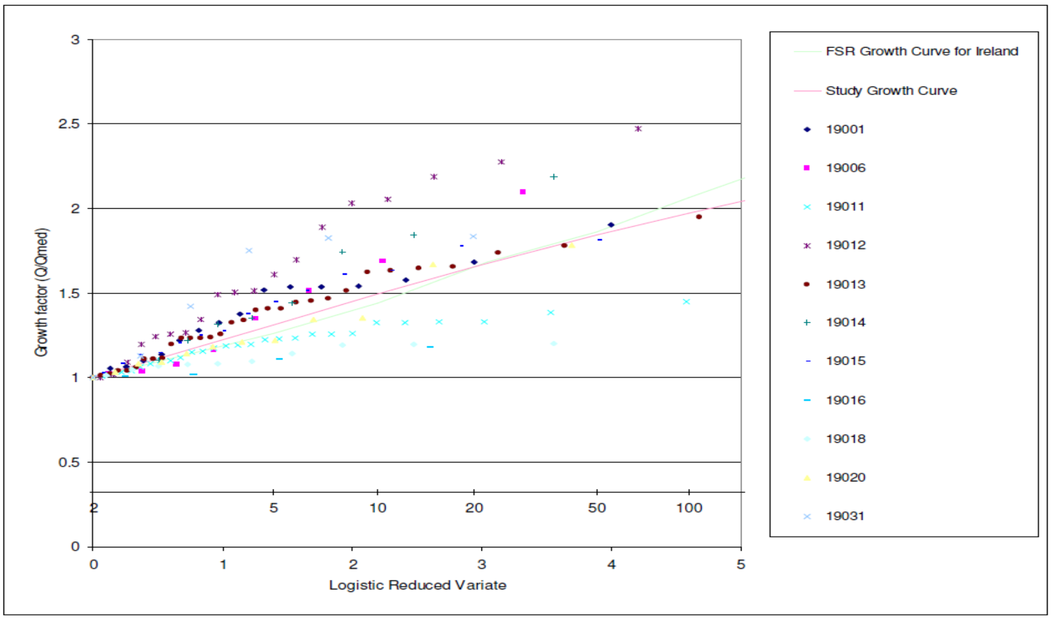
Estimates of the index flood for ungauged catchments were derived using the FEH donor catchment approach in conjunction with the FSR unit hydrograph method. The FEH donor catchment method is based on scaling runoff parameters at gauged catchments to match statistically derived flow and then inferring the proportion of scaling used to ungauged catchments. By calibrating the scale parameters at gauged catchments, the method ensures that all flow estimates are either directly obtained from actual flood records or inferred from flood records.

For the calculation of the frequency growth curve, the study undertook pooled analysis using only gauges within the Lee Catchment. The total record used, excluding gauges on the Lee downstream of the reservoirs was only 157 years, with an average data record of 20 years. Since the completion of the study, FSU methodologies have become available. FSU WP 2.2 recommends creating pooling groups that contain 5T years of data in total, where T is the return period of interest. The design standard is the 1% AEP, and thus each pooling group contains just over 500 years of data.

The study proposed that one indicative study growth curve should be appropriate for the study area. The study averaged L-Moment ratios form the basis of the inputs to the GEV study growth curve. Close proximity of the derived study growth curve with the FSR Ireland growth curve was found, suggesting that the FSR Ireland growth curve is appropriate for use for events in excess of that supported by the statistical record.

Based on close correlation between the study growth curve and the FSR Ireland growth curve for return periods less than 50 years as seen in Figure 3-1, the study pooled growth curve was used for estimates less than 50 years and the FSR Ireland growth curve for all estimates above. The FSR Ireland growth curve was found to be contained within the study pooled 95%le confidence limits thus confirming the appropriateness of the FSR Ireland growth curve to the study.

Figure 3-1: Lee CFRAM Growth Curve



The Lee CFRAM's main aim was to generate catchment scale flood maps and has many limitations. In contrast, the main aim of this study is to design a flood relief scheme for Cork City and to generate design flows for the Lee in Cork City. While the Lee CFRAM hydrology will be referenced, it will be updated based on the newer hydrological approaches that have been developed in the interim period since the completion of the Lee CFRAM. This study also benefits from the recorded data of the November 2009 event

4 Catchment Overview

4.1 Catchment Characteristics

The Lower Lee catchment has a catchment area of 1,151 km^2 by the time it reaches Waterworks Weir in Cork City. There are two large reservoirs situated on the River Lee at Inniscarra and Carrigadrohid and the Lee's flow regime downstream of these reservoirs is greatly impacted by the operating rules of the reservoirs. By the time the Lee discharges at the second of the two reservoirs, Inniscarra, its catchment area is 797km^2, 69% of the total catchment area at Waterworks Weir. Critical storm duration in the catchment is in the region of 48 hours and the critical storm will be a widespread catchment scale event, similar to the event in 2009. It is not sensitive to short duration fluctuations in rainfall intensity.

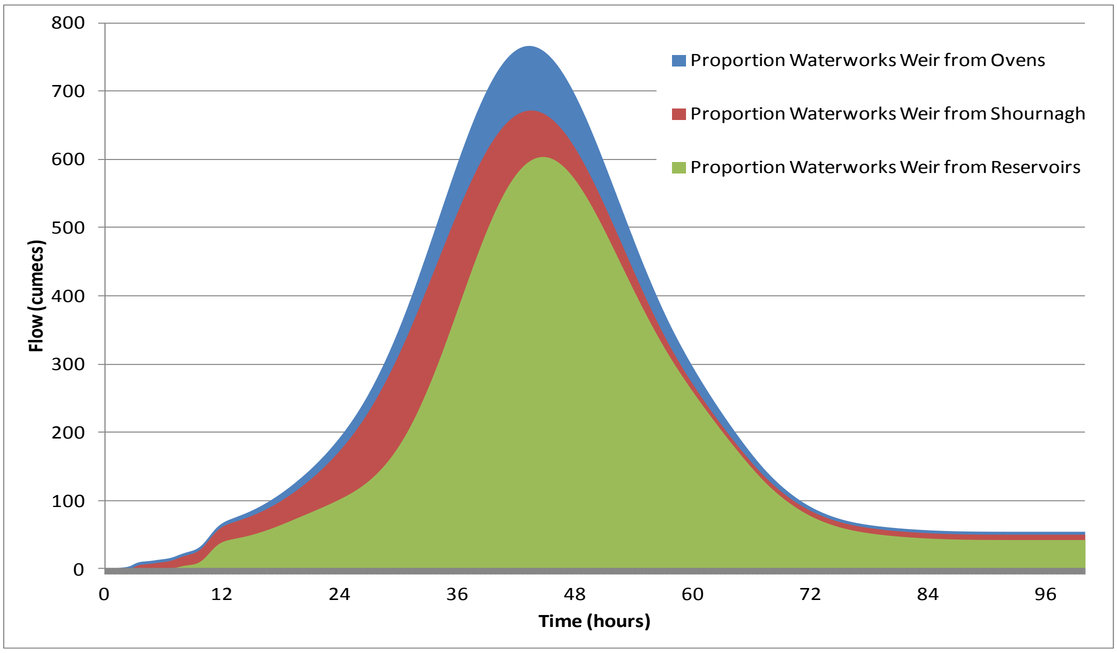
The lower Lee catchment is very wet. The SAAR values vary from 1700mm in the western side of the catchment to 1100m at the eastern extent of the catchment. Upstream of the reservoirs there is a high rate of runoff with little variability. Though soil moisture deficit with the presence of some karst areas on the eastern tributaries of the Bride and the Blarney River is more significant, overall the catchment lends favourably to a rainfall runoff simulated model.

4.2 Impact of the Reservoirs

Carrigadrohid and Inniscarra reservoirs has 69% of the Lower Lee catchment flowing into them. It is evident that the flow in Cork City during an event can be greatly affected by the starting level of the reservoir at the beginning of an event. Therefore, flow in Cork depends on reservoir levels and operating rules, and any method of determining design flows must be capable of representing these rules.

Downstream of Inniscarra two significant tributaries join the Lee upstream of Waterworks Weir; the Shournagh from the north at Leemount, and the Southern Bride joins the Lee at Ovens. Figure 4-1 shows the proportion of the catchment at Waterworks Weir that is controlled by the reservoir and the percentage contributed by these two main tributaries. This is a graph from the rainfall runoff model. The time to peak of the tributaries is slightly faster than the main Lee, especially during intense rainfall events.

Figure 4-1: Contributing flows at Waterworks Weir



5 Introduction to continuous simulation

Carrigadrohid and Inniscarra reservoirs, alter the natural flow regime of the River Lee. The two dams are operated for hydropower, but also offer some flood protection to Cork. On average, they attenuate high flows and redistribute that water to low flow periods (Section 8.3). Such a specific intervention means that traditional flood estimation methods are not applicable to the Lee. Any design flow for the river has to take account of the reservoirs' effects on flood flows. The reservoirs make downstream flows dependent on a multitude of factors, including:

- The starting state of the reservoirs;
- How they are operated;
- Their peak inflow; and
- The total volume of inflow.

Combining these variables in a robust probabilistic framework is essential to design a reliable flood relief scheme for Cork.

Figure 5-1: Inniscarra dam

Continuous simulation modelling (CSM) is such a framework for solving multi variate problems. In this context, the method involves simulating a very long rainfall series and applying it to a complete model of the Lee catchment - including rainfall runoff, flow routing and reservoir operation processes. The result is 1,000 years of simulated annual maxima, which may then be treated as if it were an observed series. Simulated annual maxima of the parameter of interest (flow or level) are ranked for the location of interest and a return period assigned using the Gringorton formula. If the stochastic rainfall series is representative of what might be expected in reality, and the catchment model is accurate, the results should be robust.

5.1 Why continuous simulation is necessary

Simulating flow and reservoir operation continuously allows us to quantify the impact of the reservoirs on peak flows downstream. This is essential for two reasons:

- To establish the flood frequency curve for the River Lee in Cork, with reservoirs operated as currently, allowing a baseline flood risk and damage assessment to be calculated; and
- To test new operational procedures and determine new design flows that will result from them.

The continuous series should contain events of different shapes and durations having the correct probability associated with each. To understand why continuous simulation is necessary, it is easiest to discount the other potential methods.

Statistical techniques for deriving design flows from observed data must be discounted first. Pooled analysis of annual maximum (AMAX) flows is not reliable because the influence of the reservoirs is unique to the Lee. Added to this, the reservoirs currently have the biggest impact in lower order events like QMED, which is the index flood for most statistical methods of design flow estimation. Single site analysis of Lee annual maximum flows is not possible because the record in Cork is very short and the operation of the reservoirs has changed over time. Data from the extensive hydrometric network in the Lee catchment was reviewed (Appendix A) and gauged flow estimates were obtained in order to validate the continuous simulation at the inflow boundaries as well as a "no reservoir" scenario. This work can be found in Appendix B, but is limited by the data gaps and poor performance of the gauging station ratings.

Modelling the behaviour of the reservoirs, and their outflow structures, is therefore a pre-requisite of any design flow estimation technique. Only this approach can simulate their unique influence and test the impact of new operational procedures. If this is accepted, the question of design flow estimation shifts to the model's boundary conditions: i.e. what flows should be input to the model to determine a design flow for Cork. The remainder of this chapter is concerned with obtaining these boundary conditions.

There are four possible choices, considered in turn below:

1. Applying an observed event

Applying inflows from an observed event is the simplest solution to providing the model with upstream boundary conditions. November 2009 was a serious flood in Cork and could be used for this purpose. However, the event is just one potential hydrograph of many and its probability, in terms of peak flow and volume, is not well understood. Following a review of this event as the basis for a scheme design it was concluded that it would not be robust and could lead to over or under design.

2. Scaling an observed event to statistical flow estimates at upstream boundaries

Scaling an observed event, rather than just applying observed data, could ensure that the model inflows have a peak flow that matches the n-year event at one or more of the boundaries. The main drawbacks of this approach are that the flow exiting the reservoirs is as dependent on the volume of the overall simulation as the peak flow. Choosing a single event shape is a big assumption that could, like the observed event approach, lead to over or under design. An additional problem is that applying the same probability flow at each boundary is likely to give a conservative result when they are combined in the model. A 100-year flow in Cork is unlikely be accompanied by 100-year flows on all of the tributaries because each has its own critical storm duration. This approach was attempted on the Lee CFRAM and upon review had serious limitations.

3. Applying the FSSR16 rainfall runoff model

Rainfall runoff approaches like FSSR16 package together depth, duration and frequency of rainfall with a rainfall runoff model to give a flow of a given probability. This should give volumes and peak inflows for a given probability. However, the assumption of a single design hyetograph (single or multi peaked) is not reasonable and the method does not easily account for this variability. A multi peaked event caused the severe flooding in Cork in November 2009.

4. Continuous simulation

Continuous simulation provides multi peaked rainfall events within a rational framework for assigning a probability to those. When applied to a rainfall runoff model continuously, this approach should give a peak flow series that reflects the FSU statistical approach while preserving realistic hydrograph shapes and volumes. As an added safeguard, the continuous simulation peak results can be checked against the required peak flood frequency curve at the location of interest (Cork), and if required, adjusted, without losing the benefit of the variability in hydrograph shape, timing and volume. By process of elimination, continuous simulation is the only remaining viable choice.

5.2 General approach

Continuous simulation is a multi-stepped process that, for the Lee, involved:

- Deriving a stochastic rainfall series for the catchment using a statistical model. We use the Bartlett Lewis Rectangular Pulse Model, described further in Section 6 below.
- Post processing the stochastic rainfall to get better agreement with design rainfall depths (in this case, the FSU rainfall statistics) in the critical range of durations and return periods (also covered in Section 6).
- Developing, calibrating and proving a catchment model that can predict flow at Cork. This model must take account of all flow production processes, from the generation of runoff from rainfall to the operation of the reservoirs and the phasing of tributaries. Rainfall runoff

modelling and river/reservoir modelling are the key components of this catchment model - and both are discussed in Section 7 below.

- Applying the stochastic rainfall to the catchment model and extracting annual maxima at the locations of interest. Versions of the model with and without the reservoirs are simulated in this way, as described in Section 8. A design version of the model and its parameters are also implemented and run.
- Comparing the peak flow frequency curve from the continuous simulation without reservoirs to design flows at Waterworks weir and elsewhere. These comparison design flows are calculated using traditional 'index flood and multiplier' methods.
- Reinstating the reservoirs and testing Baseline and Design scenarios with the same inflow series (Section 8.3). Results are also extracted for these scenarios as annual maxima to enable comparisons to be made in the resulting flow frequency curves.

6 Stochastic rainfall modelling

6.1 Introduction to stochastic rainfall models

A stochastic rainfall model generates artificial rainfall data with statistical characteristics that are intended to be similar to real rainfall.

In this study, we have used a “point” rainfall model, which produces a single sequence of rainfall at a representative location or area in a catchment - in this case the Macroom sub-catchment. A UK EA/Defra research project FD2105[1] found that point rainfall models were as good as, if not preferable to, spatial-temporal models on slower responding catchments. The catchment to Inniscarra Reservoir is somewhat damped by the two impoundments and a point rainfall runoff model is felt to be appropriate. The quicker responding Rivers Bride and Shournagh combine with the Lee downstream of Inniscarra. Their location makes it unlikely that they would receive exactly the same rainfall hyetograph as the Upper Lee during a design flood. However, their contribution is smaller than that of the Lee at Inniscarra. A spatial-temporal model could capture variability in rainfall across the catchment, but is significantly more complicated (and expensive) to implement. The point approach is justified because:

- It is the outflow from Inniscarra that is critical in any flooding scenario for Cork. In such a flood, likely to result from widespread, prolonged rainfall, the hyetograph probably will be similar on the Bride and Shournagh to that falling on the Lee. Locally intense events with high spatial variance are less likely to cause a problem for Cork, precisely because they do not affect the entire catchment. In support of this assumption, Figure 6-1 shows the hourly rainfall hyetograph for five gauges that were operational in November 2009 - the largest flood experienced on the Lee in the period where records are available (see Figure 6-2 for a map showing the rain gauges). Although the rainfall totals vary across the catchment, the general timing and shape of the rainfall plots is similar.
- It is conservative to assume the same rainfall hyetograph for the tributaries as the main river;
- The additional time and cost required to develop a spatio-temporal model is difficult to justify.
- The approach is tried and tested by JBA on other catchment scale modelling studies such as the Don in Northern England[2].

There are numerous examples of stochastic rainfall models in the hydrological literature, including several that have been designed or calibrated specifically to generate realistic extreme rainfalls: for example Cowpertwait (1998)[3], Cameron *et al*. (2000)[4], Burton *et al.* (2008)[5] and the references in the following section.

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[1] EA/Defra (2005). Improved methods for national spatial-temporal rainfall and evaporation modelling for BSM. R&D Technical Report F2105/TR. Link here.

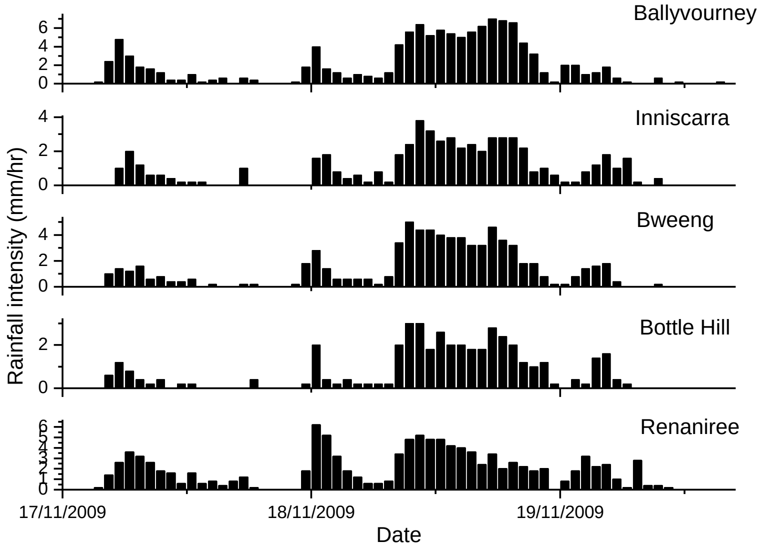
[2] Faulkner, D. and Wass, P. (2005) Flood estimation by continuous simulation in the Don catchment, South Yorkshire, UK. WEJ (Journal of CIWEM), **19** (2), 78-84.

[3] Cowpertwait, P.S.P. (1998). A Poisson-cluster model of rainfall: high-order moments and extreme values. Proc. R. Soc. Lond. A (1998) **454**, 885-898.

[4] Cameron, D., Beven, K. and Tawn, J. (2000) An evaluation of three stochastic rainfall models. *J. Hydrol.* **228**, 130-149.

[5] Burton, A., Kilsby, C.G., Fowler, H.J., Cowpertwait, P.S.P., O'Connell, P. E. (2008). RainSim: A spatial-temporal stochastic rainfall modelling system. Env. Modelling & Software **23** (12), 1356-1369.

Figure 6-1: November 2009 rainfall hyetographs from gauges around the Lee catchment



A growing number of studies have used such models in conjunction with a continuous simulation rainfall-runoff model to produce flood estimates. Examples in the published literature include Faulkner and Wass, 2005[6]; Blanc *et al.*, 2012[7], Grimaldi *et al.*, 2012[8] and Smith *et al.* (2014)[9]. Several UK flood mapping studies have applied continuous simulation, including some on highly permeable catchments, low-lying catchments with tide locking, and rivers with controlled flood storage areas. However, the vast majority of UK and Irish flood studies continue to be based either on statistical analysis of flow or on rainfall-runoff models that simulate single design rainfall events.

Analysis of the hydrometric network acted as a means of validating the results of the continuous simulation model to ensure it was producing realistic and accurate results (Appendix A and B).

To be useful in flood estimation, a rainfall model must be capable of reproducing extreme rainfall depths aggregated over a range of durations. It must also represent typical storm durations and storm profiles. Because the simulated rainfall series is to be run through a set of PDM rainfall-runoff models, it is also important that the rainfall model can produce realistic annual and seasonal rainfall totals to ensure that the catchment water balance is correctly modelled.

We have used a version of the Bartlett-Lewis Rectangular Pulse Model, an example of a "pulse-based" rainfall model. It generates storms composed of a cluster of rain cells. Each cell has a random duration and a random constant intensity. Several cells can be active at once. The total storm intensity at a certain time is found by adding the intensities of all active cells.

The model generates rainfall on a continuous basis which can then be aggregated at any time step, with no need for a separate step to downscale, for example from daily to hourly.

6.2 Description of modified Bartlett-Lewis model

Various versions of the Bartlett-Lewis model have been developed, partly to improve its representation of extreme rainfalls. Four successive versions are described by Onof and Wheater (1993[10], 1994[11]), Cameron et al (2001)[12] and Faulkner and Wass (2005). These versions have 6,

[6] Faulkner, D. and Wass, P. (2005) Flood estimation by continuous simulation in the Don catchment, South Yorkshire, UK. WEJ (Journal of CIWEM), **19** (2), 78-84.

[7] Blanc, J., Hall, J.W., Roche, N., Dawson, R.J., Cesses, Y., Burton, A. and Kilsby, C.G. (2012). Enhanced efficiency of pluvial flood risk estimation in urban areas using spatial–temporal rainfall simulations. J. Flood Risk Man. **5**, 143-152.

[8] Grimaldi, S., Petroselli, A. and Serinaldi, F. (2012). A continuous simulation model for design-hydrograph estimation in small and ungauged watersheds. Hyd. Sci. J., **57** (6), 1035-1051

[9] Smith, A., Freer, J., Bates, P. and Sampson, C. (2014). Comparing ensemble projections of flooding against flood estimation by continuous simulation. J. Hydrol. **511**, 205–219

[10] Onof, C.J. and Wheater, H.S. (1993) Modelling of British rainfall using a random parameter Bartlett-Lewis Rectangular Pulse Model. *J. Hydrol.* **149**, 67-95.

[11] Onof, C.J. and Wheater, H.S. (1994) Improvements to the modelling of British rainfall using a modified random parameter Bartlett-Lewis Rectangular Pulse Model. *J. Hydrol.* **157**, 177-195.

[12] Cameron, D., Beven, K. and Tawn, J. (2001) Modelling extreme rainfalls using a modified random pulse Bartlett-Lewis

7, 8 and 9 parameters respectively and correspond to different approaches to raincell intensity simulation. Onof and Wheater (1993) initially used an exponential distribution, which was later replaced with a gamma distribution for improved intensity simulation (Onof and Wheater, 1994). In order to improve the simulation of short duration extreme rainfalls, Cameron et al (2001) added a Generalised Pareto Distribution (GPD) to the exponential model of Onof and Wheater (1993). Faulkner and Wass (2005) used a similar approach but with the gamma distribution (Onof and Wheater, 1994) representing low intensity raincells and the GPD representing high intensity raincells.

Following a period of sensitivity testing, it was found that the gamma distribution version (Onof and Wheater, 1994) of the model was sufficient for modelling the flood producing storms in the catchment. This model selects a cell intensity from a gamma distribution with shape parameter ρ and scale parameter $1/\delta$.

The modelling of extreme rainfalls can be improved by introducing a minimum threshold value for η, the parameter of the exponential distribution of cell duration[13]. If η is too small for a particular storm, the model produces very long cells that result in unrealistically extreme rainfall depths. After some trial and error, the threshold was set to 0.5. Values of η below the threshold were reset to 0.5.

The modified version of the rainfall model has seven parameters and is programmed in Fortran.

6.3 Strategy for calibrating the model

The objective of calibration is to reproduce features of rainfall that are important for producing high flows in these catchments. Our aim was to reproduce extreme rainfall depths for a wide range of durations, while ensuring realistic annual rainfalls and other characteristics. Without reservoirs, the critical duration of the Lower Lee at Cork is around 48 hours. Using single peaked FSSR design events, the critical storm duration with the current reservoir operational rules is also 48 hours. However, we know that reservoir levels remain elevated for some days after a storm, so rainfall durations longer than 48 hours should also be considered. To cover this, we aimed to reproduce rainfall depths for durations up to 150 hours.

Where data allow, separate parameter sets can be derived for summer and winter seasons (defined as April to September and October to March). In the Lee catchment, rainfall records are intermittent and only extend from around October 2002 (refer to Section 7.3). After testing seasonal calibrations unsuccessfully, we decided to carry out a lumped calibration of the rainfall model on the entire record, rather than partitioning according to season.

Average rainfall depths and statistics, such as lengths of dry spells, can be obtained from relatively short periods of observed rainfall data. However, extreme rainfall depths for return periods up to hundreds of years cannot be reliably estimated from single rainfall records. Instead, they are obtained using the statistics developed using the techniques described in the FSU14 (FSU), which are derived by fitting growth curves to local and regional rainfall data. The strategy for calibrating the model involved an initial calibration to observed rainfall data (Section 6-4), then adjustment of the parameters to reproduce some of the FSU rainfall statistics for the study catchments (Section 6.5).

We chose one of the largest gauged catchment inflowing to Carrigadrohid, Macroom, on which to base our stochastic rainfall series. Rainfall inputs to the other catchments in the model are scaled off this series according to the ratio of their annual average rain to Marcoom's (see Table 8-1 in Section 8.1).

6.4 Calibration to local rainfall data

Initial calibration of the stochastic rainfall model requires continuous hourly rainfall data for as long a period as possible. The intermittent nature of the individual hourly rainfall records meant that it was necessary to merge several series together to obtain a continuous series. This was done by calculating a catchment average for Macroom using the gauges listed in Table 6-1 (also shown in Figure 6-2). Rainfall was averaged according to the Theissen polygon weights shown in the table,

stochastic rainfall model (with uncertainty). *Adv. Water Resour*. **24**, 203-211.

[13] Based on research at CEH Wallingford. Personal communication from Christel Prudhomme.

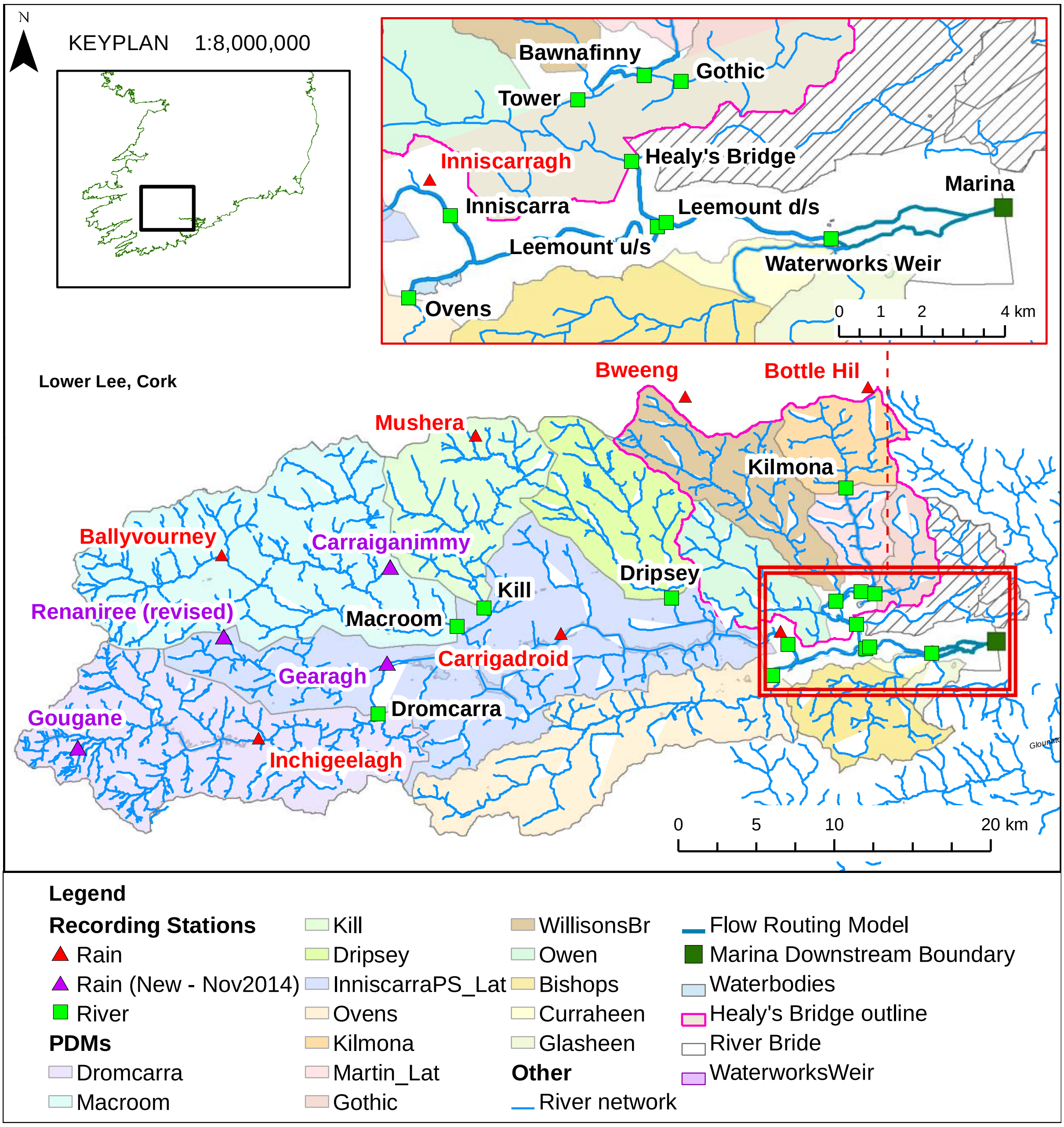
[14] Fitzgerald, D.L. (2007) Met Éireann Irish Meteorological Service Technical Note 61: Estimation of Point Rainfall Frequencies, WorkPackage 1.2, Flood Studies Update.

and during periods of missing data, weight was re-allocated among the remaining gauges where data was available. For the period looked at (23 Oct 2002 to 9 August 2013) the annual average rainfall was 1672mm compared to the FSU SAAR value for the catchment of 1744mm (within 95.8%).

Table 6-1: Rain gauge weights used to calculate Macroom catchment average for 23 Oct 2002 to 9 August 2013

Rain gauge	Weight
Ballvourney	0.697
Renanirree	0.191
Mushera	0.028
Inchigeelagh	0.005

Figure 6-2: Lee catchment map



Several of the model parameters are adjusted after the initial calibration to local rain data, making the absolute accuracy of rainfall depth less critical. More important is that the model generates stochastic rainfall with the right the temporal characteristics: hyetograph shape, timing of rainfall storms etc.

Although individual rain gauge series' have missing data, the averaging process (where missing data is replaced by the average of the remaining gauges) gives us a continuous series. Averaging also means it is broadly representative of the catchment as a whole (if not any one individual gauge) and the annual average for the 10 years available is within 5% of that specified by the FSU rainfall statistics. Furthermore, when the rainfall series is applied to the catchment model, flow predictions are a reasonable match (refer to Section 0). We therefore deemed the Macroom rainfall series adequate for initial calibration of the model.

The stochastic model was initially fitted by choosing a set of characteristic variables describing the rainfall data and solving equations that define these variables in terms of the model parameters. The equations, which have been determined analytically from the structure of the model, are given in the papers listed above.

The variables, chosen to emphasise the properties of rainfall totals and dry spells over a wide range of durations, are given in Table 6-2, along with their values calculated for the Macroom catchment.

Table 6-2: Variables characterising rainfall at Macroom (lumped - no season)

Description	Value	Weight used in fitting model
Mean of 1-hour rainfall depths (mm)	0.2436	5
Variance of 1-hour rainfall depths (mm^2)	0.4998	5
Lag-1 covariance of 1-hour rainfall depths (mm^2)	0.3439	5
Proportion of 1-hour spells that are dry	0.6991	4
Variance of 24-hour rainfall depths (mm^2)	75.7461	2
Proportion of 24-hour spells that are dry	0.1871	2
Variance of 72-hour rainfall depths (mm^2)	348.0937	1
Proportion of 72-hour spells that are dry	0.0723	1

To find the model parameters, the equations have to be solved simultaneously. They are highly non-linear, and a unique solution may not exist. They were solved using an approach suggested by Wheater et al. (2000)[15] of minimising the sum of weighted squared differences between observed variables and model variables (given by the equations mentioned above).

Each term in the summation was normalised, converting the differences into proportional differences, to avoid bias due to the different orders of magnitude of the various statistics. The minimisation was carried out by the Simplex optimisation method[16]. This gave one set of initial parameters representing the total rainfall series for the modified Bartlett-Lewis model. Following this initial fit, the gamma distribution parameters controlling raincell intensity were further adjusted by comparison with the FSU statistics and frequency curves.

6.5 Calibration to FSU rainfall statistics

A two part approach was adopted for calibration to FSU rainfall statistics:

1. Fitting to rainfall frequency curves with a focus on durations of 6, 24, 72 and 144 h; then
2. Post processing the model outputs to improve the fit further.

This approach is described as follows.

The FSU provides extreme rainfall statistics relating rainfall depths, durations and frequencies (from a DDF model). There are few examples in the literature of stochastic rainfall models that have been fitted to DDF models rather than solely to observed rainfall records. Onof et al. (1996)[17]

[15] Wheater, H.S., Isham, V., Cox, D.R., Chandler, R.E., Kakou, A., Northrop, P.J., Oh, L., Onof, C. and Rodrigeuz-Iturbe, I. (2000) Spatial-temporal rainfall fields: modelling and statistical aspects. *Hydrol. and Earth System Sci.* **4**, 581-601.

[16] Nelder, John A.; R. Mead (1965). "A simplex method for function minimization". Computer Journal 7: 308–313.

[17] Onof, C., Faulkner, D. and Wheater, H.S. (1996) Design rainfall modelling in the Thames catchment. *Hydrol. Sci. J.* **41**, 715-733.

fitted a Bartlett-Lewis model to Flood Studies Report rainfall statistics, although this was used to produce discrete events rather than a continuous sequence of rainfall.

In this project, parameters of the Bartlett Lewis model were adjusted so that the modelled extreme rainfalls gave a reasonable match to the above sets of FSU statistics, for durations of between 6 and 144 hours. This was done on a trial and error basis (explained further below) as there are no analytical expressions for the moments of extreme rainfall simulated by the Bartlett-Lewis model. Trial and error adjustment employed a mixture of judgement and knowledge of the model's structure.

Two parameters were varied: the scale (δ) and shape (ρ) parameters of the gamma distribution of initial cell depth. Parameters that control the temporal characteristics of the rainfall, i.e. the rate of storm arrival, the duration of storms and the duration of cells, were left unchanged during this stage.

A total of 1000 model runs were carried out initially, each with slightly different parameter values. Each run produced 400 years of simulated rainfall, from which the annual maxima for a range of durations were extracted and their moments calculated.

The parameters which were judged to give the best results were then subject to some minor manual adjustments in order to provide a better fit to the FSU frequency curves. The final parameter values are given in Table 6-3.

Table 6-3: Parameter values for the Macroom rainfall model

Parameter	Symbol	Value
Rate of storm arrival (hr^{-1})	λ	0.0081
Rate of cell arrival divided by cell duration	κ	0.0794
Mean storm duration divided by cell duration	ϕ	0.0058
Shape parameter of gamma distribution for cell duration	α	3.1653
Inverse of scale parameter of above gamma distribution	υ	3.7780
Shape parameter of the gamma distribution of initial cell depth	ρ	3.35
Inverse of scale parameter of above gamma distribution	δ	1.57

Long (1,000-year) series of simulated rainfall data were generated using the parameter set in Table 6-3 and then compared to FSU rainfall statistics (discussed further below).

6.6 Results: extreme rainfalls

Raw outputs from the stochastic rainfall (following the iterative calibration described above) are compared with FSU rainfall statistics for the Macroom catchment average in Table 6-4. The stochastic rainfall depths are expressed as a percentage of the FSU value for a wide range of durations and return periods. Raw model rainfall depths are higher than FSU depths at durations of around 24 hours (cells shaded red). At shorter and longer durations, the proportion is less than 1 (cells shaded blue). This discrepancy prompted the post processing of the data which brings the stochastic rainfall closer to the FSU expected depths.

Table 6-4: Performance of the 'raw' Macroom stochastic rainfall model compared to FSU rainfall statistics

	Return period (years)								
	2	5	10	20	30	50	100	150	200
1hrs	73%	65%	63%	58%	56%	53%	50%	48%	47%
2hrs	89%	82%	78%	75%	72%	69%	64%	62%	59%
3hrs	99%	92%	87%	82%	80%	78%	73%	69%	66%
4hrs	104%	97%	93%	87%	86%	83%	80%	78%	76%
6hrs	109%	103%	100%	94%	92%	93%	88%	87%	85%
9hrs	109%	107%	104%	100%	101%	97%	93%	91%	89%
12hrs	106%	108%	107%	105%	104%	100%	98%	93%	91%
18hrs	104%	109%	110%	114%	111%	109%	106%	107%	107%
24hrs	100%	105%	111%	110%	112%	113%	116%	111%	112%
48hrs	97%	101%	104%	106%	104%	109%	108%	104%	102%
72hrs	96%	99%	101%	103%	105%	104%	105%	104%	103%
96hrs	92%	96%	98%	98%	100%	100%	100%	100%	100%
144hrs	88%	91%	92%	92%	91%	93%	94%	91%	91%
192hrs	83%	87%	88%	87%	90%	89%	89%	89%	92%
240hrs	81%	85%	85%	85%	84%	85%	86%	89%	88%
288hrs	78%	82%	82%	83%	83%	82%	83%	84%	88%
384hrs	75%	77%	80%	80%	79%	79%	82%	82%	82%
480hrs	72%	75%	77%	77%	77%	78%	78%	78%	80%
600hrs	70%	73%	74%	75%	75%	74%	75%	75%	77%

The aim of post processing was to:

- Reduce the positive bias at the middle durations;
- Reduce the negative bias at long durations; and
- Improve the match to annual average rainfall (1362mm from the simulated compared to 1774mm from FSU)

The best results were obtained by:

- Scaling all rainfall values by 1/1.12; and
- Extending every rainfall event at its start and end by repeating the first/last value of that event.

An example result of post processing is given in Table 6-5 below for a small event.

Table 6-5: Example of post processed rainfall

	Rainfall (mm)	
Hour	**Raw**	**Post processed**
1		0.8
2	0.9	0.8
3	1	0.89
4	1.5	1.34
5	2.1	1.88
6	1.8	1.61
7		1.61
Total	**7.3**	**8.93**

Table 6-6: Post processed model output compared to FSU rainfall statistics

				Return period (years)					
	2	5	10	20	30	50	100	150	200
1hrs	66%	59%	56%	52%	50%	47%	45%	43%	42%
2hrs	81%	73%	70%	67%	65%	62%	58%	55%	53%
3hrs	89%	83%	78%	74%	72%	70%	66%	62%	61%
4hrs	95%	88%	84%	78%	78%	75%	72%	70%	68%
6hrs	100%	95%	91%	85%	84%	83%	81%	78%	78%
9hrs	103%	98%	94%	95%	92%	90%	83%	81%	81%
12hrs	101%	101%	98%	95%	94%	91%	88%	85%	82%
18hrs	99%	102%	102%	103%	100%	98%	95%	96%	96%
24hrs	97%	99%	102%	102%	102%	103%	103%	101%	100%
48hrs	100%	100%	100%	101%	100%	103%	102%	99%	98%
72hrs	101%	101%	101%	102%	101%	101%	100%	100%	101%
96hrs	100%	99%	101%	100%	100%	98%	97%	98%	97%
144hrs	97%	97%	97%	96%	96%	95%	95%	92%	92%
192hrs	94%	95%	96%	95%	95%	93%	94%	92%	93%
240hrs	92%	94%	95%	92%	92%	90%	94%	91%	93%
288hrs	90%	92%	92%	92%	91%	90%	90%	90%	90%
384hrs	88%	90%	89%	90%	89%	89%	88%	86%	86%
480hrs	86%	88%	88%	88%	88%	87%	84%	85%	85%
600hrs	84%	85%	86%	86%	86%	84%	83%	82%	84%

In the revised results (shown in Table 6-6, above), the positive bias is largely removed and the under prediction at long durations is curtailed. Annual average rainfall is also increased to 1744mm.

At our risk area in Cork, the Lower Lee's 'natural' critical storm duration (without reservoirs) is around 48 hours. By introducing the two reservoirs, and applying rules to draw them down prior to a large event, the catchment becomes sensitive to multiple storms over a period of several days. We are therefore most interested in storm durations around these values - from 24 hours to 144 hours and beyond.

The post processed stochastic model yields design rainfalls that are mostly within 10% of the FSU values (indicated by red outlining). Depths are shallower than FSU at shorter durations (less than 12 hours), especially at higher return periods. Given the aim to achieve good results at durations of 24 hours and longer, the agreement is considered good. The post processed dataset was therefore taken forward for use in continuous simulation.

Note that the 'point' rainfall series derived at this juncture is actually representative of the Macroom catchment (not a single rain gauge). It is subsequently scaled, according to Standardised Annual Average Rainfall, to obtain time series for the other catchments as described in Section 8.

7 Development of a catchment model

A catchment model is needed to convert the stochastic rainfall series to flow. Several processes are encompassed within this model, which is a collection of individual components. It generates runoff from rainfall, routes this in river channels and stores and releases it from reservoirs. The development of this model is discussed in the following sections of the report.

7.1 Requirements of the model

Several requirements were considered when developing the catchment model. It needed to be able to:

- Simulate runoff and catchment wetness continuously in order to reproduce the observed wetting and drying of soil between events;
- Route flows accurately through the river network as far as Waterworks Weir (but it need not predict water levels anywhere other than in the reservoirs);
- Model water being stored and released from Carrigadrohid and Inniscarra reservoirs. In particular, it needed to mimic the rules that govern releases made from the reservoirs before, during and after a flood event.
- Be flexible enough to test other operational scenarios for the reservoirs;
- Be capable of running in continuous simulation over a long (1,000-year) period.

These requirements are very similar to those of the forecasting model also being developed for this project. In fact, this catchment model became a dual purpose tool, used for flood estimation and also to be used for the eventual forecasting system. The fact that a forecasting model was already being developed meant that continuous simulation was feasible from a budget point of view - otherwise continuous simulation may have been prohibitively expensive.

7.2 Choice of model software

We chose the UK's Centre for Ecology's Probability Distributed Moisture model to simulate runoff (refer to Section 7.5) and ISIS to reproduce the effect of the reservoirs and river. These models meet all of the requirements of continuous simulation (outlined above) and of forecasting tools (described in our forecasting report). Importantly, they are also tried and tested in previous continuous simulation projects and are operational in many forecasting applications in the UK and elsewhere in the world.

7.3 Data availability

Observed hydrometric data are needed to calibrate and verify the catchment model. The availability of hydrometric data was discussed in Appendix A and Figure 6-2 shows the locations of all rainfall and river gauges. PDM rainfall runoff models are calibrated directly for gauged catchments and therefore require observed rainfall and flow. Although flow records at gauged locations were very patchy, there was sufficient data to attempt a calibration at several locations (refer to Section 7.5). The rating curves used to calculate flow are those used in the Lee CFRAM where available and their parameters are given in Appendix C.

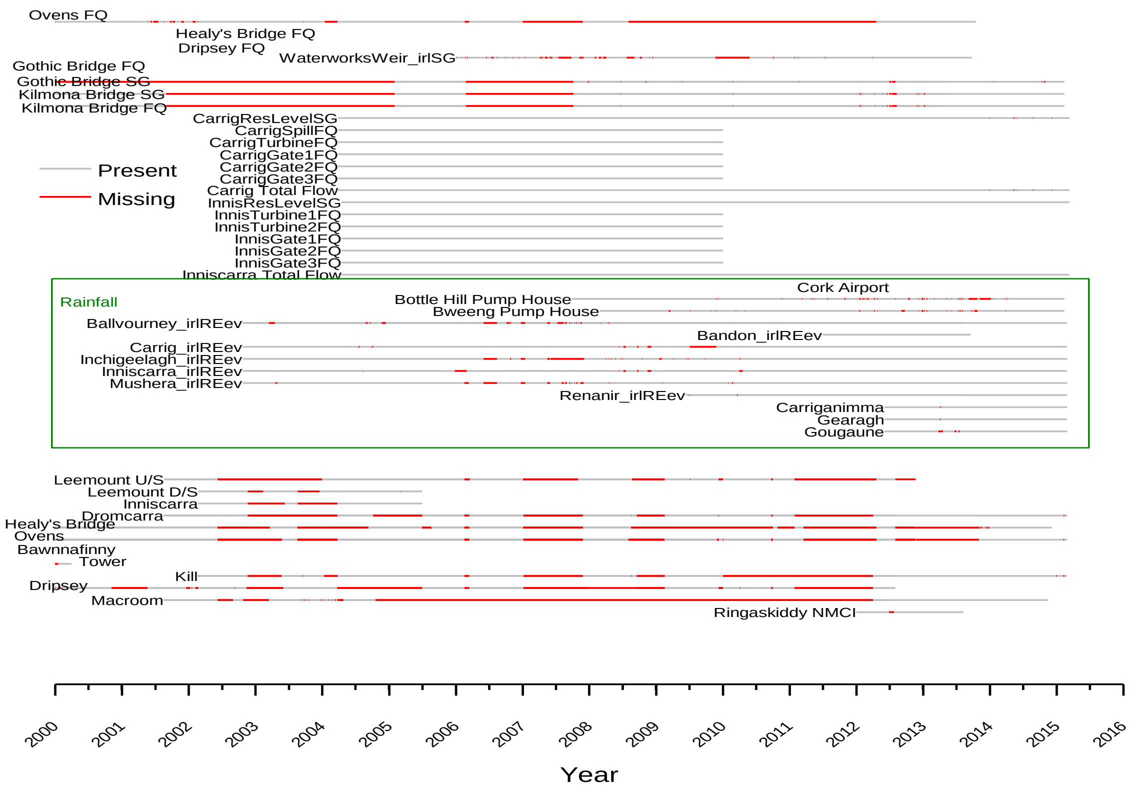
A new rating curve was developed for Macroom when the CFRAM rating showed strong bias. This was evidenced by PDM results (which required an unreasonable rainfall scaling factor). The Macroom flows in combination with Kill and Dromcarra flows formed the ESB derived inflow series to Carrigadrohid.

Reservoir level and outflow records for Carrigadrohid and Inniscarra are more complete than gauged records elsewhere and are available from 2002 to present. These are used for validating the full model over a long simulation of historical data, run from observed rainfall. The outflow data is derived by ESB from a rating calculated from the gate hydraulics.

Existing models are the other main source of data. Hydraulic models developed for the Lee CFRAM were supplied. These contain cross sections through the Lee Valley, allowing an accurate calculation of the volume of the storage available.

ESB's 'Regulations and Guidelines for the control of the River Lee' were another important reference for calibrating the outflow structures of the reservoirs.

Figure 7-1: Availability of hydrometric data series in the Lee catchment (red indicates periods missing)



7.4 Schematisation

Schematising a catchment model involves deciding how the different sub-catchments are represented, what the extent of the river model will be and how reservoirs are represented. This depends partly on:

- Where in the network results are required. As a minimum, this should include: anywhere that has gauged data; both reservoirs (level and outflow); and Cork itself as the main risk area (Waterworks Weir);
- How the river basins combine to physically make up the full catchment (shown in Figure Figure 6-22); and
- The availability of gauged data for calibration.

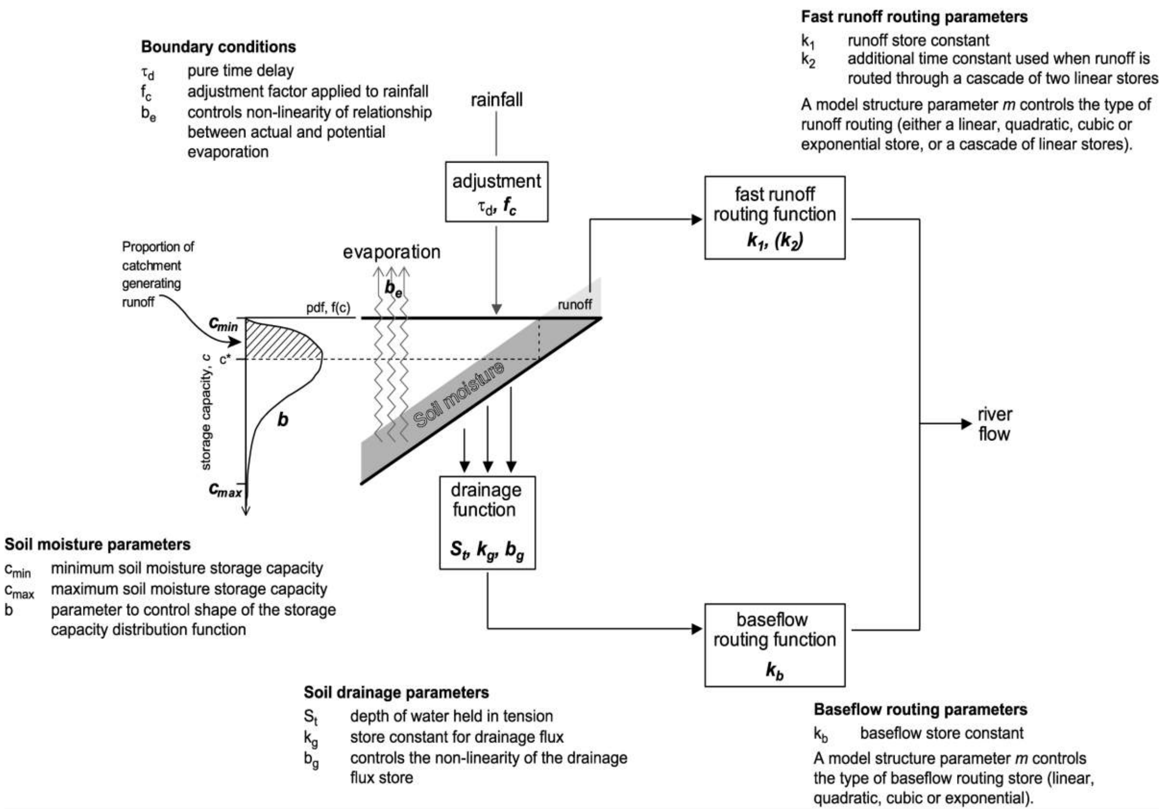
Figure 6-2 shows: the extent of the river model developed; the 12 sub-catchments that are represented by a rainfall runoff model; and the locations where data is available and results can be extracted. The implemented solution has sufficient spatial resolution to deal with differing rainfall inputs and contrasting hydrological responses, while still being simple enough to calibrate. For example, the Macroom catchment is modelled as a single rainfall runoff component but the Shournagh is subdivided into multiple reaches - reflecting the need to model its faster response and channel travel time. The focus of these rainfall runoff models was to simulate the flows into and out of the reservoirs and how they combine with the major tributaries to produce the flood wave into the outskirts of the City. The lateral inflows within the valley downstream of the dam contribute a minor portion of flow and will not significantly alter the flow estimated at Waterworks weir. They were excluded from the forecasting model as they are not measured and would fall within the uncertainty bound of any estimate.

Model schematisation for the forecasting model (and therefore this model also) is discussed in more detail in the forecasting report.

7.5 Rainfall runoff model development

PDM is a conceptual rainfall runoff model that describes a catchment as a series of three main stores (Figure 7-2). A soil store, surface runoff store and baseflow store are parameterised by calibration against observed data. Rainfall is intercepted by the soil store and a proportion, determined by soil wetness, runs off as fast flow. The remainder infiltrates the soil to increase its content. Water drains from the soil to a baseflow store at a rate proportional to its water content. The surface runoff and baseflow stores attenuate and smooth their inputs and are combined to give a total catchment flow. The PDM has 12 main parameters, outlined in Figure 7-2. A full description of the PDM is given by Moore (2007)[18].

Figure 7-2: The PDM nodel structure and main parameters



For gauged catchments, PDM is calibrated against observed data by trial and error adjustment of its parameters. It requires observed rainfall and some measure of potential evaporation as an input. In this case PE was provided by an annual SINE curve with a maxima of 3mm/day on 1 July and a minima of 0mm/day on 1 January. Simulated flows are then compared against observations to make informed adjustments to the parameters. Normal practice is to split a calibration dataset into calibration and verification periods. The large proportion of missing flow data at most stations meant that this was not possible in the Lee catchment. Instead, the PDM was calibrated against whatever data were available by running the PDM with rainfall and PE continuously and making comparisons with observed data when it was available.

Parameters for the ungauged catchments were transferred from gauged neighbours, where direct calibration had been undertaken. Physically based parameters such as catchment area, inputted rain gauges and their weights were set using observed data. The parameters controlling the timing and shape of the hydrograph were also adjusted. This was achieved by taking the ratio of the time to peak (from FSU catchment descriptors) for the gauged and ungauged catchment and adjusting the surface routing time constant accordingly. Values for all other remaining parameters were retained.

Parameters for the ungauged sub catchments of the Shournagh (Martin_Lat, Owen and WillisonsBr in Table 7-1) were verified against observed flows at Healy's Bridge and can therefore be considered to be indirectly calibrated. The large InniscarraPS PDM is also indirectly calibrated as it inflows to Carrigadrohid and Inniscarra. Only the Bishops PDM is entirely unvalidated against observed data and it joins downstream of Waterworks Weir. That model has parameters donated from the Bride at Ovens with an adjustment for Tp.

PDM parameters (for all catchments) and calibration performance (for gauged catchments) are documented in model evaluation sheets in Appendix D and tabulated below (Table 7-1). The values are all in line with those expected for wet catchments with high rates of runoff and no

[18] Moore, R.J. (2007). The PDM rainfall-runoff model. Hydrol. Earth Syst. Sci 11(1), 483-499.

parameters are unusual. The model was also verified as a whole for a longer period using levels and outflows from the reservoirs. Model proving is discussed in Section 0.

Table 7-1: PDM parameter values

	Area	Fc	Cmin	Cmax	b	Be	Kg	Bg	K1	K2	Kb	Tdly
	Km^2	mm	mm	mm	hrs	hrs	hmm^2	m^3/s	hrs	hrs	hmm^2	
Bishops*	46.29	1	0	60	2	2.5	500	1.7	3	-	36.8	0
Dripsey	76.6	0.95	10	100	1	2	6000	2	2	-	36.8	2
Dromcarra	169.5	1.05	10	130	1	3	8000	1.7	8	-	4.6	0
Gothic	22.5	1.05	0	60	1	2.5	1000	1.7	4	20	21.5	1
InniscarraPS*	240.8	1	10	90	1	3	700	1.7	4	-	36.8	0
Kill	86.1	1	0	80	1	3	5000	1.8	2	6	36.8	1
Kilmona	39.6	1	0	60	0.75	2.5	200	1.5	1	1.5	79.4	1.75
Macroom	213	1	10	100	1	2	8000	1.7	4	-	4.6	2
Martin_Lat*	23.6	1	0	60	0.75	2.5	200	1.5	2	1.5	79.4	0
Ovens	121.6	1	0	60	2	2	500	1.7	8.5	-	36.8	0
Owen*	45	1	10	60	1	1	2800	1.7	2	-	36.8	2
WillisonsBr*	71	1	20	100	0.75	1	8000	1.7	2	-	3.684	2.5

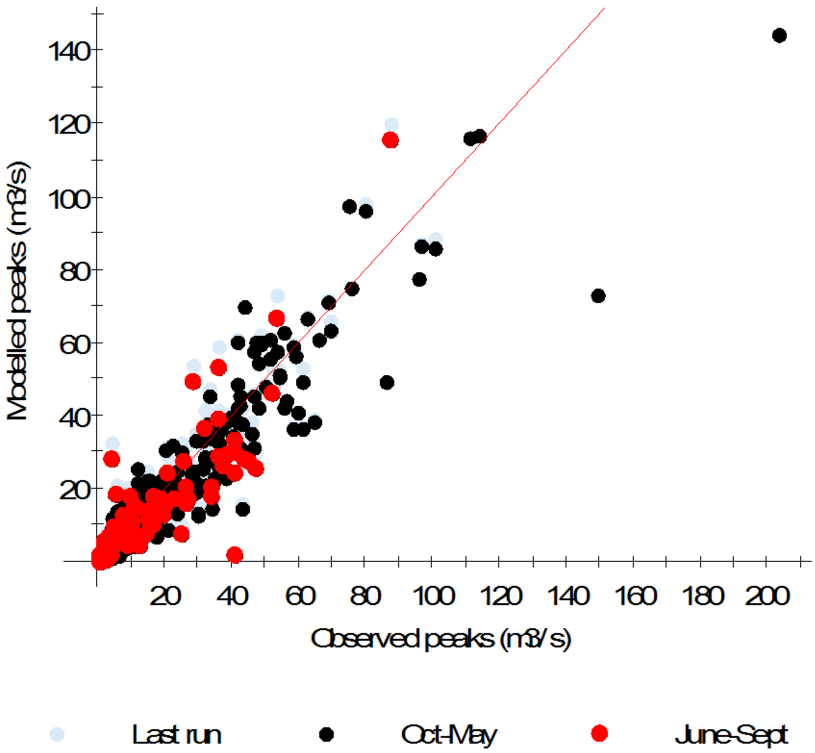
Notes: refer to Figure 7-2 for an explanation of the parameters and to Figure 6-2 to see the location of the catchments. Rain gauge weights are given below. All PDMs have a cubic baseflow store, a Pareto distribution for the soil and use gravity drainage. The St parameter is zero in all cases.

Table 7-2: Rain gauge weights

PDM Name	Ballvourney	Bottle Hill	Bweeng	Carrigadrohid	Carriganimma	Gearagh	Gougaune	Inchigeelagh	Inniscarra	Mushera	Renanirree
Bishops	-	-	0.01	-	-	-	-	-	0.99	-	-
Dripsey	-	-	0.3	0.6	-	-	-	-	0.05	0.05	-
Dromcarra	0.05	-	-	-	-	0.053	0.421	0.421	-	-	0.105
Gothic	-	0.297	0.222	-	-	-	-	-	0.481	-	-
InniscarraPS	-	-	-	0.388	-	-	-	0.019	0.583	0.01	-
Kill	-	-	0.05	0.05	0.45	-	-	-	-	0.55	-
Kilmona	-	0.6	0.3	-	-	-	-	-	0.015	-	0.015
Macroom	0.592	-	-	-	0.107	-	-	-	-	-	0.194
Martin_Lat	-	0.6	-	-	-	-	-	-	0.015	-	-
Ovens	-	-	-	0.521	-	-	-	0.054	0.277	-	0.148
Owen	-	-	0.7	-	-	-	-	-	0.3	0.01	0.01
WillisonsBr	-	0.15	0.7	-	-	-	-	-	0.2	0.05	-

Sensitivity of simulated flows to PE was tested by running the model with a maxima of 2mm/day rather than 3mm/day. The results at Dromcarra (which has the largest Cmax of the PDMs) are shown below (Figure 7-3) and illustrate the lack of sensitivity.

Figure 7-3: Peak simulated and observed flows with 3mm/day PE maxima and 2mm/day (grey points)



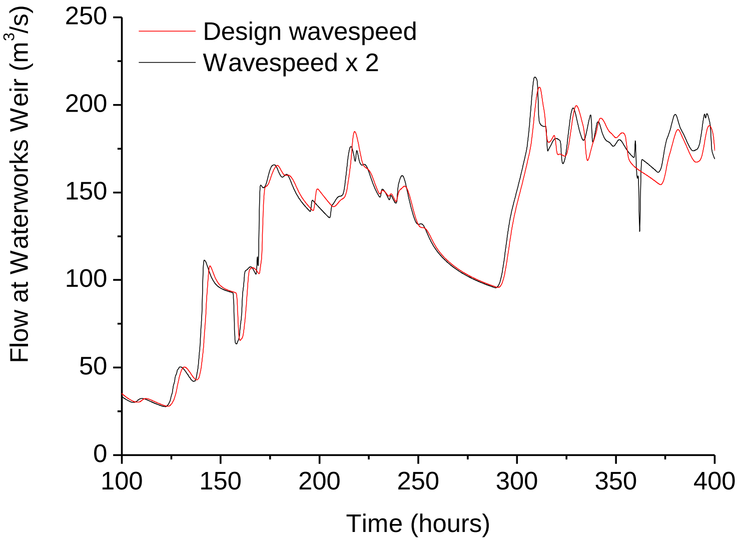
7.6 River model development

7.6.1 General

Carrigadrohid and Inniscarra Reservoirs are the dominant hydraulic features of the Lee river network. Their size, outflow structures and operational control rules are the main part of the river model. Also included are flow routing reaches to translate the hydrograph through the river network and flow boundaries that take the inputs from the PDMs.

River reaches within the model (mainly downstream of the reservoirs) are short and their limited attenuation effects can be adequately described using the Muskingum Cunge flow routing method. The critical reach is between Inniscarra and Waterworks Weir, and there was sufficient calibration data (outflow from Inniscarra and observed levels at Waterworks Weir) available to set the wavespeed for this part of the model. A fixed value of 1m/s was applied to all the model's routing reaches, but sensitivity testing showed that doubling the wave speed made very little difference to the peak flow in Cork. This is illustrated in the plot below (Figure 7-4) which shows an event from the stochastic simulation with the wavespeed as per the design model and double. However, the detailed 1D-2D hydraulic model does give a faster wavespeed, but without reliable data for larger events at Waterworks Weir the wavespeed in the Routing model has been left to be calibrated as part of the FEWS implementation.

Figure 7-4: Simulated flow for a synthetic event at waterworks weir with original and scaled wavespeed



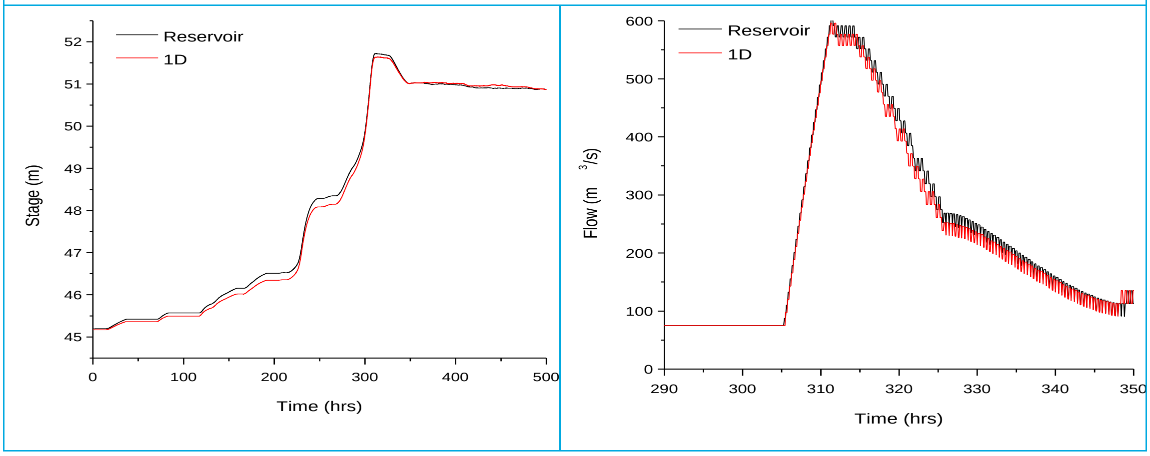
16 flow boundaries feed the model with flow from the 12 PDMs.

7.6.2 Model geometry

Carrigadrohid and Inniscarra Reservoirs are modelled using reservoir units, which employ level pool routing to translate flow. The reservoirs themselves are defined as elevation-surface area tables, with these dimensions derived from the existing CFRAM hydraulic model. That model used cross sections to simulate the reservoirs in a 1D representation.

Inniscarra is a long, narrow reservoir, which may therefore develop a surface gradient - affecting the 'flat surface' assumption made in our reservoir approach. We checked this by comparing otherwise identical models: one with a reservoir unit for Inniscarra, one using the original cross section data. Both gave very similar answers, as shown below for a simulation of the November 2009 event. This satisfied us that reservoir units are appropriate for both Inniscarra and Carrigadrohid (which is much wider and less 'channel-like'). Reservoirs are more stable than 1D reaches and can be 'state updated' in real time in a forecasting system - a crucially important factor for the predictive role of the model.

Figure 7-5: Comparison of simulated water level in and flow out of Inniscarra for 1D and reservoir models



Volumes for reservoir units were obtained from the 1D CFRAM model by:

- Replacing the CFRAM model's reservoir outflow (QHBDY) with a vertical sluice and removing everything downstream

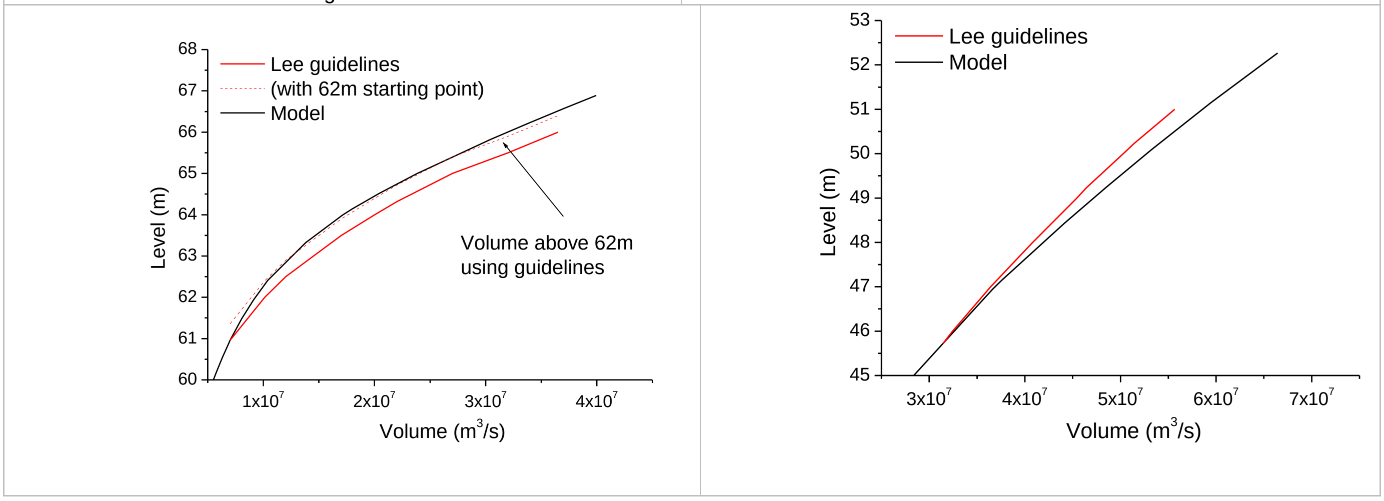
- Running it model from a low initial water level with a constant flow and closing the sluice gate within the first hour of the run
- Continuing the simulation until the water level in the reservoir reached the required maximum (this simulation ran for 1,000hrs)
- Pairing the cumulative inflow (i.e. total volume) with the level in the cross section upstream of the Dam.

Stage-volume relationships for Carrigadrohid and Inniscarra are plotted below with those from the Lower Lee guidelines. These do not match exactly, but are close (Figure 7-6). Carrigadrohid total volume is smaller than the guidelines, but if 62m is taken as the baseline water level then the volumes above this are almost an exact match. This is important because water levels are unlikely to fall below that level during ay wet period. Volume in Inniscarra is slightly larger and diverges with increasing level: for example at 50m it is almost 5% greater. A discrepancy that is mostly stable throughout the range. The relationships based on cross section survey data were taken forward, in preference to the tables from the guidelines, as being derived from a known data source.

The impact of using ESB dimensions for both reservoirs, over those from the CFRAM model, was

Figure 7-6: Model depth - storage relationships compared to those from the ESB guidelines

Carrigadrohid | Inniscarra



sensitivity tested for the design case using otherwise identical models. The ESB dimensions (transcribed from volume to area relationships as required by ISIS) gave slightly higher design flows because of the smaller area available in Inniscarra. However, results at the 100-year return period for the Design case at Cork were within 1% of one another and were within 2% at all other return periods. Agreement at the point of interest demonstrated that the sensitivity to this decision was small.

Outflow from the reservoirs is controlled in the model by a combination of:

- An abstraction unit to simulate discharge through the turbines at Carrigadrohid (max 75m^3/s) and Inniscarra (max 80m^3/s). The abstraction is controlled by logical rules which decide how much water to discharge on the basis of reservoir levels. The rules aim to keep the reservoirs at an operational head called Maximum Normal Operating Level (MNOL, refer to Table 7-7 for the values used in the different scenarios). The suitability of abstraction units for this purpose was checked by looking at observed data. In December 2010, Inniscarra reservoir levels were just below 46m when a discharge of almost 80m3/s was observed - confirming the choice of model unit. At Carrigadrohid, releases are always possible through the 'deep' sluices.
- A vertical sluice gate at both Inniscarra and Carrigadrohid, representing all three sluices as a single unit with physically based dimensions (Table 7-3). The gates are controlled by a complex set of logical rules which aim to replicate the 'real' flood operation of the reservoirs (the rules are outlined in Appendix F). In the 'Baseline' scenario, this includes pre-releases of an average of 120m^3/s to lower water levels and the operational tables specifying outflow rates for reservoir levels for the 'rising flood'.

- At Carrigadrohid, a spillway which takes flow in excess of that which can be handled by the three 'deep sluices' in the dam (dimensions in Table 7-3).

Section 8.3 discusses the operation of the reservoirs by the model in more detail.

Dimensions for the sluice gates are taken from the Guidelines document (see Table 7-3). The ISIS model is calibrated to reproduce flow rates published in the guidelines for a given head as closely as possible. Calibration involved adjusting the under gate flow coefficient from its ISIS default of 1.0 - which is analogous to a 'sharp edged orifice' - to 1.3 for an orifice more representing a 'short tube'. The aim was to match the outflow from the gates under a given head to that predicted in the ESB guidelines. Table 7-4 compares sluice outflows calculated by the model with those taken from the Guidelines document. The agreement is within 6% at the highest levels and considered an adequate match.

Table 7-3: Sluice gate dimensions and coefficients

	Carrigadrohid		**Inniscarra**
	Sluice	**Overspill**	**Sluice**
Crest elevation (m)	47.24	65.2	45.11
Weir breadth (m)	9.15	50	36.57
Gate height (m)	Large		5.79
Under gate flow coefficient	1.3		1.2
Weir flow coefficient	1.0	1	1.0

Table 7-4: Check on model discharges against tables in guidelines

Carrigadrohid				
Water level (m)	**Gate opening (cm)**	**Discharge required by guidelines (m3/s)[1]**	**Model discharge (m3/s)[2]**	**Difference**
65.2	488	585	588	+1%
65	390	450	474	+5%
64.84	310	350	378	+8%
64.66	180	200	219	+10%
Inniscarra				
50.85	250	550	585	+6%
50.80	210	475	510	+7%
50.40	170	375	399	+6%
50.20	150	325	354	+9%

Notes:
[1]Discharge and gate openings are from Tables 1.3 and 1.8 of the Lee Guidelines, which tell operators how to achieve the required outflow for a given level. They are checked against the printed discharge curves in the appendices.
[2]A version of the model was run using a range of flows and levels. The gate opening/water level combinations were 'read off' the resulting plots.

Another critical part of model behaviour is that, in flood conditions, it observes the discharges prescribed by the Lee Guidelines (in the 2007 edition of the rules). Those flows are given in tabular form by the guidelines document. We parameterised the relationship so that a flow can be calculated on the basis of reservoir level. Figure 7-7 shows the parameterised relationship (red line) plotted with the tabulated values (black squares) for both reservoirs. During model simulations, the 'aimed for' discharge is calculated continuously (red line in Figure 7-7) and the gates move to try to match that flow (grey line in Figure 7-7). The demonstration illustrates that the model is making sensible releases when the discharge tables should be followed.

Figure 7-7: Parameterised discharge tables from the Lee Guidelines

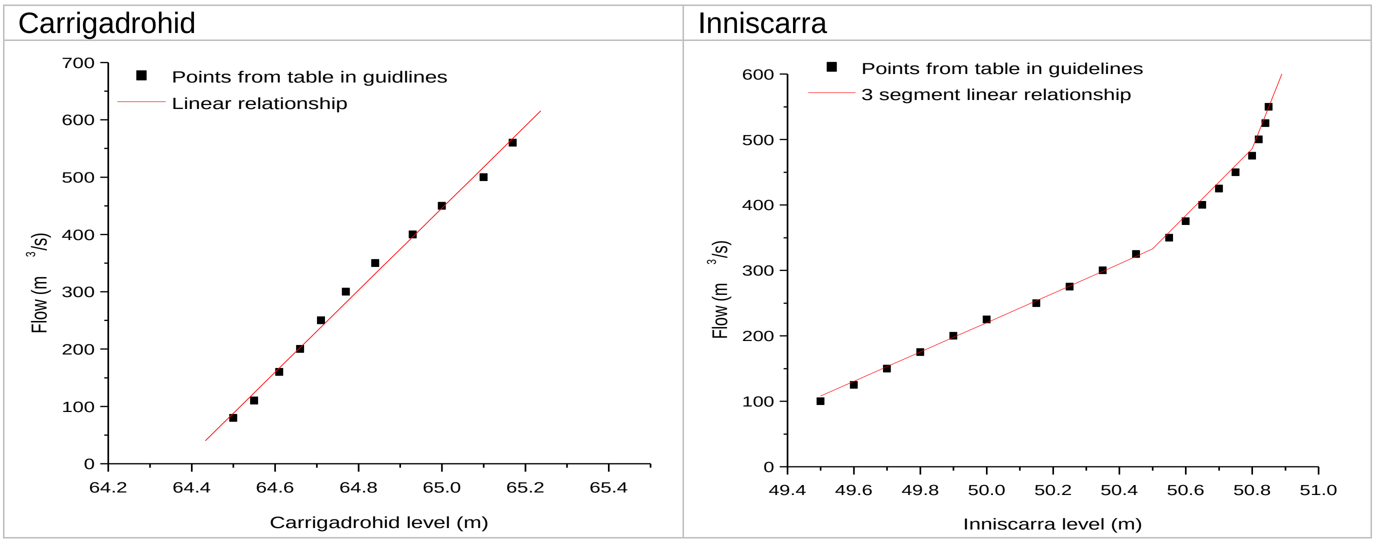
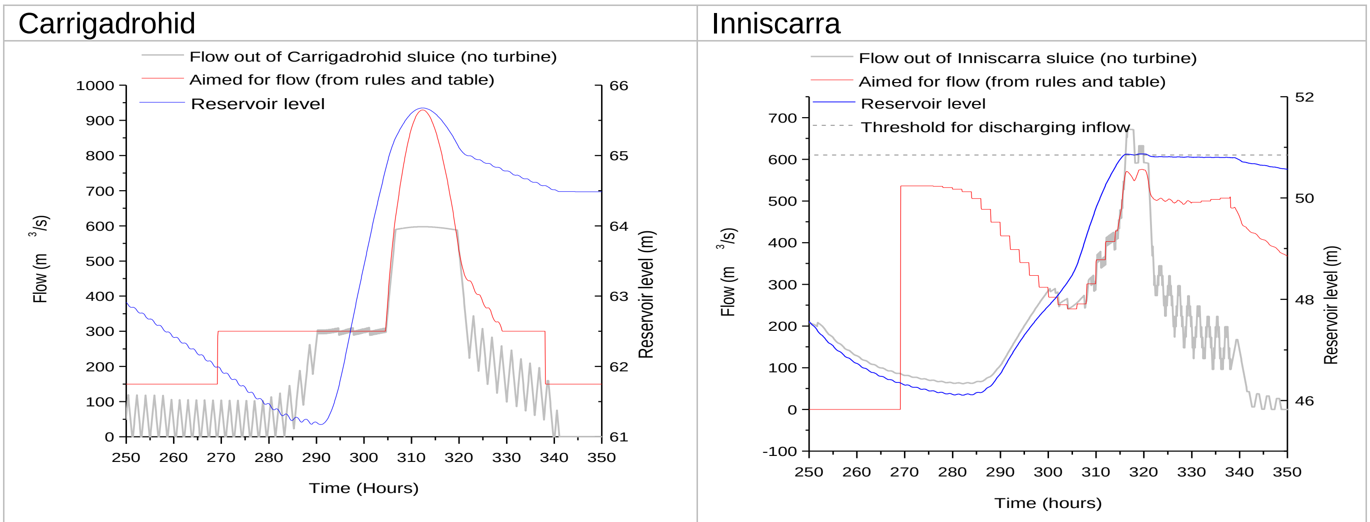


Figure 7-8: 'Aimed for' flow (red), calculated from relationships fitted to the tables in the guidelines, and the actual flow (grey)



Note that the oscillations in 'flow out of Carrigadrohid sluice' (grey line) are a result of flows being stopped and started to maintain the maximum allowable drawdown rate in Carrigadrohid (variously 0.6m per 24 hours or 1.0m per 24 hours). Observe the small steps in the receeding Reservoir level (blue) as drawdown is checked to maintain the required rate.

7.6.3 Logical rules

Beyond its basic geometry, the rules that control the sluice and turbine operation are the key part of the river model. These are, by necessity, complex. Existing and proposed gate operation takes several variables into account, including:

- Current inflows to the reservoirs
- The rate of change of inflows
- The level in the reservoirs
- The rate of change in level in Carrigadrohid reservoir
- The future inflow to the reservoir and in the Shournough and Bride
- The current gate setting

ISIS allows some unit types (sluices, abstractions etc.) to be controlled by a set of logical statements (If…Then…else). This functionality controls the operation of the gates. There is too much complexity to achieve this by coding rules into the gates alone. Instead, abstraction nodes are used to carry out many calculations and 'store' results as a flow. Gates can reference flow in abstraction nodes to achieve what they need to. The process is analogous to a computer programme with sub routines and variables.

Appendix F contains a schematic that illustrates how the different units interact with one another to achieve the desired result.

7.6.4 Overview of model operation

A simple overview of how the model operates the reservoirs is as follows:

During normal conditions (i.e. no flood is predicted or ongoing), an abstraction unit releases water to maintain the reservoir at a prescribed level (assumed for purposes of testing new rules to be Maximum normal operating level). Two of these represent the turbines. Up to 75m^3/s can be abstracted from Carrigadrohid and up to 80m^3/s from Inniscarra. Abstractions ignore the available head.

When a flood is forecast, additional releases are made from the reservoirs to create storage. The rate of release is parameterised and can be adjusted by the modeller (or, in the forecasting system, by the forecaster). If the reservoirs exceed MNOL and rising flood conditions are in place, releases are made according to the tables set out in the Guidelines for operation of the Lee reservoirs.

To achieve this, the model uses three 'forward offset' flow series' that represent all the inflow to Carrigadrohid. One looks 96 hours ahead, another 48 hours and the third 24 hours. This allows the model to know when a flood is expected and how soon. An abstraction unit (FloodPrdctd) references these and returns a flow indicating if, and how soon, a flood is expected.

Rules governing the releases during 'rising flood' conditions are also represented as abstraction unit (CarrigTable and InnisTable). The abstraction returns a flow rate which is appropriate for the current model time. It is calculated using the current reservoir level and the parameterised discharge table.

Carrigadrohid sluice then references the FloodPtrdctd node, the CarrigTable node, the reservoir level and the previous reservoir level (an hour ago) to determine how the gates should move.

Inniscarra sluice is similar, but has additional complications. It also references:

- An abstraction unit (RISINGFLD) which indicates if the 'rising flood' condition specified in the guidelines is in force;
- A pair of abstraction units that determine what the 'aimed for' release should be from the dam
- A complex abstraction unit that controls gate movement during the pre-release phase (called PreRelease) which in turn references the FloodPrdctd abstraction and a Carrigadrohid pre release unit (CarrigOT).

The overall effect is to allow the reservoirs to be simulated as they are now and as they might be after scheme implementation. A simulation from the stochastic series is outlined and explained in Section 8.

Reservoir operational behaviour is controlled by a set of parameters, referenced by the model. The model can be made to behave as the 'Baseline' or in a way that reflects the 'Design' case, just by changing the parameters. Parameters are outlined below for both cases and are best understood alongside an explanation of the proposed operational procedures for the reservoirs which is included in the Flood Relief Scheme Options Report and replicated in Appendix G.

7.6.5 Model proving

Without comprehensive hydrometric data series covering multiple large events, it is difficult to prove the model's performance entirely. There are some reasonable data available however, including from the very large November 2009 flood, and we have made the most of these resources to demonstrate that the model is reasonable. We looked at the model's outputs in four main ways to gain confidence in its performance:

1. Comparing simulated flows from PDM models with observed data at any directly gauged catchments
2. Comparing long term simulated outflows and levels with observe at Carrigadrohid and Inniscarra reservoirs
3. Looking at simulations of the November 2009 event in detail; and
4. Checking the behaviour of the ISIS routing model for one or more of the stochastic events from the continuous simulation (this is more relevant for the design case).

Each of these is considered in turn with supporting evidence in the report appendices.

7.6.6 Key model parameters

Table 7-5: Reservoir operation parameters used in Design and Baseline scenarios

Parameter		Description	Scenario: Design	Scenario: Baseline
Inflow threshold	96hrs	Threshold indicating that flood operations (i.e. drawdown) will be required. Tested against the total inflow to Carrigadrohid reservoir. Note this is the actual flow threshold, not that used with forecasts, which is lower.	400	n/a
	48hrs		400	150
MNOL	Carrig	Maximum normal operating level. Level assumed to be maintained when no flood is forecast.	64.5m	63.3m
	Innis		49.5	48.5
FRL	Carrig	Flood Risk Level. Level aimed for in reservoirs at a point 48 hours before the inflow threshold is crossed	63.1	n/a
	Innis		48.0	n/a
Minimum level	Carrig	Minimum allowable level in both reservoirs	61.0	61.0
	Innis		45.7	45.7
Turbine	Carrig	Allowable flow through the turbine is available during the drawdown and flood period	0	0
	Innis		0	0
Lead times	Early	Lead time when drawdown to FRL occurs, based on a forecast threshold crossing (see inflow threshold)	96hrs	n/a
	Mid	Beginning of prescribed flow releases (rather than drawing down to a target level)	48hrs	48hrs
	Late	Release rate increases	24hrs	24hrs
Carrig spill		Allowable release from Inniscarra when Carrigadrohid is spilling water	150m^3/s	120m^3/s
Max Early pre event release rate	Carrig	Maximum release rate allowed at an early lead time (96 to 48hrs). Note that maximum Carrigadrohid discharge is conditional on levels in Inniscarra. If Innis levels < 49, the rate is 300. If >-49.5 it is 150m^3/s. In between the limit is interpolated.	300m^3/s (->150m^3/s)	120m^3/s
	Innis		150m^3/s	120m^3/s
Target Mid pre event release rate	Carrig	Carrig operated in same way as for early release rate. Maximum discharge increased for Inniscarra.	300m^3/s (->150m^3/s)	120m^3/s
	Innis		200m^3/s	120m^3/s
Target Late pre event release rate	Carrig	Inniscarra target release rate is now regardless of Inniscarra levels.	300m^3/s	120m^3/s
	Innis	For the design, Inniscarra releases now back-calculated from: Target flow for Cork - (flow in Shournagh + Bride). For the 'Baseline' it is a fixed release.	540m^3/s	120m^3/s

7.6.7 PDM simulations

Simulated flow from the PDM models is compared to the observed for all events having data available (refer to model evaluation sheets in Appendix D). Although observations are sporadic in places, the results seem reasonable for events where data is available and reliable. Importantly, the PDM's accuracy should improve as the catchment tends towards full saturation - as will happen in higher order events. This is because there is less room for error as the soil is fully saturated; runoff rates will be consistently high. A brief summary interpretation of the full model evaluation sheets in Appendix is given at gauged locations below.

Model performance in the Upper catchment (Dromcarra, Macroom, Kill, sum of these three and Dripsey) is mostly good. Rainfall is the most likely reason for large discrepancies between simulated and observed (either erroneous data, missing data or sparse coverage - now addressed by the newer gauges).

Flow at Dromcarra in November 2009 is a good example of this. The model predicts 149m^3/s but the observed was reportedly greater than 200m^3/s. In that event, rainfall was only available at

Inchigeelagh, Renaniree and Ballvourney. There is now a gauge at Gougane Barra (in the upper catchment) which, on the basis of recent events, can record >60% more than Inchigeelagh. For example:

- In October 2013 it recorded 85mm against Inchigeelagh's 52mm
- In December 2013, it recorded 114mm and Inchigeelagh recorded 105mm.
- In January 2014, it recorded 238mm and Inchigeelagh recorded 160mm.

Therefore, if better rainfall were available in November 2009, the simulated would have been much closer to the observed at Dromcarra. Performance of the PDMs combining to give the total inflow to Carrigadrohid in November 2009 is good despite this. The model predicted a peak inflow within 5% of that calculated by ESB (refer to Table 7-6).

Simulated results in the downstream tributaries, like the Shournagh and Bride, still look good when rainfall are accurate, but there is more variability in results. The drier eastern part of the catchment is more permeable (parts of the Shournagh) and there are some floodplain effects visible (on the Bride). Ultimately, the models seem to work well in large scale rainfall events (like November 2009) of the type that are likely to trigger flood operations on the Lee. For example in 2009, the simulated peak for the Bride was 60m^3/s at Ovens - close to the 70m^3/s observed value.

In conclusion:

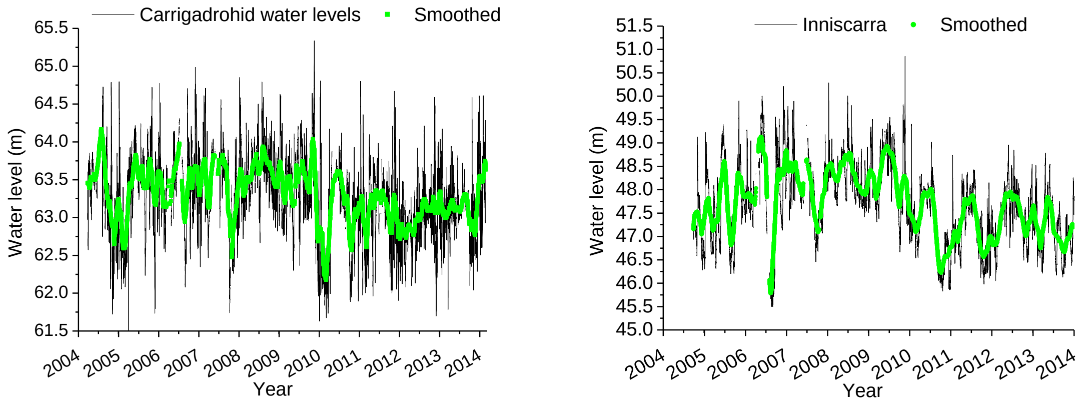
- When fed with reliable rainfall, the PDMs simulate flow with good accuracy - especially in the upper catchment and in winter conditions
- The model parameters are as good as they can be within the constraints of the data available. They are in line with what is expected for these types of catchment.
- Some minor adjustment of the rainfall scaling factor may be needed for Dromcarra PDM when enough data is available.
- PDMs in the lower catchment (Shournagh and Bride) have slightly more uncertainty than those in the upper. However they are less critical to the operation of the system.

7.6.8 Long term reservoir simulations

A simulation of the 'Baseline' scenario was carried out in the Flood Early Warning System (FEWS) system for 2007 to 2016. FEWS provides an open data handling platform for managing hydrological forecasting processes and warning systems. It used real forecast rainfall to trigger drawdown operations, but observed rainfall to simulate runoff into the reservoirs. The reservoirs themselves were controlled automatically according to the parameters in Table 7-5. Although those parameters are not a perfect representation of how the reservoirs are operated, they at least give a means for comparison.

Following the run, simulated outflows and levels from Carrigadrohid and Inniscarra reservoirs were compared to the observed series. These are valuable because they are the most complete hydrometric datasets in the catchment, allowing a near continuous simulation of flow for the period 2007 to 2016. Reservoir outflows are very much controlled by the operation of the reservoirs, so the 'natural' response of the system is obscured. Reservoir operation has changed over time and, when not in the 'rising flood' situation, reservoir levels are managed subjectively. However, reviewing the performance in flood periods allowed us to check for general response patterns and any falsely predicted floods. Some significant high flow events for the period are plotted in Appendix E. These show no events that are seriously over predicted and the general pattern of releases from the reservoirs is reasonable. In interpreting this data the reservoirs appear to have generally been maintained at a lower level since November 2009 than before (Figure 7-9). The model is therefore less accurate for the earlier period since its rules are based on the post 2009 dataset.

Figure 7-9: Observed water levels at Carrigadrohid and Inniscarra showing changes since 2009



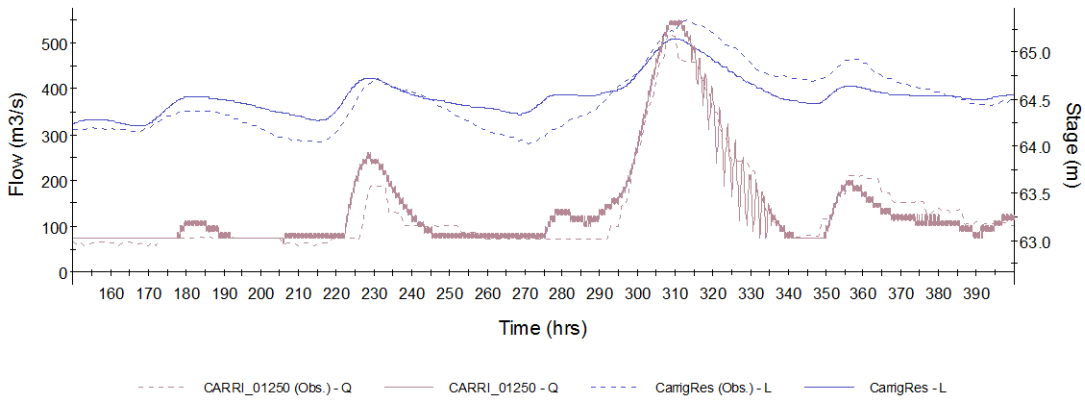
The smoothed (green) reservoir levels are a moving average designed to highlight how maintained reservoir levels changed after the November 2009 event.

7.6.9 November 2009 event

More closely examining performance in the November 2009 event at, and downstream of, the two reservoirs gives our best insight into how the model reproduces the existing system in a large flood. November 2009 and December 2015 are the only really big floods where data is available. Simulating it accurately gave confidence that a) runoff predictions from the PDMs were reasonable; b) pre-releases from the reservoirs were generally similar to those observed; and c) outflows from the reservoirs matched the discharge tables in the 'rising flood' section of the operational procedures. A dedicated run, using adjusted parameters was carried out for this important event.

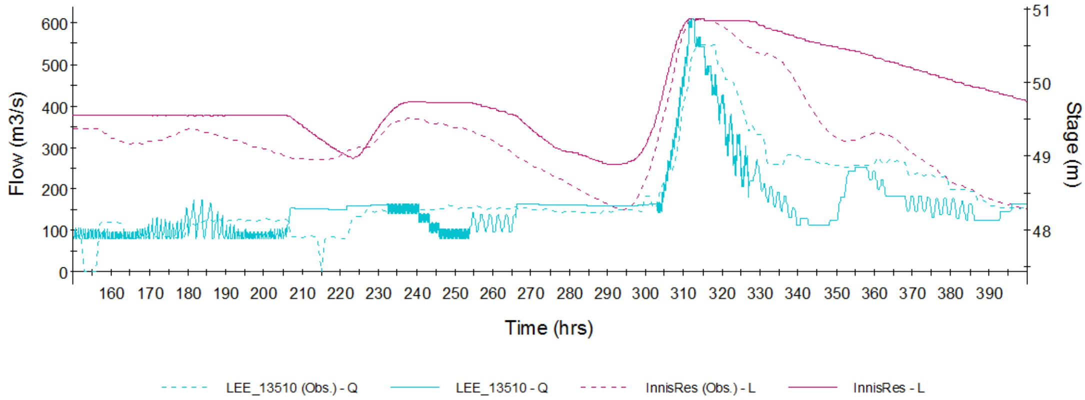
Reservoir levels and outflow hydrographs from the specific run are shown in Figure 7-10 and Figure 7-11 below. A summary of some of the key maximum flows and levels are given in Table 7-6. Given that these simulations are from rainfall data (with variable quality), continuously simulated using PDMs, and the releases are being governed by logical rules, the accuracy of the predictions of level and flow is impressive. The logical rules cannot simulate the actual operational actions during such a large event, hence the differences in levels before and after the peak of the event. Our simulated inflow to Carrigadrohid is just 5% higher than that estimated by ESB on the basis of releases made and known changes in reservoir volume. Outflow from Carrigadrohid is 6% higher and from Inniscarra it is just under 12% higher. Thus, the model predicts a higher flow than observations made at the dams, but the agreement is close.

Figure 7-10: Simulated and observed level and outflow from Carrigadrohid in November 2009



Note: Observed levels are shown as dashed, simulated as solid. CARRI_01250 is the outflow, CarrigRes the reservoir level.

Figure 7-11: Simulated and observed level and outflow from Inniscarra in November 2009



Note: Observed levels are shown as dashed, simulated as solid. LEE_13510 is the outflow, InnisRes the reservoir level.

Table 7-6: Modelled and observed maxima at important locations for the November 2009 event simulation (where available)

	Maximum flow (m^3/s)			Maximum stage within reservoirs(mOD)		
	Obs	Sim	% diff	Obs	Sim	Diff
Total Inflow to Carrig	494[1]	521	+5%			
Carrigadrohid	518	549	+6%	65.21	65.14 (65.34[2])	-0.07 (+0.13)
Inniscarra	546	609	+12%	50.85	50.85	0
Waterworks Weir	n/a	678				
Healy's Bridge	102[3]	132	+29%			
Ovens	70	60	-14%			

Notes

[1] This flow value is from the ESB calculated derived inflow series to Carrigadrohid (supplied to JBA)

[2] The maximum stage in Carrigadrohid was 65.34m, but this occurred AFTER the flow peaked. At the time of the peak flow, the level was 65.14m.

[3] The maximum flow at Healy's Bridge was estimated from maximum stage observations by EPA following the 2009 event. Should be treated with caution.

Parameters controlling the behaviour of the reservoirs in the model are slightly different to those applied in the Baseline simulation (listed in Table 7-7). Adjustments were required to get a reasonable match to the observed level and flow in the reservoirs. The differences to the Baseline parameters are shown in the table below.

Table 7-7: Reservoir operation parameters used in Baseline and November 2009 scenarios

Parameter		**Description**	**Scenario**	
			Nov 2009	**Baseline**
Inflow threshold	96hrs	Threshold indicating that flood operations (i.e. drawdown) will be required. Tested against the total inflow to Carrigadrohid reservoir. Note this is the actual flow threshold, not that used with forecasts, which is lower.	$400m^3/s$	n/a
	48hrs		$400m^3/s$	$150m^3/s$
MNOL	Carrig	Maximum normal operating level. Level assumed to be maintained when no flood is forecast.	same	63.3m
	Innis		same	48.5m
FRL	Carrig	Flood Risk Level. Level aimed for in reservoirs at a point 48 hours before the inflow threshold is crossed	same	n/a
	Innis		same	n/a
Minimum level	Carrig	Minimum allowable level in both reservoirs	same	61.0m
	Innis		same	45.7m
Turbine	Carrig	Allowable flow through the turbine is available during the drawdown and flood period	75	0
	Innis		80	0
Lead times	Early	Lead time when drawdown to FRL occurs, based on a forecast threshold crossing (see inflow threshold)	same	n/a
	Mid	Beginning of prescribed flow releases (rather than drawing down to a target level)	same	48hrs
	Late	Release rate increases	same	24hrs
Carrig spill		Allowable release from Inniscarra when Carrigadrohid is spilling water	$150m^3/s$	$120m^3/s$
Max Early	Carrig	Maximum release rate allowed at an early lead time	$150m^3/s$	$120m^3/s$

release rate	Innis	(96 to 48hrs). Note that maximum Carrigadrohid discharge is conditional on levels in Inniscarra. If Innis levels < 49, the rate is 300. If >-49.5 it is 150m^3/s. In between the limit is interpolated.	150m^3/s	120m^3/s
Target Mid release rate	Carrig	Carrig operated in same way as for early release rate.	150m^3/s	120m^3/s
	Innis	Maximum discharge increased for Inniscarra.	150m^3/s	120m^3/s
Target Late release rate	Carrig	Inniscarra target release rate is now regardless of Inniscarra levels.	150m^3/s	120m^3/s
	Innis	For the design, Inniscarra releases now back-calculated from: Target flow for Cork - (flow in Shournagh + Bride). For the 'Baseline' it is a fixed release.	150m^3/s	120m^3/s

7.6.10 Winter events of 2015/2016

Subsequent to the design of the operational rules a prolonged period of continuous storm events occurred, in a similar pattern to the extreme event in 2009. Two significant high flow events occurred in the Lee catchment in December 2015. They rank higher than any event previously simulated except November 2009 and provide new information to:

- Further prove the forecasting models;
- Re-test the proposed operational procedures and their trigger thresholds
- The performance of the foreasting system was tested

The results of this further testing showed that the individual forecasting models performed well during December 2015. We do not recommend any further calibration/adjustment to the underlying PDM models until further improvements in the gauging station network are achieved.

Operational Procedures worked as expected. The event(s) were detected in advance and flows would have been easily constrained to 500m3/s. There was very little wasted (spilled) water and reservoir levels were shown to recover to MNOL quickly should that be needed by ESB. The Forecasting System also worked as expected and made the correct decisions about how to draw down and refill the reservoirs. Trigger levels still seem appropriate and we see no reason to adjust them.

Further details of this testing are provided in Appendix H.

7.6.11 Sensibility checks on stochastic simulation

Complex logical rules control the ISIS model and are difficult to verify in isolation. Figure 7-12 is a way of visualising the behaviour of the ISIS model for an event in the stochastic simulation. It is a check that the model is working correctly.

The ISIS model takes action to mitigate a flood when flows into Carrigadrohid (Plot 1 in Figure 7-12) exceed 400m^3/s. Otherwise it uses turbine capacity to maintain reservoir levels at MNOL and spills water according to the discharge tables (in the Guidelines for operating the Lee reservoirs) when turbines are insufficient. Taking action means lowering water levels in the reservoirs at lead times of 96 hours and shorter, then using the storage created to minimise flow in Cork. The proposed operational rules are described in Appendix G; this part of the report simply illustrates that the model follows them faithfully.

Plots in Figure 7-12 show a flood event from the stochastic series run through the Lee ISIS routing model (Event 06Nov2256 – approximately the 150-year event at Waterworks Weir). It aims to simulate the proposed operational rules for Carrigadrohid and Inniscarra reservoirs. These graphs are used to demonstrate that the model is correctly simulating the operation of the rules and reservoirs. There are 6 graphs, each showing different parameters that indicate how the model is behaving. Important milestones in the flood event are indicated by a letter (A to E in Plot 1), each with a pink line linking that time in the other plots. Each is described below.

1. Total inflow to Carrigadrohid (to get the actual flow, multiply by 1000). This is the time series that triggers drawdown operation at 400m^3/s. This happens 295 hours into the simulation (point D). In this event there are is small initial peak followed by a large one.

2. Indicates what flood conditions the model is anticipating on the basis of the total inflow to Carrigadrohid. If the red line is 0.005, a flood is anticipated at 96 hours out (i.e. at 295-96hrs=199hrs). If it is 0.01, it is anticipated at 48 hours out. If it is 0.02, the flood is only 24 hours

away and this extends until the flood has passed. The black line shows when the Lee is in the 'rising flood' condition, as defined by ESB's Lee Flood Guidelines. If the black line is non zero, there is a 'rising flood'. Discharge tables are imposed on outflows from the dams when there is a rising flood condition.

3. Shows the level in Carrigadrohid (blue) and the total flow out through the dam (black). There are two rainfall events in this series, resulting in increasing peak flows (refer to Plot 1). The main flood is signalled 199 hours into the simulation. This switches off the turbine, which stays off until the peak of the event has passed (when the turbine has no impact). Gates begin to move to drop the reservoir at a maximum rate of 0.6m/24hours, without exceeding an outflow of 150m^3/s. An intermediate event occurs in this period, causing the reservoir to rise (as a 150m^3/s maximum is still enforced by high levels in Inniscarra). 'Rising flood' conditions are met and releases are made according to regulations. Reservoir levels begin to drop once more after the middle event has passed, reaching a low of around 64m. The main flood then arrives and flows are initially regulated to 300m^3/s. Reservoir levels then rise into the operating table range and releases are prescribed that exceed 300m^3/s. Flows through the gates are approx. 590m^3/s as the spillway level is reached (within 5% of the flow stipulated by the tables in the guidelines). They peak at around 604m^3/s (with some flow over the spillway).

4. Shows the level in Inniscarra (blue) and the total flow out (black). The release pattern is very similar to Carrigadrohid. The main difference is that the gates open to their full extent in the last phase of the event (to let out as much water as possible – resulting in free flow over the gate sill), only closing again to regulate the flow out of the dam according to flows in the Bride and Shournagh (Plot 5).

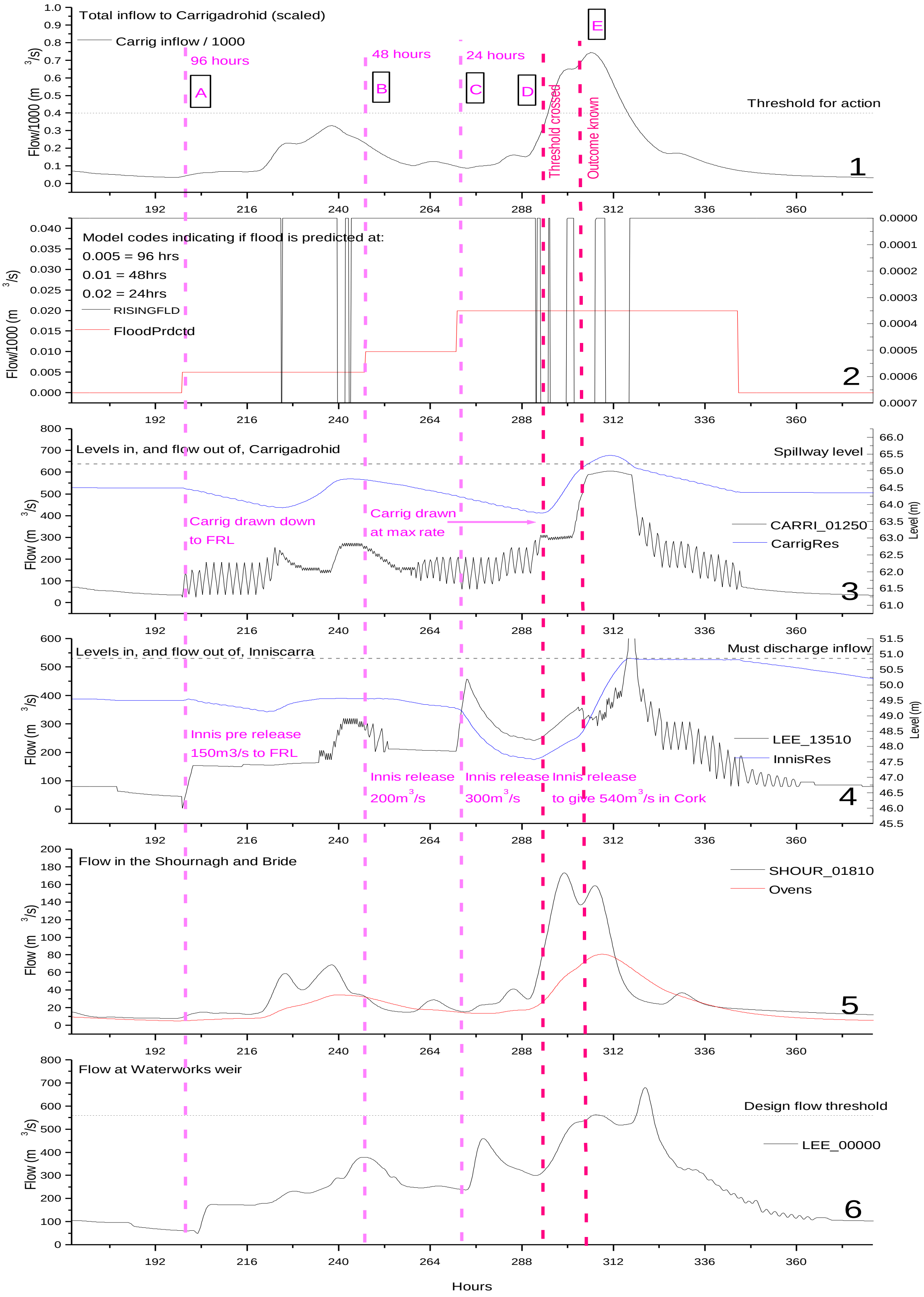
5. Shows the flow hydrographs for the Bride at Ovens (red) and Shournagh at Healy's Bridge (black).

6. Is the flow at Waterworks Weir (i.e. the Lee, Bride and Shournagh combined). In this event, a flow of 540m^3/s is maintained until storage in Inniscarra is exhausted and the control rules dictate that the reservoir discharges the inflow. Flows peak at just over 600m^3/s – an exceedance event.

7.7 Summary

These checks give us confidence that all parts of the model (rainfall runoff processes, flow routing and reservoir operation) work reasonably and produce sensible results - particularly in the highest flow events.

Figure 7-12: Example double peaked (exceedance) event from continuous simulation to illustrate the steps in the Flood Operations



8 Stochastic flow modelling

8.1 General

To derive design flows for Cork, the stochastic rainfall series is inputted to the PDM rainfall runoff models, together with the same annual PE sine curve used in calibration, to give a 1,000-year flow series' at all of the catchment model's boundaries. Fixed scaling factors account for variations in rainfall depth across the catchment. These are calculated according to the ratio of annual average rainfall in the Macroom catchment (on which the stochastic series is based) to the catchment of interest. The values used are tabulated below.

Table 8-1: Scaling factors applied to stochastic rainfall to account for difference in AAR from Macroom

PDM model	Rainfall scaling factor
Curraheen	0.62
Glasheen	0.62
Dripsey_REH	0.77
Dromcarra	1.17
Inniscarra PowerStn	0.93
Kilmona	0.68
Owen	0.65
Ovens	0.71
WillisonsBridge U/S	0.71
Gothic	0.64
Macroom	1
Kill	0.9
Martin Lateral	0.67

Rather than try and run the catchment model (the ISIS model) continuously for this period, we identified all annual maxima in the combined inflow series to Carrigadrohid where the flow exceeded QMED (the 2-year flow). Events exceeding QMED were also added from the flow series at three other PDMs, representative of the Bride and Shournagh (Ovens, Kilmona and Willisons Bridge). These additions ensured that large events on the tributaries were not overlooked. 870 simulations were required in total. Peak flow return periods of between 2-years and 200-years can be extracted from the set.

These storm hydrographs, beginning 300 hours before the inflow peak and ending 100 hours after, are fed to the ISIS catchment model. Annual maxima are then extracted from the results (there can be more than one event for a given year in the stochastic simulation) at all nodes and design flows calculated for the required return period using the Gringorten formula. It is expected that the design event for all nodes downstream of the reservoirs, for return periods greater than 2-years, will be somewhere within the 870 events executed.

Running a version of the catchment model without reservoirs allows us to compare with flows that have been calculated at waterworks weirs, using the FSU regression equation and simulating a no reservoir scenario, as detailed in Appendix B with a set from the continuous simulation. Results at the model boundaries can also be compared against gauged flow estimates from conventional methods detailed in the Appendices.

Hydrological Estimation Points are often adopted in flood studies in Ireland, and used as defined internal "calibration" points for the hydraulic model. It was agreed at the outset of this study that due to the reservoirs, the floodplains downstream and the lack of gauged data in the lower reaches that a process based model, driven by stochastic rainfall would be used. There will always be a difference between the routing model and the hydraulic model, as there will always be variations in output from the reservoirs depending on what starting level is assumed. This difference or uncertainty would be resolved in the scheme condition as the flow will remain in bank and an operational gauge installed at Waterworks Weir would allow the forecasting model to adjust the "design" flow.

8.2 No reservoir simulation and results

The reservoired ISIS model (used to simulate the 'Baseline' and 'design' cases), which was calibrated and proved in Section 7, was modified to remove Carrigadrohid and Inniscarra Reservoirs by:

- Taking out the reservoir units and all associated outflow structures (the abstraction, the sluice gates and the spill)
- Adding in a routing reach with a fixed wavespeed to replace the reservoirs.

Results of the 'no reservoirs' model run at Cork, and other model boundaries where gauged flow estimates have been calculated (See Appendix B), are tabulated and compared below (Table 8-2). Only Healy's Bridge and Waterworks Weir are nodes within the ISIS model, the rest of the sites are at model inflow boundaries.

Table 8-2: Comparison of peak flows for return periods at various locations in the catchment

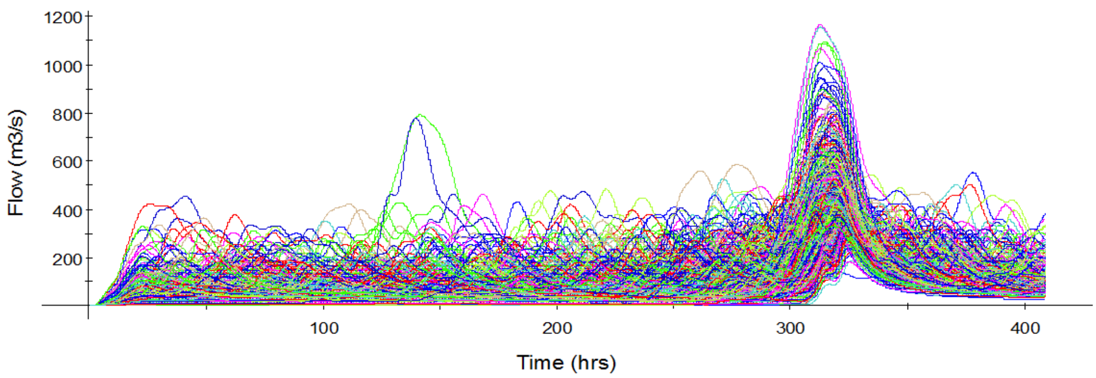
	Flow (m^3/s) for return period (years)								
Continuous simulation									
Location	**2**	**5**	**10**	**20**	**30**	**50**	**100**	**150**	**200**
Dripsey	28	37	44	52	57	64	73	75	78
Dromcarra	95	125	145	161	178	195	213	224	234
Ovens	34	46	53	61	67	75	82	86	91
Macroom	122	164	191	218	238	263	279	281	292
Kill	50	65	74	84	90	97	104	106	111
Total Carrig. Inflow	319	429	495	568	630	700	752	776	808
Healy's Bridge (ISIS)	75	100	118	136	147	161	180	187	192
Waterworks Weir (ISIS)	386	510	599	674	765	820	921	953	1039
Design flows from Single Site / FSR methodologies									
	2	**5**	**10**	**20**	**30**	**50**	**100**	**150**	**200**
Dripsey	41	60	74	92	100	103	116	125	152
Dromcarra	82	117	145	175	191	198	217	231	272
Ovens	27	40	52	71	76	78	86	91	107
Macroom	148	208	254	296	320	328	370	403	501
Kill	50	66	78	95	104	108	122	134	168
Healy's Bridge	63	89	107	129	141	145	165	182	230
Waterworks	364	513	623	754	817	845	944	1042	1301
Difference									
	2	**5**	**10**	**20**	**30**	**50**	**100**	**150**	**200**
Dripsey	-32%	-39%	-40%	-44%	-43%	-38%	-37%	-40%	-49%
Dromcarra	16%	7%	0%	-8%	-7%	-2%	-2%	-3%	-14%
Ovens	29%	16%	3%	-14%	-12%	-3%	-5%	-6%	-15%
Macroom	-18%	-21%	-25%	-26%	-26%	-20%	-24%	-30%	-42%
Kill	-1%	-2%	-5%	-12%	-14%	-10%	-15%	-21%	-34%
Healy's Bridge	19%	12%	10%	5%	4%	11%	9%	3%	-16%
Waterworks Weir	6%	-1%	-4%	-11%	-6%	-3%	-2%	-9%	-20%

Our key point of interest is at Waterworks Weir in Cork (node LEE_00000). Here, flow from the Lee, Shournagh and Bride has combined upstream of the city. This is the point where we want to compare the outputs from the continuous simulation with those derived using standard flood estimation methods. The flow we refer to here is the total flow from the three tributaries, not strictly the flow over Waterworks Weir. Local flood flow pathways mean that some flow escapes the Lee and flows across the floodplain to the Curraheen. This process is NOT considered in these Waterworks Weir flow estimates, but it is considered by the detailed hydraulic modelling. In the hydraulic modelling the focus for the HEP approach is in the contained design case. In the existing case the HEP check is from the routing model described above. However, differences in routing techniques means that an exact match is not possible, but an adjustment of lateral inflows is not justified in this situation.

At the 100-year return period, the flow at Waterworks Weir from the continuous simulation (921m^3/s) is 2% lower than the calculated flow based on conventional statistical analysis (See Appendix B) of (944m^3/s). The difference is greater (-11%) at the 20-year return period but then diminishes again at lower return periods. These differences are considered small enough to be accepted without further adjustment of the continuous simulation results. It gives us confidence that when the reservoirs are re-introduced to the model, the peak flows entering the system will be compatible with the flow estimates derived in Appendix B of this report.

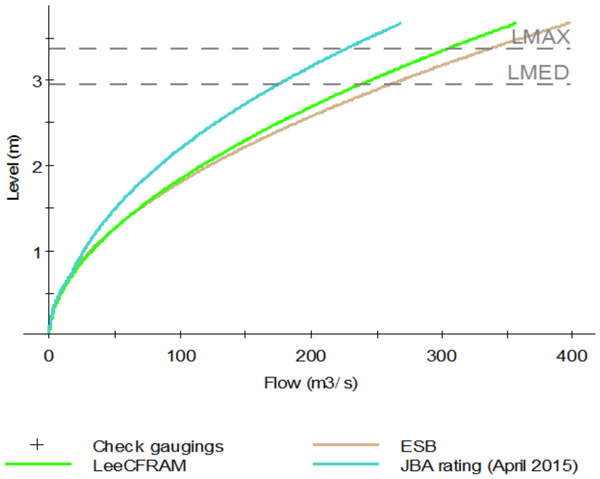
Figure 8-1 also shows that the event set at Waterworks Weir looks sensible and contains a number of multi-peaked events - as might be expected in reality. This set of design flows has therefore been accepted as the inflows to the catchment model for the 'Baseline' and various 'Design' scenarios to be investigated by the design team.

Figure 8-1: 870 simulated stochastic flow hydrographs at Waterworks Weir for the No Reservoir simulation



Differences between the continuous simulation and design flows are more significant at some locations in the network. Although these locations are of less interest than Waterworks Weir, some have observed data available and discrepancies were investigated. Macroom is one of the largest inflows to Carrigadrohid reservoir. The 100-year flow from continuous simulation at that location is 24% lower than that predicted by the statistical method (Table 8-2). When developing the forecasting model, we found that the flows at Macroom were not consistent with the total flow entering Carrigadrohid (as calculated by ESB). Our rainfall runoff modelling suggested the observed flows should be lower. A new rating curve was developed using results from rainfall runoff modelling to make the system work reasonably - without it, a large rainfall scaling factor was needed to get a good match. The new rating predicts a flow 25% lower than that indicated by the CFRAM rating around QMED (Figure 8-2). This difference is very similar to the discrepancy in design flows. Furthermore, the design flows for Dromcarra - Macroom's large neighbour, are well matched to the statistical estimates (within 3% at the 100-year flow). Thus, we suspect that the rating, used in the QMED calculation for Macroom, is suspect and that the flows from the CS are more reliable. Outlined in Appendix B is a review of Macroom rating curve developed during the LEE CFRAM. A large degree of uncertainty was found with changes in staff location, lack of flow gaugings and failure to model the Laney tributary that joins 100m upstream.

Figure 8-2: New and existing rating curves for Macroom



Note that the LMED and LMAX values in this plot is based on the very short available period of observed data and are unreliable.

Further support for using the continuously simulated values comes from them being based on calibrated PDM models, with a physically based catchment extent, fed with stochastic rainfall checked against the FSU rainfall statistics and have not been adjusted retrospectively. When routed together, through the catchment model, they align well with the estimates for Cork itself.

There are many potential reasons why gauged catchment flows at sub-catchments might differ from the continuously simulated results - even if the PDM models and design flows are themselves correct.

Our stochastic rainfall series matches the long term depth duration and frequency statistics best at durations of around 24 to 48 hours and return periods of around 100-year. Several catchments have a critical storm duration that is significantly shorter than this, making them sensitive to the rainfall depths that our stochastic rainfall series is known to under represent. Peak flows for flashy catchments will therefore tend to be under predicted by our series. However, because we are interested in the design flows at Cork - which DOES have a long critical storm duration - these differences in sub catchments are acceptable. Individual flow estimates for the tributaries are included in the hydraulics report.

Another limitation of the lumped stochastic rainfall model is that the proportion of rain falling on any one catchment does not vary between (or during) events. We know that heavy rainfall in the upper catchment may be accompanied by much smaller rainfall totals in the lower catchment, but our simplifying assumption is that this proportion is fixed. This will tend to make high order events on the Lee, Shournagh and Bride more likely to coincide and, probably, increase the design flow for Cork. As a result we have ignore the small contribution coming from laterals in the Lee valley downstream of Inniscarra. Having said this, the flows from the two methods align well at Cork.

8.3 Baseline simulation results

Flows for the 'Baseline' scenario are needed to determine baseline economic damages. A baseline version of the model has been developed and tested (as described in the model proving section). Simulating the current operational regime of the reservoirs is more complex than the 'no reservoir' case. It requires account to be taken of inflows and release patterns on the run up to an event as well as the prescribed operating rules during the rising flood.

The parameters used in the Baseline (or 'As Now') model were agreed with the project steering group and are listed in Table 7-5. The main differences to the 'Design' scenario are:

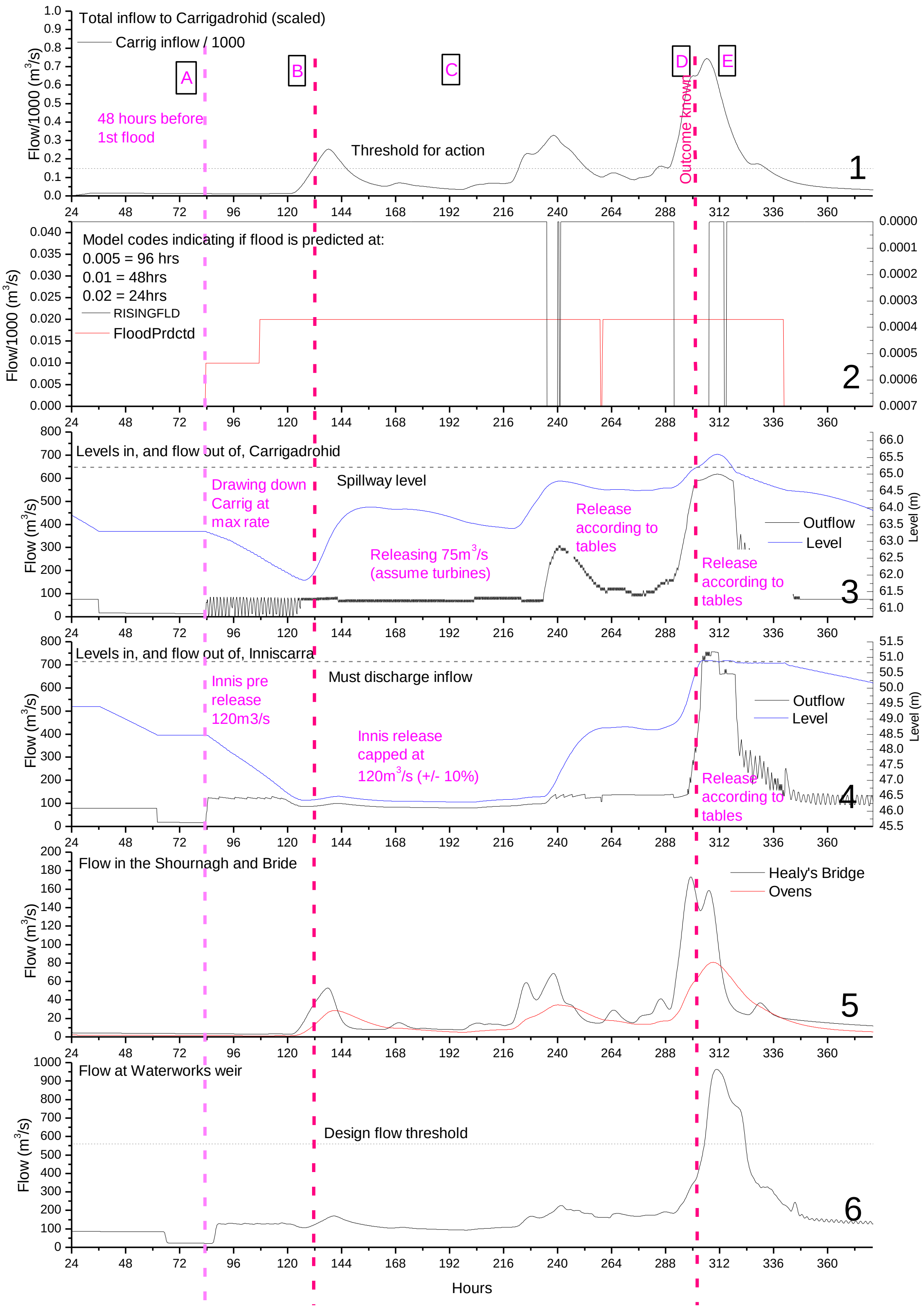
- Lower MNOL for both reservoirs to reflect their winter operation;
- A reduced maximum allowable discharge rate from Inniscarra (120m^3/s);
- Smaller pre-releases (120m^3/s) in anticipation of a large event, with pre-releases only commencing 48 hours before a threshold crossing;

- A lower threshold for making pre-releases (150m^3/s), still based on the total inflow to Carriadrohid;

The same model is used in the Baseline and Design scenarios. The only differences are the parameters used within it (as per Table 7-5). Its behaviour is illustrated and described for the same event as Figure 8-3 below.

1. Total inflow to Carrigadrohid is shown in the first plot. This is exactly the same as the design simulation in Figure 7-12. Only the threshold for action is different: now 150m^3/s rather than 400m^3/s. There are three crossings for this threshold, the first being at 223hrs (B). This triggers drawdown in the model at 48 hours out (A). The proximity of the events to one another means that, once the first has crossed the threshold, the 'all clear' is not signalled until after the largest event has passed.
2. As a result, the second plot shows that the simulation is in a state of 'flood predicted' for most of the run.
3. Releases are made from Carrigadrohid at a rate of 75m^3/s, while not exceeding the maximum drawdown rate from point A. 75m^3/s is then sustained for the model run until the rules dictate that releases should increase at 240 hours. Towards the last, and largest, event flows are again dictated by the rules and water levels rise above the spillway (300hrs at point D).
4. A persistent release of 120m^3/s from Inniscarra results in levels reaching 46.5m before the first event. This level is held by the continuing release until 234hrs, when Carrigadrohid releases > 75m^3/s are made. Levels then rise quickly, levelling off at 49m. The final, and largest, event pushes levels into the 'release inflow' zone and as a result, flows in Cork exceed 900m^3/s (Plot 6) when combined with lateral inflow from the Bride and Shournagh (Plot 5).

Figure 8-3: Example double peaked (exceedance) event from continuous simulation to illustrate the Baseline scenario



8.4 Design simulation results

Parameters of the design simulation are tabulated in Table 7-5 and the behaviour of the model in a simulation was described in Section 7.6.7. Results from a continuous simulation for the 'Design' scenario are compared to the 'No Reservoirs' and 'Baseline' scenarios in Figure 8-4 and tabulated in Table 8-3 (below).

Simulations without the reservoirs, and therefore without storage or attenuation, give the highest flow at all return periods. In this scenario, the 100-year flow is 921 m^3/s.

The Baseline model gives the lowest flows at low return periods (up to around the 25-year event). Baseline maximum releases are capped at 120m^3/s. When MNOL is exceeded in the reservoirs on the rising flood, releases must be made according to the dam safety rules. Some storage is created 48 hours in advance of an event in this scenario and MNOL is set lower than the first point in the dam safety tables. Flows rise steeply after the 25-year return period to approach the 'No Reservoir' flow at the 100-year return period (861m^3/s).

Design flows are much higher than the Baseline at low return periods for two reasons:

- Pre-releases to create storage can be as high as 300m^3/s; and
- Flood storage is only created when the inflow to Carrigadrohid exceeds 400m^3/s. Flows less than this pass through the reservoirs starting at MNOL and are dealt with by the dam safety rules.

In reality, the reservoirs are very unlikely to be at MONL because uncertainty in flow forecasts means drawdown happens for most large events. The flood frequency curve for the design case is therefore conservatively high at the lower end. The design frequency curve aims to keep flows in Cork around 550m^3/s (100-year flow is 555m^3/s). This is only possible while there is storage available in the reservoirs. When this is exhausted, as in higher return period events, flows increase rapidly. This is an important consideration for scheme safety in relation to exceedance events.

Figure 8-4: Flood frequency curves at Waterworks Weir for the three scenarios

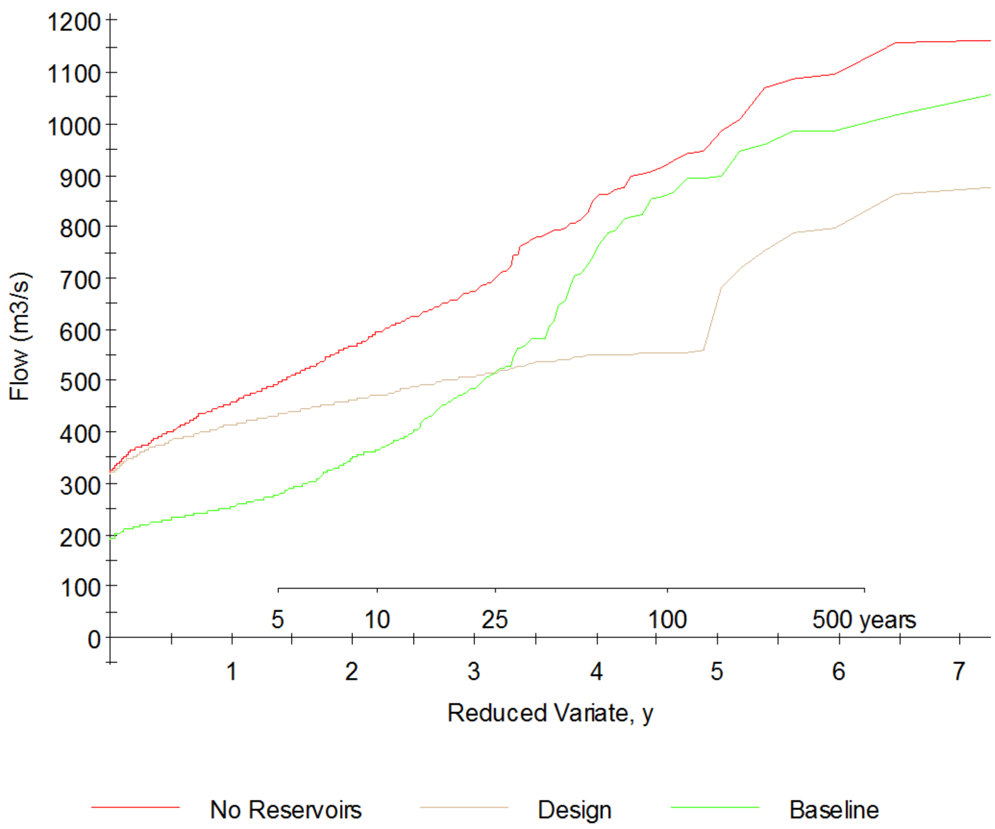


Table 8-3: Peak flows at Waterworks Weir for three scenarios and a range of return periods

	Flow (m^3/s) for return period (years)								
Continuous simulation									
Scenario	**2**	**5**	**10**	**20**	**30**	**50**	**100**	**150**	**200**
No reservoirs	386	510	599	674	765	820	921	953	1039
Design	372	440	472	507	530	547	555	575	734
Baseline	234	284	351	452	536	705	861	892	906

8.5 Climate change simulations

This has been updated and included in the Lower Lee Options Report.

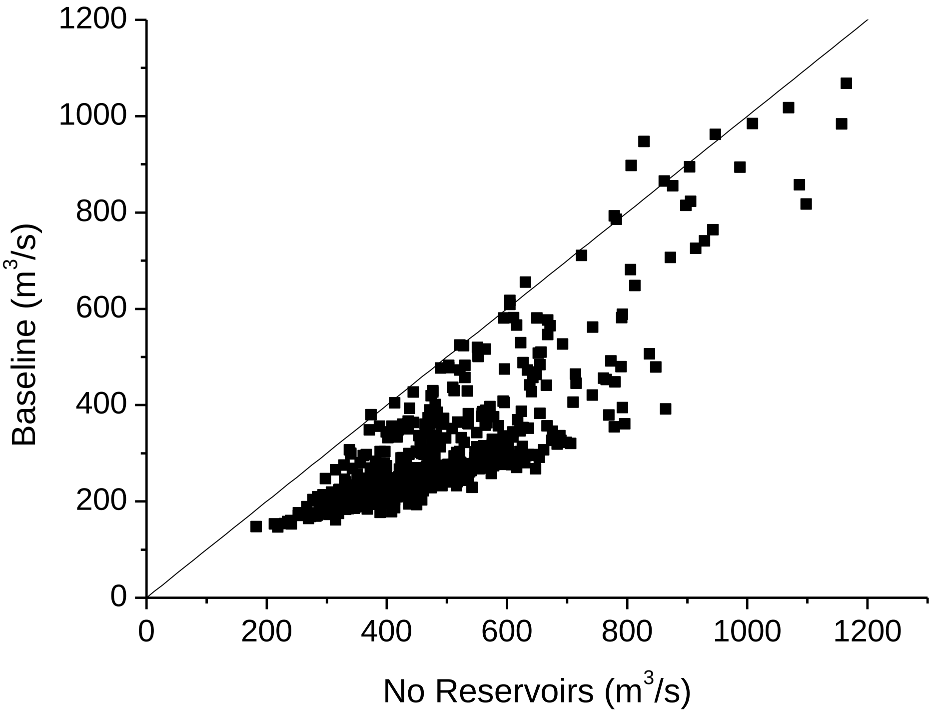
8.6 Context of the November 2009 event

November 2009 is the largest observed event on the Lee at Cork in living memory - and probably since the reservoirs were constructed. The context of this flood in terms of the continuous simulation is therefore highly relevant and is considered here. Although the exact peak flow into the Cork reach during the event is not known (flows at Healy's Bridge or Waterworks Weir are not available for that event), we are able to simulate it using the 'Baseline' catchment model. Predicted flows at Carrigadrohid and Inniscarra are very close to the observed values for this event (see Figure 7-10 and Figure 7-11), giving us some confidence in the accuracy of the Upper Lee part of the model. Observed continuous flows from ESB in the Bride and Shournagh are not available, but are simulated by the model. An estimated peak flow is available at Healys Bridge. Our catchment model predicts a peak flow of 678m^3/s at Waterworks Weir in November 2009, which equates to a 46-year peak flow in our continuous simulation using the 'Baseline' model (with the assumptions set out above - but the return period is sensitive to these). Running the same inflows through the 'no reservoirs' model gave a flow of 712m^3/s which has a return period of around 25-years. The event is therefore rarer with the reservoirs in place than without.

This result is due to the multi peaked nature of the November 2009 event. Figure 7-10 and Figure 7-11 show how there was a significant 'mini event' only hours before the main event struck. Its effect was to raise water levels in both reservoirs, reducing storage and bringing forward the time at which discharges from the reservoirs would be governed by the operational discharge tables. These tables happened to prescribe releases, for given levels, at roughly the same rate as water entered the reservoirs (as it was a gradually rising flood), meaning the attenuation provided was small.

We checked the continuous simulation for similar events - where the flow at Waterworks weir is similar for the 'Baseline' and 'no reservoir' cases. Figure 8-5 shows the peak flow from all 870 'no reservoir' simulations, correlated with peak flows from the same events for the 'Baseline' simulations. The great majority of the stochastic flows are reduced by the reservoirs (points sitting below the 1:1 line), but there are some for which there is little or no attenuation (falling on, or even above, the 1:1 line). These events are similar to November 2009, having 'pre events' which raise reservoir levels, leaving little storage for attenuation of the main peak. These finding show the importance of representing the probability of multi-peaked, as well as single peaked, flood events in the design flow calculation procedure.

Figure 8-5: Simulated peak flow at Waterworks weir correlated for No Reservoirs and Baseline scenarios



8.6.1 Sensitivity testing the Baseline scenario

Previous testing of the Baseline model to different starting water levels, turbine availability and spill rates yielded variations of less than 10% in the 100-year flow. This is because, at higher return periods, the reservoirs fill and the dam safety rules take over. High order return period flows are therefore not particularly sensitive to parameter choices

9 Design flows for ungauged catchments

9.1 Calculation of a Qmed catchment adjustment factor

In ungauged catchments it is necessary to revert to statistical methods to calculate an estimation of Qmed in the catchment. For this study, the FSU regression equation has been chosen. It is possible to improve on the initial estimate of Qmed by refining it using the process of data transfer, in which a representative gauged catchment with suitable quality data is identified and an adjustment factor for Qmed calculated as the ratio of the gauged to the ungauged estimate of Qmed at the gauging station. This factor is then used to adjust the initial estimate of Qmed at an ungauged site or gauging site with poor data records, under the assumption that the factorial error in the Qmed regression model is similar for two catchments. In the terminology of the FSU research reports, the gauging station where the adjustment factor is calculated is referred to as a donor site.

Table 9-1 shows the results from the different Qmed estimation techniques. The gauges have been classified according to catchment type.

Table 9-1: Summary of Qmed in Gauged Catchments

	FSU	FSR RR Winter	FSR RR Summer	Lee Cfram	Single Site Average	Adjustment Factor (Single Site/FSU)
Lake Influence						
Lee Dromcarra	81.228	76.96	80.08	80.22	81.51	1.00
Karst Influence						
Ovens	21.701	39.57	40.65	29.5	26.63	1.23
Tower	33.819	56.99	58.5	70.2	70.14	2.07
Excluded Catchments						
Macroom (Sullane)	80.225	76.96	102.17	141.7	148	1.84
Standard Catchment						
Kill (Laney)	30.061	44.36	45.99		50.17	1.67
Dripsey	20.137	35.48	36.49		40.96	2.03
Healy's Bridge	40.848	66.33	68.26	70.5	62.64	1.53
						1.75

The gauges have been classified according to catchment type.

(a) Lee Dromcarra which is influenced by Lough Allua

Lee Dromcarra is influenced by the Lough Allua and this has the effect of lowering the adjustment factor as the recorded Qmed is lower due to the effect of lake attenuating the flows.

(b) Catchments that are potentially influenced but Karst geology

When an annual maximum series plot of the recorded record at Ovens is analysed it was found that the karst influence attenuates the peak. At a certain point the groundwater influence is overcome and its flow values rise rapidly in more extreme events. This is different to the expected normal distribution of an annual maximum series in Ireland.

Tower gauge is also affected by a karst influence. At present discrepancies exist between Tower gauge and Healy's Bridge gauge, with Tower, a subcatchment of Healy's Bridge registering higher flow for the same event at Tower than Healy's Bridge. Healy's bridge has been calibrated using an estimated flow for the November 2009 event derived by the EPA.. There are a number of issues with the Tower gauge including its location upstream of the bridge with the effects of the bridge difficult to model and a lack of high flow gaugings. In the location of the Tower gauge there are large floodplains that once inundated lead to a small rise in levels but a large rise in flows resulting in a rating that is very sensitivity to small changes in level. As a result Healy's Bridge gauge data has been included in the analysis and Tower has been excluded, although it should be noted that the continuous hydrograph data for 2009 is not available and the peak flow was estimated by EPA..

(c) Excluded Catchments

A large degree of uncertainty remains at Macoom and has therefore has been excluded from the analysis. The limited data record, change in gauge location, the exclusion of the River Laney that joins the Sullane just upstream of the Macroom gauge in the development rating curve for the Macroom and a lack of flow gaugings has led to its exclusion from the analysis.

(d) Standard tributaries

At the remaining catchments (Kill, Dripsey and Healy's Bridge) a Qmed adjustment factor was found to average 1.75 as shown in Table 9-1. These three remaining stations were then weighted according to their record length to give a weighted catchment adjustment factor of 1.71 as shown in Table 9-2and this will be carried forward and applied to ungauged catchments further downstream and gauges with poor data records. Full details of the individual statistical techniques are included in Appendix B.

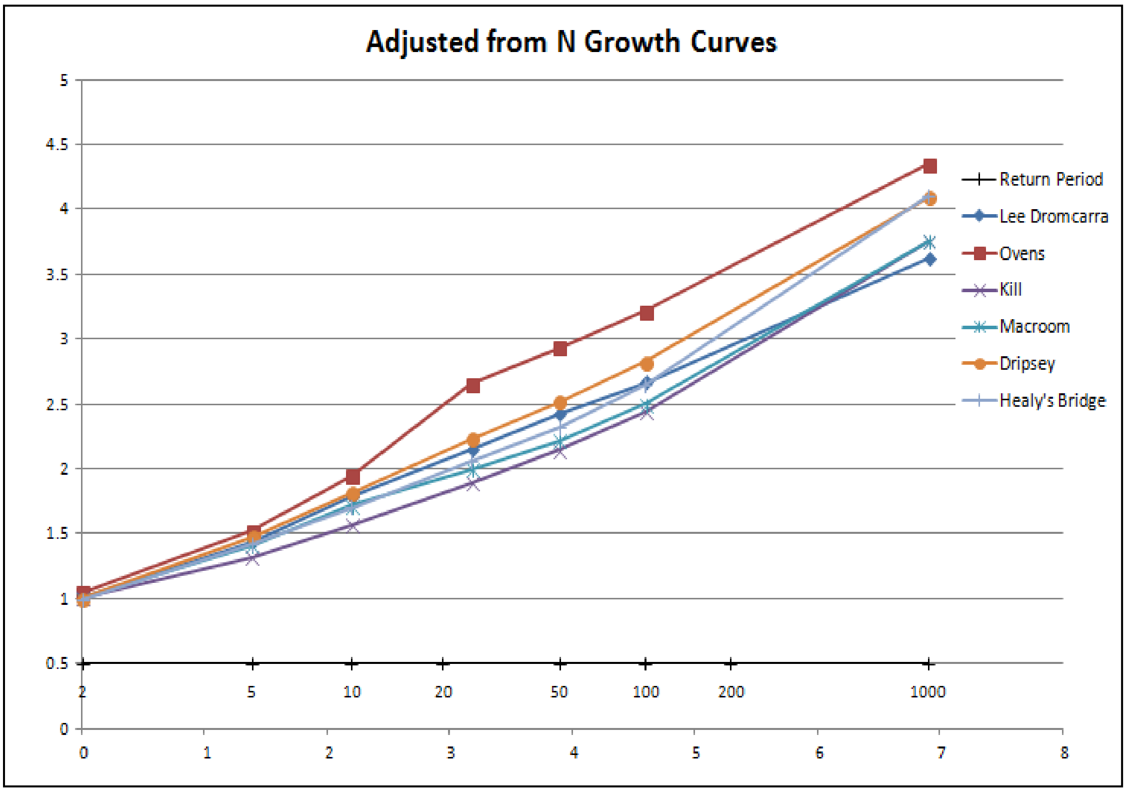
Table 9-2: Weighted adjustment factor

Weighted Average				
	Years of Data	Weight	Adjustment Factor	
Kill (Laney)	24	0.34	1.67	0.57
Dripsey	19	0.27	2.03	0.55
Healy's Bridge	27	0.39	1.53	0.59
	70	Catchment Adj. Factor		1.71

9.2 Calculation of the catchment flood frequency curve

It is needed to calculate a flood frequency curve that will be applied for ungauged catchments. Gauged flow estimates have been developed for each of the gauged inflow stations and is discussed at length in Appendix B and the results are shown below in Figure 9-1.

Figure 9-1: Gauged Site Growth Curves



As all gauged sites, except Ovens the flood frequency plot similar results as shown in Figure 9-1, and hence these were averaged to obtain a catchment flood frequency curve as shown in Table 9-3. Macroom has once again been removed from the analysis.

Table 9-3: Catchment Flood Frequency Curve

Return Period (Yr)	Lee Dromcarra	Healy's Bridge	Kill	Dripsey	Average
2	1.00	1.00	1.00	1.00	**1.00**
5	1.44	1.42	1.32	1.47	**1.40**
10	1.78	1.70	1.56	1.81	**1.68**
25	2.15	2.06	1.89	2.23	**2.05**
50	2.42	2.32	2.14	2.51	**2.31**
100	2.66	2.64	2.43	2.82	**2.62**
1000	3.62	4.11	3.75	4.09	**3.98**

9.3 Design Flows for Curraheen and Glasheen

Downstream of Waterworks Weir the Curraheen and Glasheen join the south channel of the River Lee. These are ungauged catchments and ungauged catchment methodology of calculating Qmed using the FSU regression equation, multiplying it by the catchment adjustment factor of 1.71, as was applied in Blackpool, and applying the catchment flood frequency curve as outlined in Table 9-3 gives the design flows shown in Table 9-4. It is recommended that a gauging station is established on the Curraheen in order to establish a revised adjustment factor for this catchment once it becomes an independent watercourse to the Lower Lee.

Table 9-4: Design Flows for Curraheen and Glasheen

Watercourse	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	1000-Yr
Glasheen	4.24	5.94	7.12	8.69	9.79	11.11	16.88
Curraheen	18.04	25.26	30.31	36.98	41.67	47.26	71.80

10 Conclusions

Design flows in the Lee catchment have been calculated for Cork using continuous simulation. This method is much more robust than traditional techniques because of its ability to take account of events of different volume, magnitude and antecedent conditions within a probabilistic framework (ultimately being based on rainfall statistics). The method allows comparison of different reservoir operation regimes and flows without the reservoirs in place.

A stochastic rainfall series has been synthesised for the Macroom catchment using a statistical rainfall model calibrated on observed data. The series has been post processed so that the depth, duration and frequency of rainfall matches FSU statistics as closely as possible. Rainfall for other catchments is scaled directly from this series according to SAAR ratios.

Rainfall runoff processes, river routing and, crucially, reservoir operation are encapsulated within a catchment model. It consists of 12 PDM models and an ISIS river routing model. All components are calibrated to observed data. Further improvement in this model will take place over time, as greater granularity in the rainfall network is achieved, gauging station ratings are improved and a new gauge installed at Waterworks Weir, which in the scheme situation will capture all the flow.

The 1,000-year synthetic rainfall series has been applied to the catchment model to obtain design flows for Cork. The 'No Reservoir' flow is within 2% of that calculated using traditional FSU statistical methods. Versions of the model have been developed and run for the Baseline and Design scenarios and peak flows obtained for the city centre (reported in Table 8-3).

Hydrometric data was reviewed and quality checked (Appendix A) and Qmed adjustment factors and flood frequency curves was developed for each of the individual hydrometric gauges in the Lee catchment. This allowed the validation of design flows calculated using continuous simulation but also allowed the development of a catchment based flood Qmed adjustment factor and flood frequency curve that can be used in ungauged catchments (detailed in Section 9). A Qmed adjustment factor of 1.71 and a growth factor of 2.62 was found.

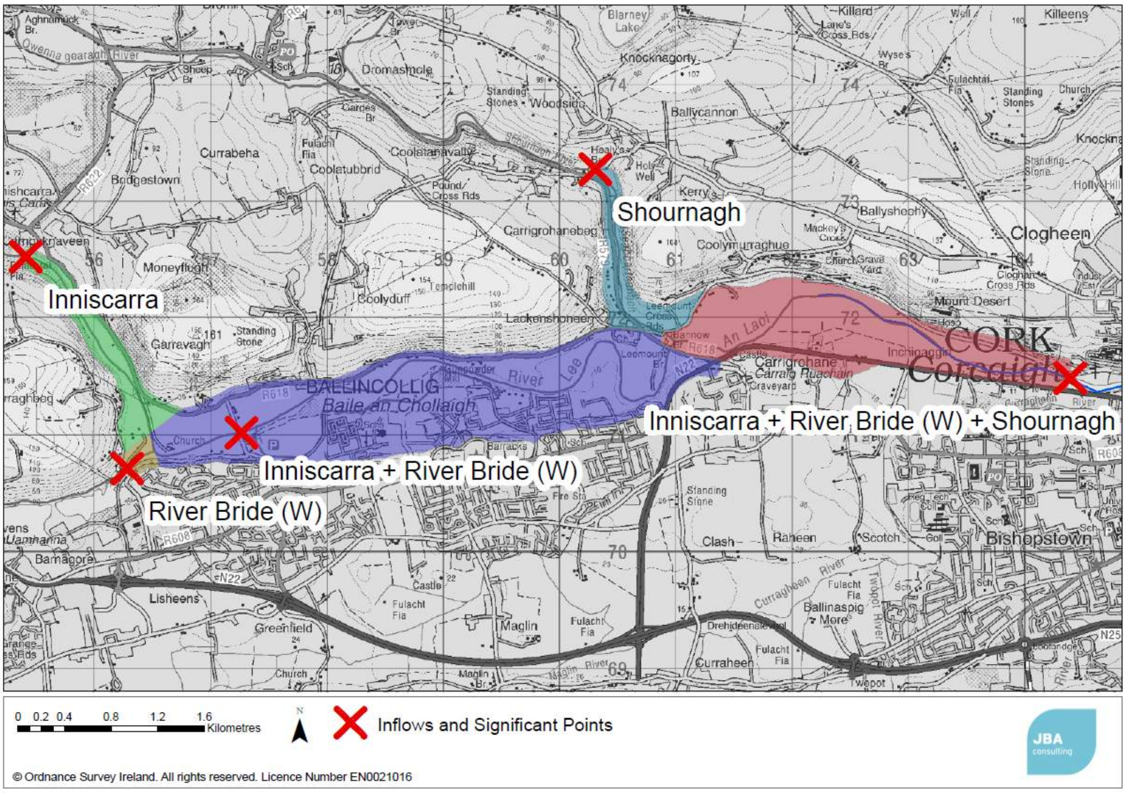
The use of continuous simulation has been grounded against the single site analysis that can be undertaken with the available data. From this continuous simulation, a suite of hydrographs for each catchment was derived for a range of events using the existing dam operation rules. Post scheme, or design event, river flows were produced following generation of revised reservoir operation rules and optimising the dam performance with flood risk management in mind. The entire continuous simulation produced a 1000-year record of events which were ranked in terms of magnitude by comparing the flow value at Waterworks weir. The continuous simulation provides the timing and magnitude for each catchment which then all combine and contribute to a design flow value. The design flows at Waterworks Weir from the continuous simulation analysis are given in Table 10-1 below.

Table 10-1: Design Flows at Waterworks Weir

	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	1000-Yr
As-Now (Baseline)	372	440	472	507	530	547	555
Design Scenario	234	284	351	452	536	705	861

Similarly, for other reaches of the catchment the continuous simulation 1000-year were ranked in terms of magnitude at a number of critical flow estimation points as show in Figure 10-1 to provide design flows for the hydraulic modelling stage for the reaches also shown.

Figure 10-1: Critical Reaches and Design Flow Estimation Points



The Curragheen and Glasheen catchment were not included in the Continuous Simulation model and design flows are based on the FSU as discussed in Section 9. Full application of hydrology is discussed in Section 2 of the hydraulics modelling report.

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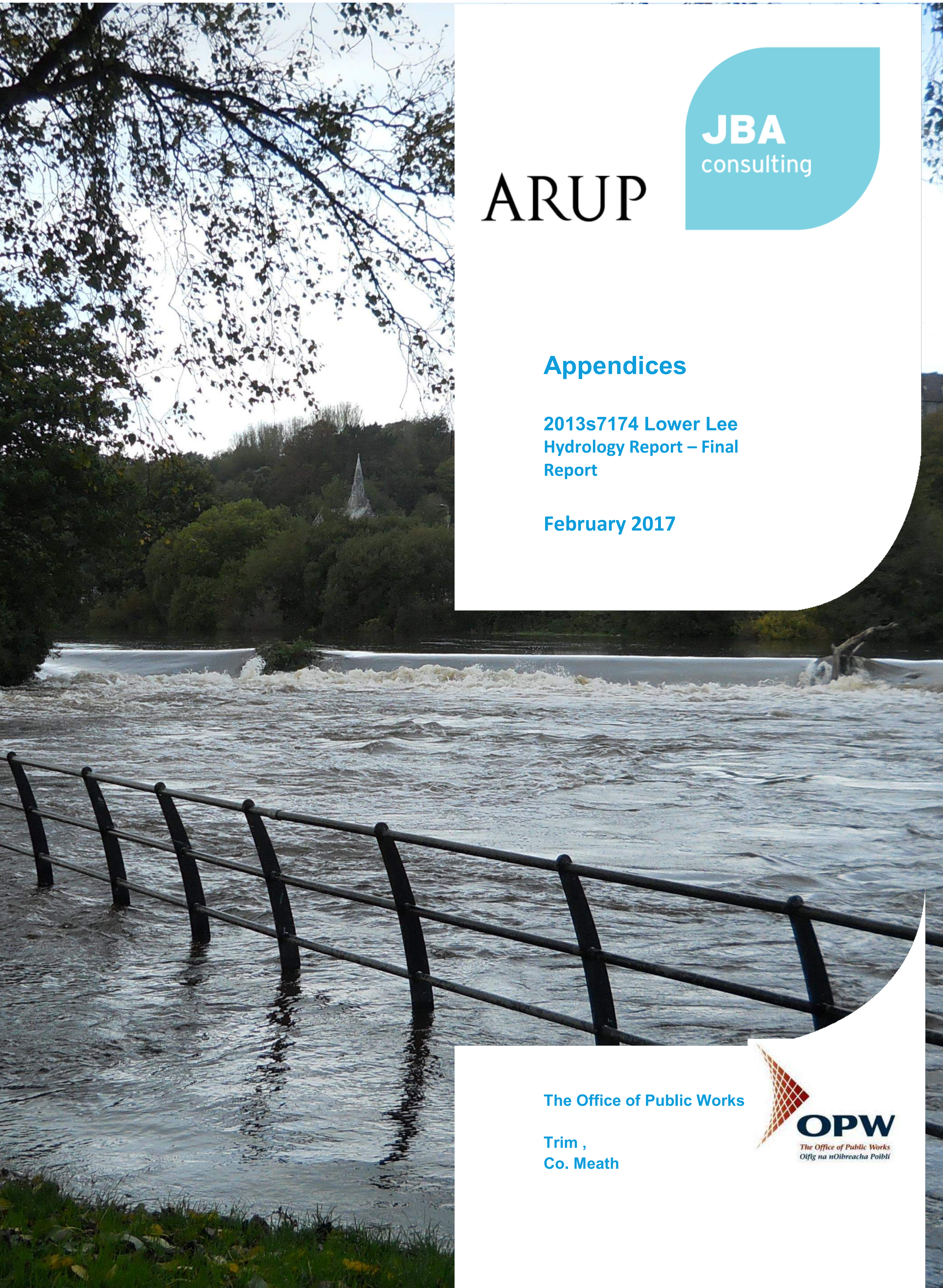

Appendices

2013s7174 Lower Lee Hydrology Report – Final Report

February 2017

The Office of Public Works

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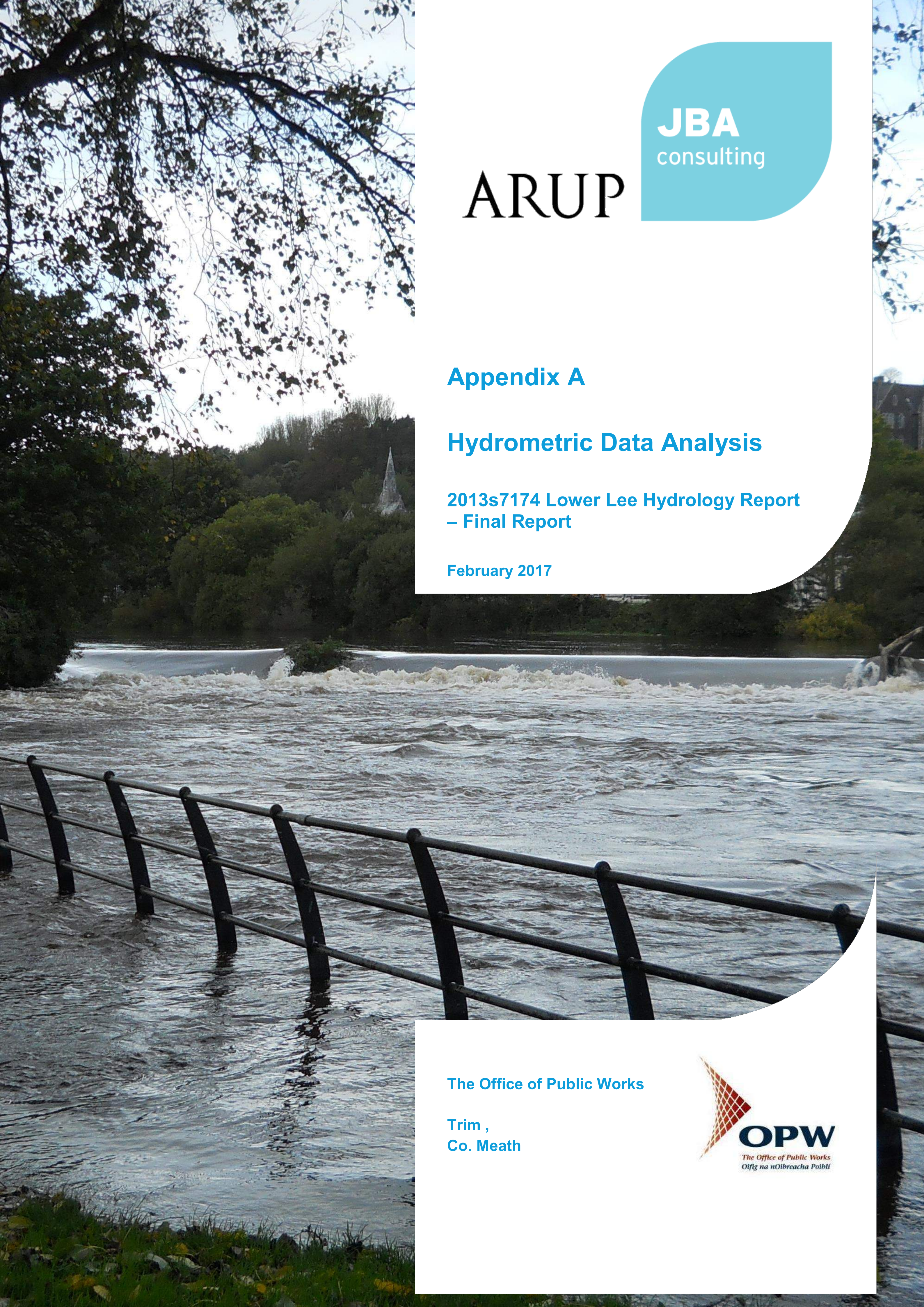
Co. Meath



Appendix A

Hydrometric Data Analysis

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A Hydrometric Data Analysis

As part of this study, 11 hydrometric gauges were analysed as highlighted in Figure A-1 below. Table A-1 details the data availability at each of the Hydrometric Gauges.

Figure A-1: Catchment Map

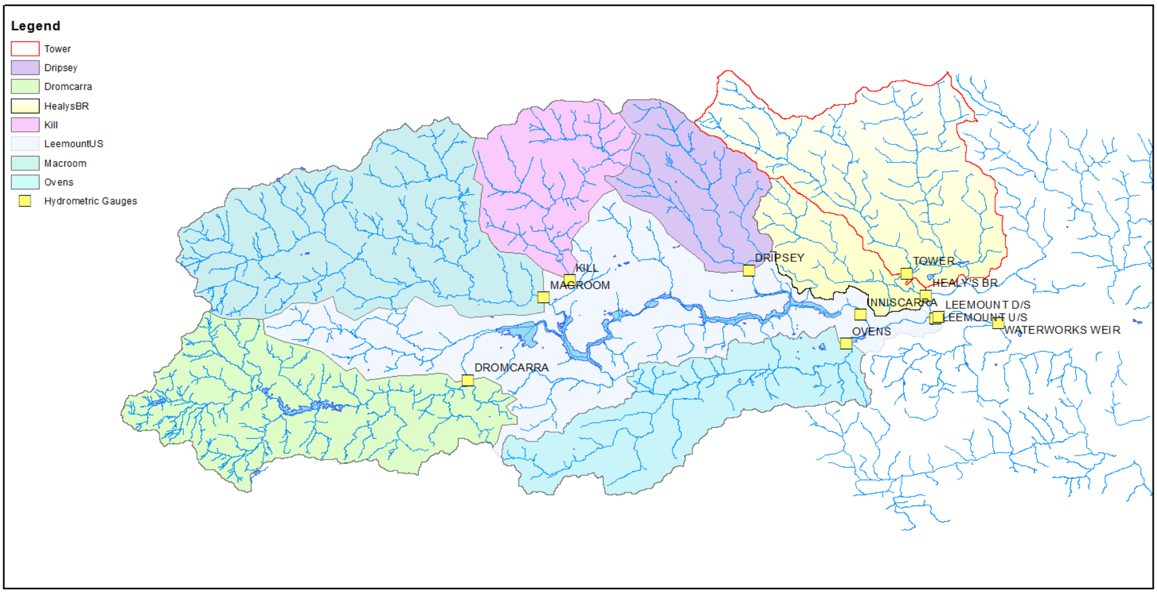
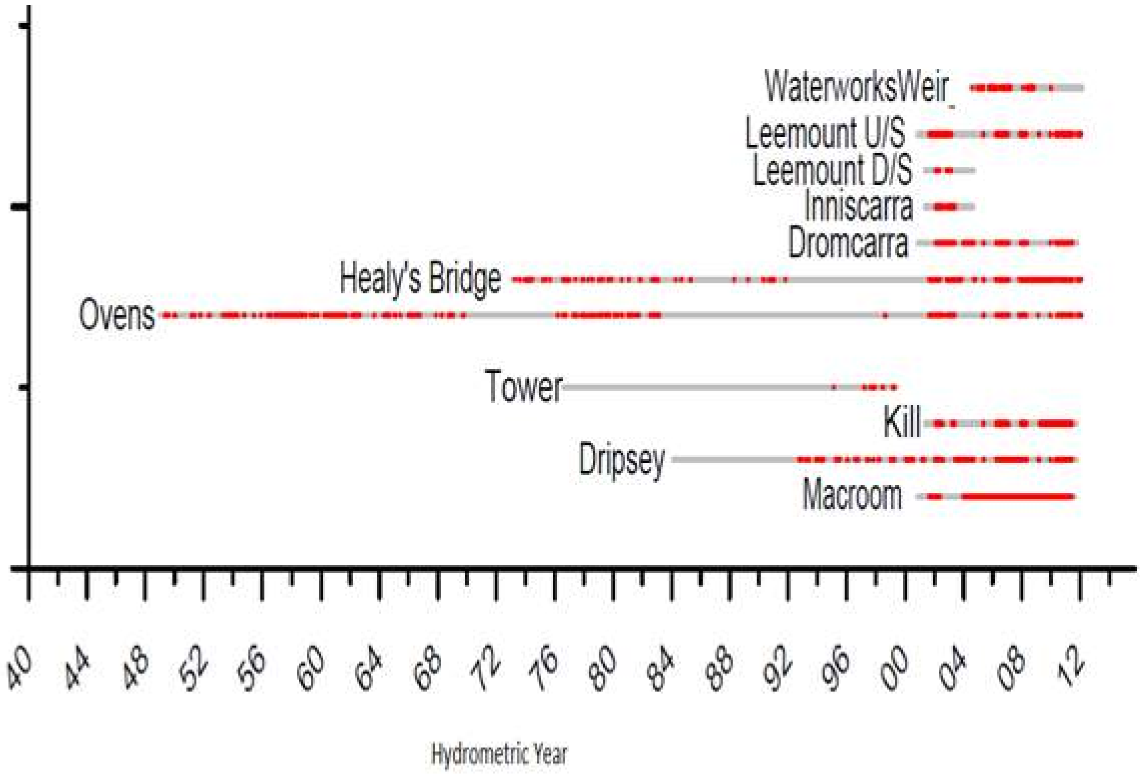


Table A-1: Hydrometric Station Data Availability

Station Number	Station Name	Amax Data	15min Water Level
19011	Leemount Upstream	1950-1999	2000-2012
19012	Leemount Downstream	1956-1993	2000-2004
19013	Inniscarra Tail Race	1942-2000	2001-2004
19014	Lee Dromcarra	1949-1995	2000-2012
19015	Healy's Bridge		1973-2012
19016	Ovens		1949-2011
19018	Tower		1976-1999
19027	Kill	1984-2001	2002-2011
19028	Dripsey		1984-2011
19031	Macroom	1982-1990	2000-2011
	Waterworks		2002-2009

It can be seen there is 15min water level data available at all the stations. Figure A-2 below indicates the data gaps that exist in this 15min water level data, with the red indicating no data recorded.

Figure A-2: 15 Minute Water Level Data Availability



For each of the hydrometric stations, records were manually reviewed for data gaps and consistency to nearby gauges. Where gaps existed, all nearby hydrometric gauges were reviewed, to ascertain whether the gap may have missed the annual maximum event. For each gauging station the following was completed:

- Comparison of data records with known historical events in the Lee Catchment.
- Comparison of event time with nearby stations to see if it aligned with Amax recording at nearby gauging stations.
- Checking the extent of missing data at the gauging station, if any, in any given hydrometric year. Where there is no alignment in event timing with adjacent stations both stations was analysed to help verify the capturing of the peak flow in a hydrometric year.
- A simulated record has been produced from rainfall runoff modelling using the PDM models across the catchment. Event data was produced for the period 2002 to 2013. This was compared against actual record to ascertain whether the gap in the hydrometric data may have missed the annual maximum event. It also allowed a check on what scale of event could have been expected.

Where the gap(s) in a given year was deemed to be inconsequential the gauge hydrometric year was accepted. If the gap was deemed to potentially contain an annual maximum event, the gauge hydrological year data was omitted from the analysis

Analysis of flood peak at the gauging stations is recorded in individual gauge data sheets at the end of this appendix and is summarised in Figure A-3, Figure A-4 and Table A-2.

Figure A-3: Comparison between Total and Useable Data

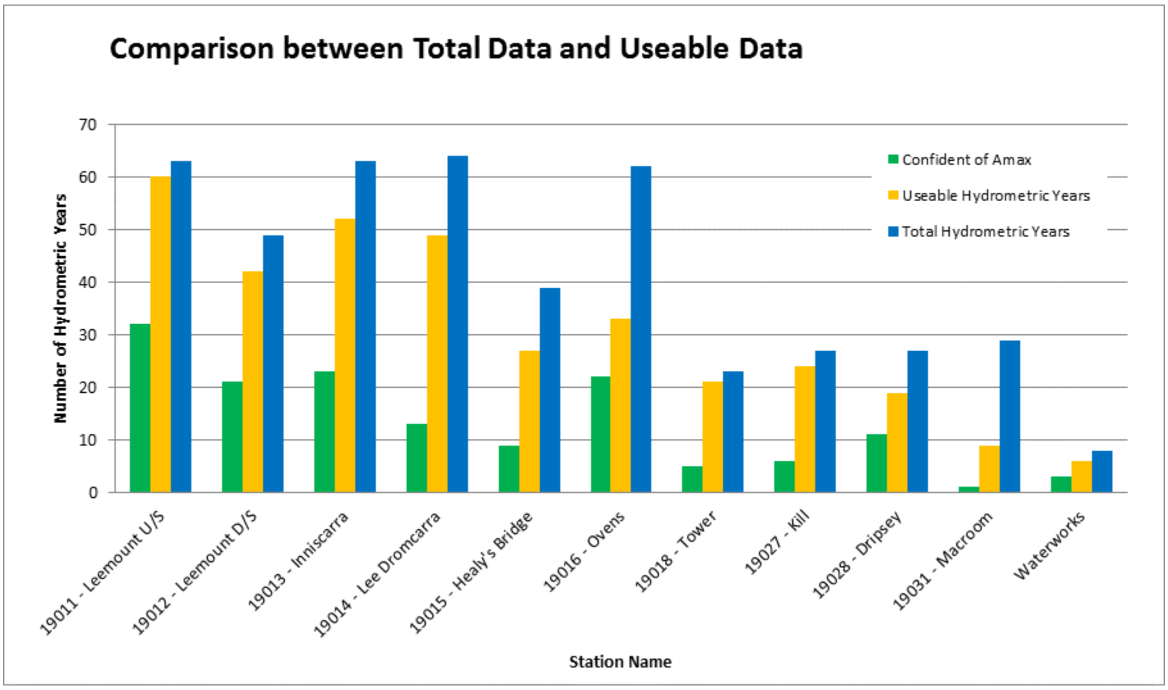
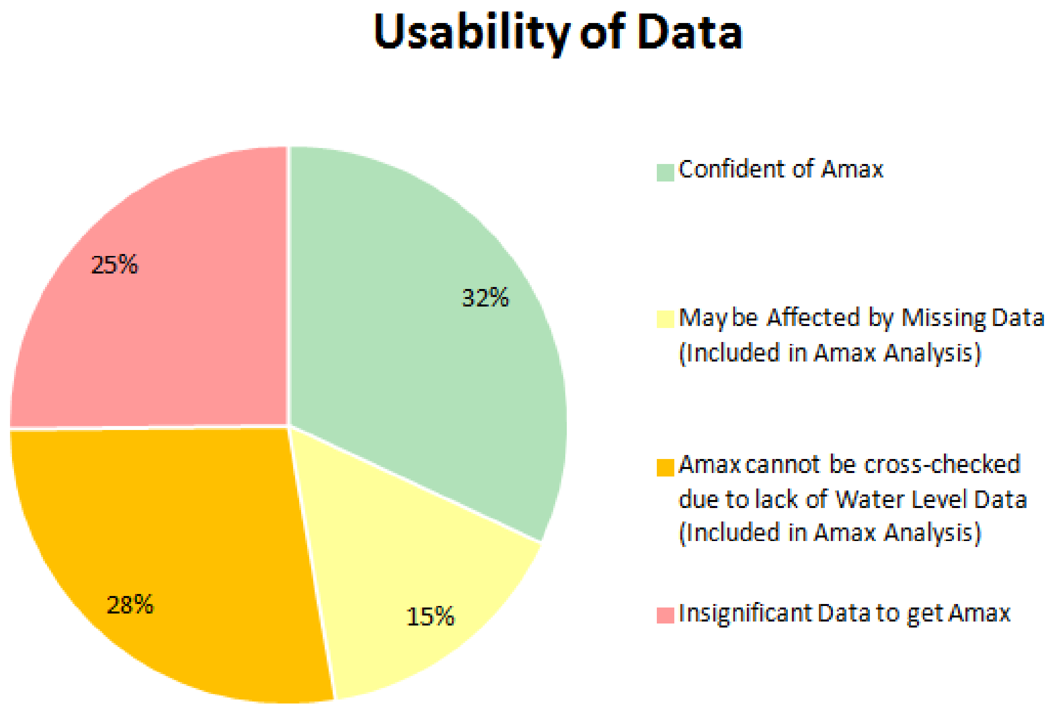


Table A-2: Summary of Results of Data Quality Check

Station	Total	Confident of Amax	May be Affected by Missing Data (Included in Amax Analysis)	Amax cannot be cross-checked due to lack of Water Level Data (Included in Amax Analysis)	Insignificant Data to get Amax	Useable Data
19011 - Leemount U/S	63	32	5	23	3	60
19012 - Leemount D/S	49	21	2	19	7	42
19013 - Inniscarra	63	23	-	29	11	52
19014 - Lee Dromcarra	64	13	2	34	15	49
19015 - Healy's Bridge	39	9	18		12	27
19016 - Ovens	62	22	11		32	33
19018 - Tower	23	5	16		2	21
19027 - Kill	27	6	1	17	3	24
19028 - Dripsey	27	11	8		8	19
19031 - Macroom	29	1	4	4	20	9
Waterworks	8	3	3		2	6
Total	454	146	70	126	115	342

Figure A-4: Summary of Usability of Data



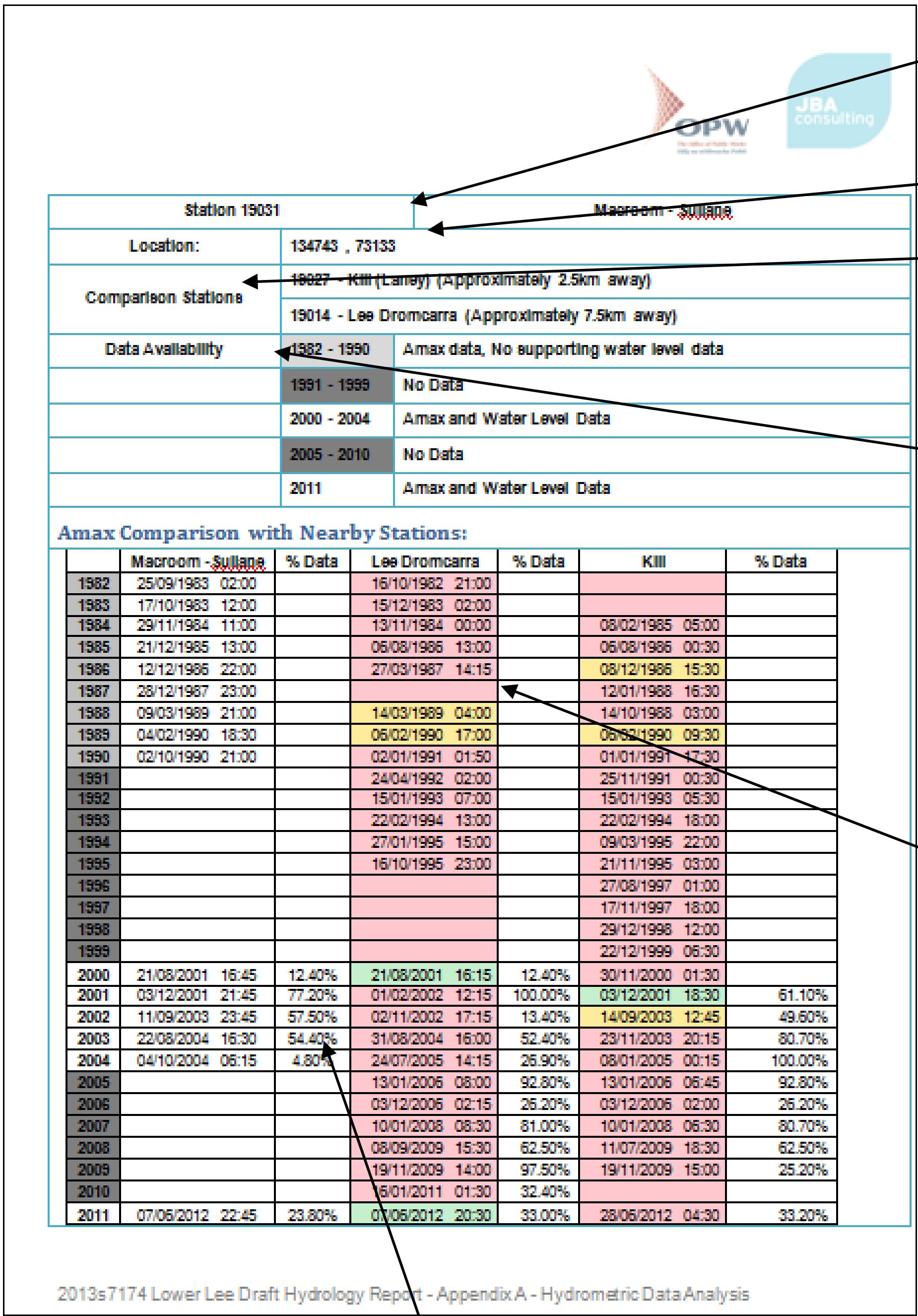
In summary:

- 454 years from 11 gauging stations were analysed.
- Only in 146 years (32%) could the capturing of the actual annual maximum be confidently assumed.
- 15% of the years data records were found to contain small data gaps leading to the possibility that the annual maximum for the hydrometric year may have been missed.
- 28% of the data was determined from annual maximums recorded from chart records, so therefore had no supporting water level data available to verify the record.
- Significant gaps were found in 25% of the data record and it was deemed unusable.

The analysis lead to a useable record of 75% of the data. The limitations in data availability can be clearly seen. By including all data in a usable record besides the years that have been highlighted red and shown to have insignificant data, it will reduce the certainty of recorders having captured the correct Amax for a hydrometric year however, it should improve the calculations of the Qmed by using a longer record for each of the stations.

It is noted, that a number of significant observed events, i.e. likely greater that the validated median flow, may have occurred during years where there was large gaps in the yearly data as is the case for Macroom 19031. It is considered that these significant events cannot be included in the above valid annual maximum data, as they would provide a skew toward a higher median flow, where this process would not take account of the years where there was large gaps in data and the annual maximum was less that the validated median flow.

Information provided in the summary sheets



Station 19031		Macroom - Sullane
Location:	134743 , 73133	
Comparison Stations	19027 - Kill (Laney) (Approximately 2.5km away)	
	19014 - Lee Dromcarra (Approximately 7.5km away)	
Data Availability	1982 - 1990	Amax data, No supporting water level data
	1991 - 1999	No Data
	2000 - 2004	Amax and Water Level Data
	2005 - 2010	No Data
	2011	Amax and Water Level Data

Amax Comparison with Nearby Stations:

	Macroom - Sullane	% Data	Lee Dromcarra	% Data	Kill	% Data
1982	25/09/1983 02:00		16/10/1982 21:00			
1983	17/10/1983 12:00		15/12/1983 02:00			
1984	29/11/1984 11:00		13/11/1984 00:00		08/02/1985 05:00	
1985	21/12/1985 13:00		06/08/1986 13:00		06/08/1986 00:30	
1986	12/12/1986 22:00		27/03/1987 14:15		08/12/1986 15:30	
1987	28/12/1987 23:00				12/01/1988 16:30	
1988	09/03/1989 21:00		14/03/1989 04:00		14/10/1988 03:00	
1989	04/02/1990 18:30		06/02/1990 17:00		06/02/1990 09:30	
1990	02/10/1990 21:00		02/01/1991 01:50		01/01/1991 17:30	
1991			24/04/1992 02:00		25/11/1991 00:30	
1992			15/01/1993 07:00		15/01/1993 05:30	
1993			22/02/1994 13:00		22/02/1994 18:00	
1994			27/01/1995 15:00		09/03/1995 22:00	
1995			16/10/1995 23:00		21/11/1995 03:00	
1996					27/08/1997 01:00	
1997					17/11/1997 18:00	
1998					29/12/1998 12:00	
1999					22/12/1999 06:30	
2000	21/08/2001 16:45	12.40%	21/08/2001 16:15	12.40%	30/11/2000 01:30	
2001	03/12/2001 21:45	77.20%	01/02/2002 12:15	100.00%	03/12/2001 18:30	61.10%
2002	11/09/2003 23:45	57.50%	02/11/2002 17:15	13.40%	14/09/2003 12:45	49.60%
2003	22/08/2004 16:30	54.40%	31/08/2004 16:00	52.40%	23/11/2003 20:15	80.70%
2004	04/10/2004 06:15	4.80%	24/07/2005 14:15	26.90%	08/01/2005 00:15	100.00%
2005			13/01/2006 08:00	92.80%	13/01/2006 06:45	92.80%
2006			03/12/2006 02:15	26.20%	03/12/2006 02:00	26.20%
2007			10/01/2008 08:30	81.00%	10/01/2008 06:30	80.70%
2008			08/09/2009 15:30	62.50%	11/07/2009 18:30	62.50%
2009			19/11/2009 14:00	97.50%	19/11/2009 15:00	25.20%
2010			16/01/2011 01:30	32.40%		
2011	07/06/2012 22:45	23.80%	07/06/2012 20:30	33.00%	28/06/2012 04:30	33.20%

2013s7174 Lower Lee Draft Hydrology Report - Appendix A - Hydrometric Data Analysis

Hydrometric Station Name and Reference Number

Gauging Station Location

Comparison Stations

List the chosen nearby stations in which the timing of the Amax was compared against

Data Availability

Summarizes the data at the gauging station under consideration

Amax Comparison with Nearby Stations

Here the recorded Amax is compared with its nearby stations to see if the Amax occurred at the same point of time to help improve the understanding of the confidence in the timing of the Amax when a complete year of data is not available.

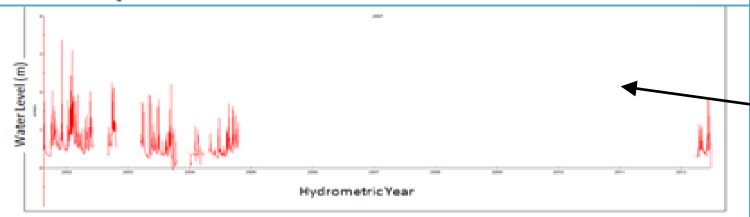
The Analysis is colour coded as follows:

	Same Amax Timing
	Amax significantly close
	Different Amax Timing

Percentage Data Availability

Percentage Water Level Data Available in a given year

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1982	25/09/1983 02:00	01/10/1982-23/06/1983	Only Summer Months Data
1983	17/10/1983 12:00		Appears Complete Record
1984	29/11/1984 11:00	21/11/1984-23/11/1984, 24/03/1985-07/04/1985	2 weeks in November, 2 weeks in March/April Missing
1985	21/12/1985 13:00	10/03/1986-24/03/1986, 09/08/1986-12/09/1986	Approximately six weeks of data missing
1986	12/12/1986 22:00	07/10/1986-20/11/1986	October- November Data Missing
1987	28/12/1987 23:00	29/09/1988-30/09/1988	Only one day missing
1988	09/03/1989 21:00	01/10/1988-10/10/1988	10 days missing in October
1989	04/02/1990 18:30		Appears complete record
1990	02/10/1990 21:00		Appears Complete record
1991			
1992			
1993			
1994			
1995			
1996			
1997			
1998			
1999			
2000	21/08/2001 16:45	01/10/2000-24/08/2001	One Month of Data
2001	03/12/2001 21:45	08/06-31/08/2002	No Summer Data, Known Flood Event
2002	21/08/2001 16:45	25/10/2002-23/03/2010	No Winter Months
2003	22/08/2004 16:30	17/10/2003-28/04/2004	Majority of Data missing from 17/10/2003 - 28/04/2004 (No Winter Months)
2004	04/10/2004 06:15		Only 16 days of Data
2005			
2006			
2007			
2008			
2009			
2010			
2011	07/06/2012 22:45	01/10/2011-04/04/2012, 27/06/2012-27/06/2012	Data only available from 04/04/2012 -27/06/2012

2013s7174 Lower Lee Draft Hydrology Report - Appendix A - Hydrometric Data Analysis

Data Availability

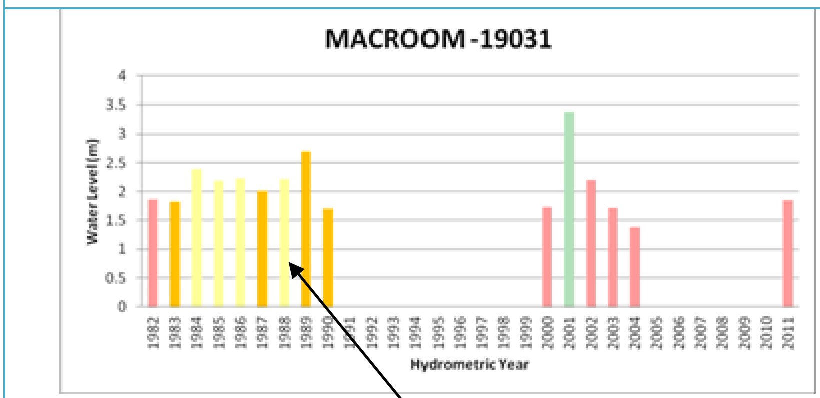
A plot of the water level available for the given station

Missing Data Check:

Here the missing data of the station is checked and its effect on the confidence of the Amax Reading is noted.

The Analysis is colour coded as follows:

	Confident of Amax (Included in Amax Analysis)
	Peak confirmed using continuous simulation rainfall model (Included in Amax Analysis)
	Amax may be affected by Missing Data (Included in Amax Analysis)
	Confidence in Amax is limited due to lack of water level data (Included in Amax Analysis)
	Insignificant data to confirm Amax


Results:

Results and Conclusions:

The graph above presents the confidence that has been determined from the available data for 19031 based on the analysis above (comparison with nearby stations and a data availability check). Though recorders have been in place intermittently since 1982, only one year can confirmed as the definite Amax. Based on the Amax data available from ESB from 1982 -1990, 4 years (1983, 1987, 1989 and 1990) appear to have complete records, however these AMaxs cannot be confirmed due to the lack of water level data.

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Results

Presents in graphical format the confidence levels of data for the hydrometric station

The Analysis is colour coded as follows:

	Confident of Amax (Included in Amax Analysis)
	Peak confirmed using continuous simulation rainfall model (Included in Amax Analysis)
	Amax may be affected by Missing Data (Included in Amax Analysis)
	Confidence in Amax is limited due to lack of water level data (Included in Amax Analysis)
	Insignificant data to confirm Amax

Conclusions

Discusses the results and conclusions of the data availability analysis carried out for the station.

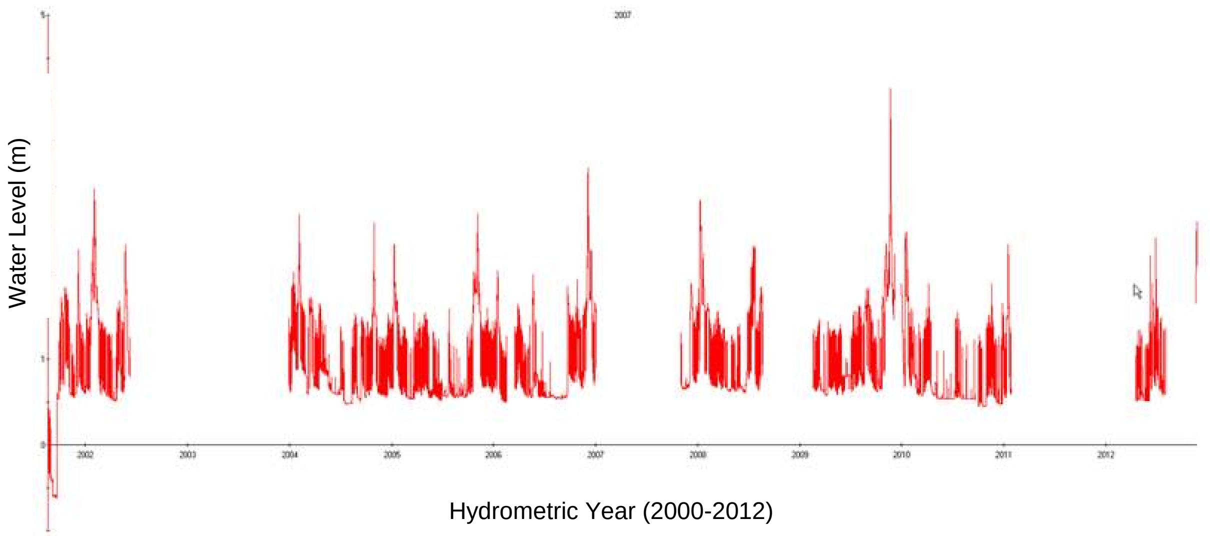
Station		Leemount Upstream 19011
Location:	160940 , 71680	
Comparison Stations	19012 - Leemount Downstream (Approximately 0.23km away)	
	19016 - Ovens (Approximately 6.24km away)	
Data Availability	1950-1999	Amax data, No supporting water level data
	2000-2001	Amax and Water Level Data
	2002	No Data
	2003 - 2012	Amax and Water Level Data

Amax Comparison with Nearby Stations:

HY	LEEMOUNT UPPER	% Data	LEEMOUNT LOWER	% Data	OVENS	% Data
1950	11/01/1951 20:00				11/01/1951 18:45	
1951	27/12/1951 23:00				01/10/1951 00:00	
1952	28/10/1952 14:00				28/10/1952 11:45	
1953	03/12/1953 18:00				04/12/1953 06:45	
1954	01/03/1955 22:00				30/11/1954 07:15	
1955	13/12/1955 07:00				13/12/1955 11:30	
1956	25/09/1957 13:00		25/09/1957 17:00		25/09/1957 17:00	80.20%
1957	28/01/1958 06:00		28/01/1958 06:00		23/12/1957 13:00	18.60%
1958	22/01/1959 07:00		19/01/1959 18:30		26/09/1959 11:00	1.20%
1959	03/02/1960 15:00		03/02/1960 16:00		01/01/1960 06:00	94.50%
1960	26/11/1960 14:00		04/12/1960 10:00		25/01/1961 16:15	51.40%
1961	13/12/1961 13:00		13/12/1961 07:00		16/01/1962 06:00	80.80%
1962	14/03/1963 22:00		14/03/1963 20:00		05/11/1962 02:45	17.60%
1963	19/03/1964 20:00		19/03/1964 18:00		17/08/1964 08:30	31.90%
1964	13/12/1964 10:00		13/12/1964 08:00		13/12/1964 19:15	92.70%
1965	15/02/1966 15:00		15/02/1966 12:00		15/02/1966 20:15	92.50%
1966	28/03/1967 02:30		28/02/1967 01:00		28/02/1967 00:45	71.90%
1967	09/01/1968 03:00		19/10/1967 07:00		16/01/1968 13:00	97.80%
1968	21/01/1969 01:00		21/01/1969 00:00		20/01/1969 23:30	74.70%
1969	23/01/1970 13:00		23/01/1970 11:30		21/01/1970 16:30	84.30%
1970	26/01/1971 23:30		26/11/1970 18:30		24/11/1970 00:00	51.10%
1971	15/01/1972 12:30		02/02/1972 20:00			
1972	20/01/1973 06:30		19/01/1973 11:00			
1973	12/01/1974 13:00		12/09/1974 22:00			
1974	25/01/1975 11:30		26/01/1975 20:00			
1975	20/10/1975 11:00		25/10/1975 13:00		17/05/1976 19:45	37.90%
1976	20/01/1977 22:30		20/01/1977 22:00		20/01/1977 21:30	92.60%
1977	23/02/1978 21:00		22/02/1978 17:30		23/02/1978 04:30	92.00%
1978	07/12/1978 20:00		07/12/1978 19:00		07/12/1978 23:45	86.20%
1979	30/09/1980 10:00		27/12/1979 01:30		05/12/1979 12:30	83.20%

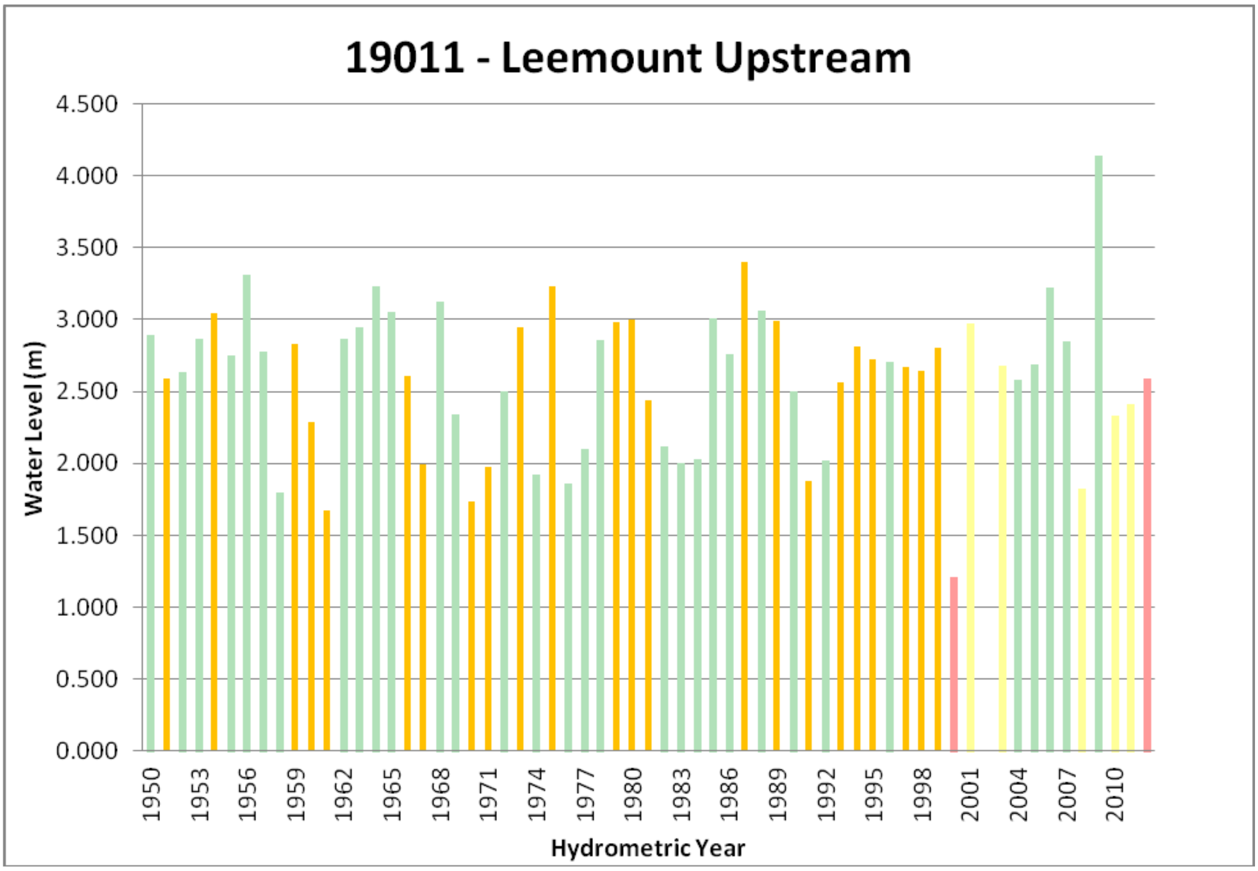
HY	LEEMOUNT UPPER	% Data	LEEMOUNT LOWER	% Data	OVENS	% Data
1980	02/10/1980 20:00		07/06/1981 21:00		02/11/1980 21:15	90.70%
1981	19/06/1982 07:40		19/06/1982 06:00		21/02/1982 15:00	79.30%
1982	08/11/1982 08:30		08/11/1982 08:30		01/11/1982 08:45	58.00%
1983	26/01/1984 16:30		26/01/1984 17:00		26/01/1984 23:30	63.10%
1984	08/02/1985 10:00		08/02/1985 10:00		08/02/1985 13:45	17.50%
1985	06/08/1986 17:00		06/08/1986 17:30		01/12/1985 15:45	15.40%
1986	13/12/1986 09:00		13/12/1986 11:00		13/12/1986 01:30	17.70%
1987	31/12/1987 05:30		12/01/1988 23:00		13/01/1988 00:30	19.80%
1988	21/10/1988 03:00		22/10/1988 03:00		21/10/1988 13:45	17.50%
1989	06/02/1990 18:00		06/02/1990 18:30		17/12/1989 06:00	15.60%
1990	04/01/1991 19:00		04/01/1991 20:00		05/01/1991 01:00	10.90%
1991	07/03/1992 04:00		25/11/1991 11:30		25/11/1991 06:45	14.90%
1992	15/01/1993 09:30		15/01/1993 12:00		18/12/1992 00:45	5.50%
1993	27/02/1994 03:00		12/01/1994 12:00		22/02/1994 22:00	13.60%
1994	10/03/1995 12:30				10/03/1995 11:45	21.20%
1995	08/01/1996 19:30				14/01/1996 22:15	21.00%
1996	20/02/1997 01:00				31/08/1997 22:30	27.00%
1997	18/11/1997 13:00				08/01/1998 16:30	29.30%
1998	31/12/1998 10:00				29/12/1998 14:45	58.30%
1999	24/12/1999 20:00				21/12/1999 03:45	63.30%
2000	28/09/2001 21:15	11.70%			17/10/2000 23:45	3.90%
2001	01/02/2002 23:30	68.40%	22/05/2002 03:30	60.90%	22/05/2002 03:30	29.30%
2001			10/06/2003 06:15	64.80%	10/06/2003 00:15	23.20%
2003	04/02/2004 20:30	75.60%	04/02/2004 18:45	79.10%	22/08/2004 21:30	52.70%
2004	29/10/2004 12:45	100.00%	29/10/2004 13:00	74.10%	08/01/2005 08:15	100.00%
2005	03/11/2005 09:30	92.20%			03/11/2005 15:45	92.80%
2006	03/12/2006 17:15	26.20%			03/12/2006 16:45	26.20%
2007	10/01/2008 21:45	80.60%			10/01/2008 05:45	67.90%
2008	02/09/2009 20:30	61.90%			31/08/2009 11:15	62.70%
2009	20/11/2009 02:45	91.00%			19/11/2009 21:30	95.10%
2010	16/01/2011 19:15	32.50%			17/11/2010 10:15	45.70%
2011	28/06/2012 09:15	28.70%			28/06/2012 19:15	28.70%
2012	22/11/2012 09:30	1.10%				

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1950	11/01/1951 20:00	No supporting Water Level Data	Same as Ovens (Good Data Coverage at Ovens)
1951	27/12/1951 23:00	No supporting Water Level Data	Different to Ovens (Good Data Coverage at Ovens)
1952	28/10/1952 14:00	No supporting Water Level Data	Same as Ovens (Good Data Coverage at Ovens)
1953	03/12/1953 18:00	No supporting Water Level Data	Same as Ovens (Good Data Coverage at Ovens)
1954	01/03/1955 22:00	No supporting Water Level Data	Different to Ovens (Good Data Coverage at Ovens)
1955	13/12/1955 07:00	No supporting Water Level Data	Same as Ovens (Good Data Coverage at Ovens)
1956	25/09/1957 13:00	No supporting Water Level Data	Same as comparison sites
1957	28/01/1958 06:00	No supporting Water Level Data	Same as Leemount Downstream, different to Ovens (poor data coverage at Ovens)
1958	22/01/1959 07:00	No supporting Water Level Data	Same as Leemount Downstream, different to Ovens (poor data coverage at Ovens)
1959	03/02/1960 15:00	No supporting Water Level Data	Different to Ovens (Good Data Coverage at Ovens)
1960	26/11/1960 14:00	No supporting Water Level Data	Different to adjacent sites
1961	13/12/1961 13:00	No supporting Water Level Data	Different to Ovens (Good Data Coverage at Ovens)
1962	14/03/1963 22:00	No supporting Water Level Data	Same as Leemount Downstream, different to Ovens (poor data coverage at Ovens)
1963	19/03/1964 20:00	No supporting Water Level Data	Same as Leemount Downstream, different to Ovens (poor data coverage at Ovens)
1964	13/12/1964 10:00	No supporting Water Level Data	Same as comparison sites
1965	15/02/1966 15:00	No supporting Water Level Data	Same as comparison sites
1966	28/03/1967 02:30	No supporting Water Level Data	Different to adjacent sites
1967	09/01/1968 03:00	No supporting Water Level Data	Different to Leemount Downstream, Similiar timing to Ovens
1968	21/01/1969 01:00	No supporting Water Level Data	Same as comparison sites
1969	23/01/1970 13:00	No supporting Water Level Data	Same as comparison sites
1970	26/01/1971 23:30	No supporting Water Level Data	Same as Leemount Downstream, different to Ovens (average data coverage at Ovens)
1971	15/01/1972 12:30	No supporting Water Level Data	Different to Leemount Downstream, No Comparison available with Ovens
1972	20/01/1973 06:30	No supporting Water Level Data	Same as Leemount Downstream
1973	12/01/1974 13:00	No supporting Water Level Data	Different to Leemount Downstream, No Comparison available with Ovens
1974	25/01/1975 11:30	No supporting Water Level Data	Same as Leemount Downstream
1975	20/10/1975 11:00	No supporting Water Level Data	Similiar timing to Leemount Downstream, Different to Ovens
1976	20/01/1977 22:30	No supporting Water Level Data	Same as comparison sites
1977	23/02/1978 21:00	No supporting Water Level Data	Same as comparison sites
1978	07/12/1978 20:00	No supporting Water Level Data	Same as comparison sites
1979	30/09/1980 10:00	No supporting Water Level Data	Different to adjacent sites
1980	02/10/1980 20:00	No supporting Water Level Data	Different to adjacent sites
1981	19/06/1982 07:40	No supporting Water Level Data	Different to Ovens (Good Data Coverage at Ovens)
1982	08/11/1982 08:30	No supporting Water Level Data	Same as Leemount Downstream, Simulliar timing to Ovens
1983	26/01/1984 16:30	No supporting Water Level Data	Same as comparison sites
1984	08/02/1985 10:00	No supporting Water Level Data	Same as comparison sites
1985	06/08/1986 17:00	No supporting Water Level Data	Known Flood Event
1986	13/12/1986 09:00	No supporting Water Level Data	Same as comparison sites
1987	31/12/1987 05:30	No supporting Water Level Data	Different to Comparison Sites
1988	21/10/1988 03:00	No supporting Water Level Data	Same as comparison sites
1989	06/02/1990 18:00	No supporting Water Level Data	Same as Leemount Downstream, Diiferent to Ovens (Poor data coverage at Ovens)
1990	04/01/1991 19:00	No supporting Water Level Data	Same as comparison sites
1991	07/03/1992 04:00	No supporting Water Level Data	Different to Comparison Sites
1992	15/01/1993 09:30	No supporting Water Level Data	Same as Leemount Downstream, different to Ovens (poor data coverage at Ovens)
1993	27/02/1994 03:00	No supporting Water Level Data	Different to Leemount Downstream, Similiar timing to Ovens
1994	10/03/1995 12:30	No supporting Water Level Data	Same as Ovens, no comparison available with Leemount Downstream
1995	08/01/1996 19:30	No supporting Water Level Data	Same as Ovens, no comparison available with Leemount Downstream
1996	20/02/1997 01:00	No supporting Water Level Data	Known Flood Event
1997	18/11/1997 13:00	No supporting Water Level Data	Different to Ovens, no comparison with Leemount Downstream available
1998	31/12/1998 10:00	No supporting Water Level Data	Same as Ovens, no comparison available with Leemount Downstream
1999	24/12/1999 20:00	No supporting Water Level Data	Same as Ovens, no comparison available with Leemount Downstream
2000	28/09/2001 21:15	Insignificant Data	Only 11% Data, Missing Known Flood Event
2001	01/02/2002 23:30	06/06/2002-30/09/2002	No Summer Data, adjacent peaks covered
2001	01/02/2002 23:30	No Data	No Data
2003	04/02/2004 20:30	01/10/2003-31/12/2005	Missing 2003 data
2004	29/10/2004 12:45		Complete Record
2005	03/11/2005 09:30	16/02/2006-17/03/2006	Missing one month of Data, Same as Ovens
2006	03/12/2006 17:15	04/01/2007-30/09/2007	Only three months of data, known flood event in area
2007	10/01/2008 21:45	30/09/2008	Missing two months of Data, same as Ovens
2008	02/09/2009 20:30	01/10/2008-16/02/2009	No Winter Data
2009	20/11/2009 02:45	30/09/2010	Missing Most of December, Known Flood Event
2010	16/01/2011 19:15	27/01/2011-30/09/2011	Only Oct-Jan Data
2011	28/06/2012 09:15	30/09/2012	Only Summer Data, Same as Ovens
2012	22/11/2012 09:30	30/09/2013	One 48 hours of Data

Results:



Results and Conclusions:

A recorder has been in place since 1950 with a digitized recorder being installed in 2000. The graph above presents the confidence that has been determined from the available data for 19011 based on the analysis above (comparison with nearby stations and a data availability check). Though recorders have been in place since 1950, only 32 years can confirmed as the definite Amax. Amax data is available from ESB from 1950-1999 however no supporting water level data is available so AMaxs are difficult to confirm.

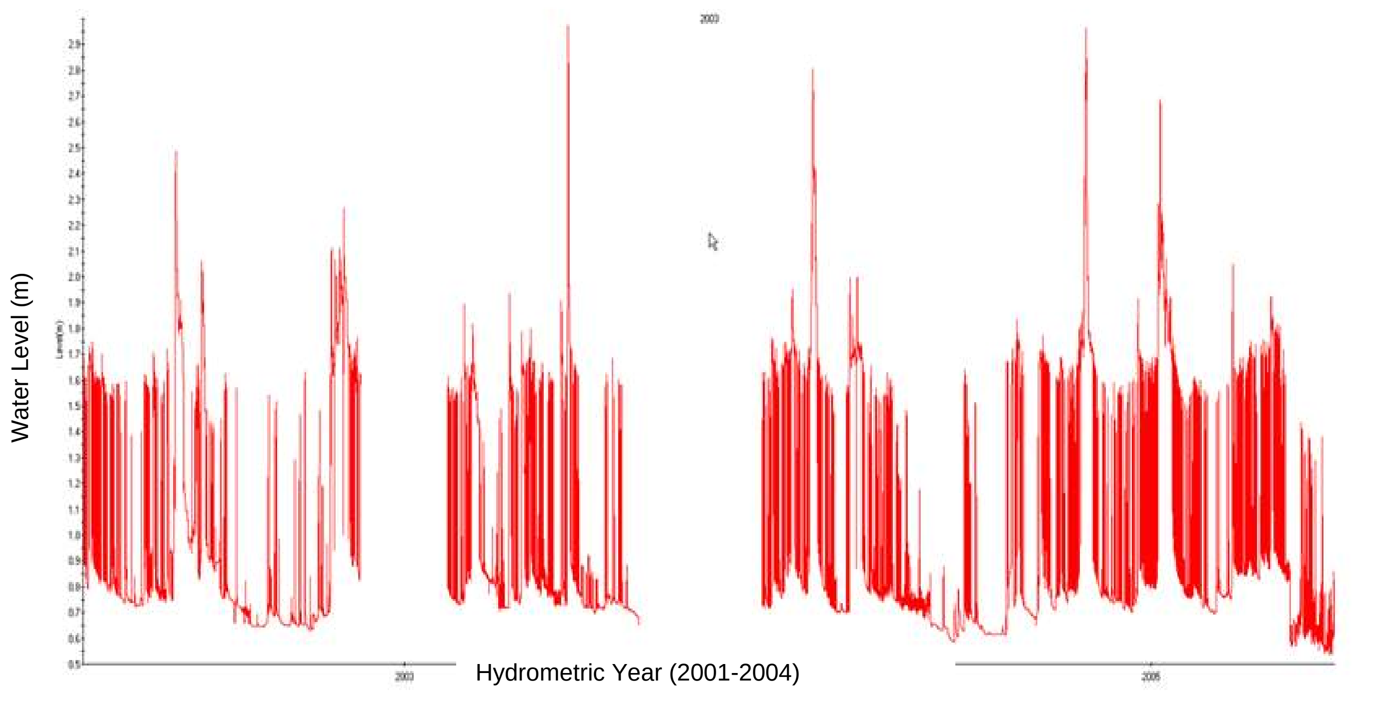
Station	Leemount Downstream 19012	
Location:	161140 , 71789	
Comparison Stations	19011 - Leemount Upstream (Approximately 0.23km away)	
	19016 - Ovens (Approximately 6.24km away)	
Data Availability	1956 - 1993	Amax data, No supporting water level data
	1994 - 2000	No Data
	2000 - 2004	Amax and Water Level Data

Amax Comparison with Nearby Stations:

	LEEMOUNT LOWER	% Data	LEEMOUNT UPPER	% Data	OVENS	% Data
1956	25/09/1957 17:00		25/09/1957 13:00		25/09/1957 17:00	80.20%
1957	28/01/1958 06:00		28/01/1958 06:00		23/12/1957 13:00	18.60%
1958	19/01/1959 18:30		22/01/1959 07:00		26/09/1959 11:00	1.20%
1959	03/02/1960 16:00		03/02/1960 15:00		01/01/1960 06:00	94.50%
1960	04/12/1960 10:00		26/11/1960 14:00		25/01/1961 16:15	51.40%
1961	13/12/1961 07:00		13/12/1961 13:00		16/01/1962 06:00	80.80%
1962	14/03/1963 20:00		14/03/1963 22:00		05/11/1962 02:45	17.60%
1963	19/03/1964 18:00		19/03/1964 20:00		17/08/1964 08:30	31.90%
1964	13/12/1964 08:00		13/12/1964 10:00		13/12/1964 19:15	92.70%
1965	15/02/1966 12:00		15/02/1966 15:00		15/02/1966 20:15	92.50%
1966	28/02/1967 01:00		28/03/1967 02:30		28/02/1967 00:45	71.90%
1967	19/10/1967 07:00		09/01/1968 03:00		16/01/1968 13:00	97.80%
1968	21/01/1969 00:00		21/01/1969 01:00		20/01/1969 23:30	74.70%
1969	23/01/1970 11:30		23/01/1970 13:00		21/01/1970 16:30	84.30%
1970	26/11/1970 18:30		26/01/1971 23:30		24/11/1970 00:00	51.10%
1971	02/02/1972 20:00		15/01/1972 12:30			
1972	19/01/1973 11:00		20/01/1973 06:30			
1973	12/09/1974 22:00		12/01/1974 13:00			
1974	26/01/1975 20:00		25/01/1975 11:30			
1975	25/10/1975 13:00		20/10/1975 11:00		17/05/1976 19:45	37.90%
1976	20/01/1977 22:00		20/01/1977 22:30		20/01/1977 21:30	92.60%
1977	22/02/1978 17:30		23/02/1978 21:00		23/02/1978 04:30	92.00%
1978	07/12/1978 19:00		07/12/1978 20:00		07/12/1978 23:45	86.20%
1979	27/12/1979 01:30		30/09/1980 10:00		05/12/1979 12:30	83.20%
1980	07/06/1981 21:00		02/10/1980 20:00		02/11/1980 21:15	90.70%
1981	19/06/1982 06:00		19/06/1982 07:40		21/02/1982 15:00	79.30%

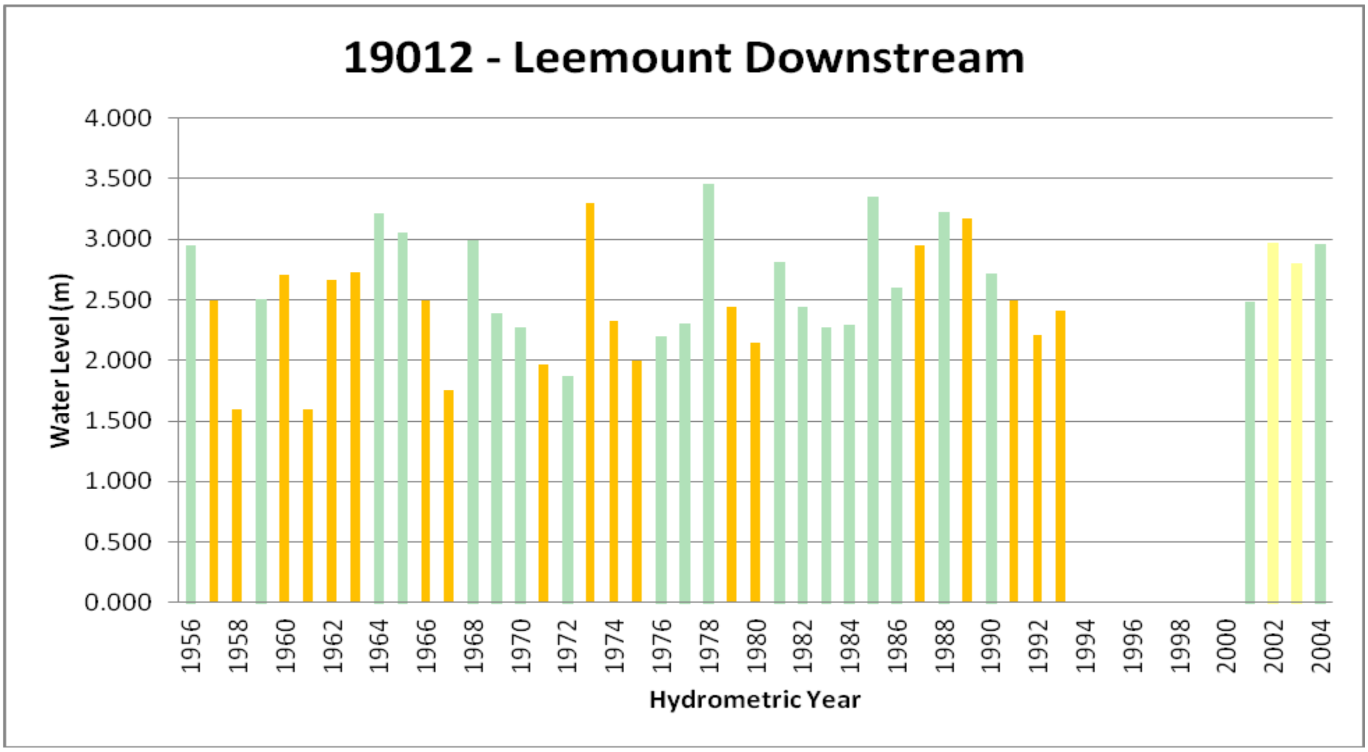
	LEEMOUNT LOWER	% Data	LEEMOUNT UPPER	% Data	OVENS	% Data
1982	08/11/1982 08:30		08/11/1982 08:30		01/11/1982 08:45	58.00%
1983	26/01/1984 17:00		26/01/1984 16:30		26/01/1984 23:30	63.10%
1984	08/02/1985 10:00		08/02/1985 10:00		08/02/1985 13:45	17.50%
1985	06/08/1986 17:30		06/08/1986 17:00		01/12/1985 15:45	15.40%
1986	13/12/1986 11:00		13/12/1986 09:00		13/12/1986 01:30	17.70%
1987	12/01/1988 23:00		31/12/1987 05:30		13/01/1988 00:30	19.80%
1988	22/10/1988 03:00		21/10/1988 03:00		21/10/1988 13:45	17.50%
1989	06/02/1990 18:30		06/02/1990 18:00		17/12/1989 06:00	15.60%
1990	04/01/1991 20:00		04/01/1991 19:00		05/01/1991 01:00	10.90%
1991	25/11/1991 11:30		07/03/1992 04:00		25/11/1991 06:45	14.90%
1992	15/01/1993 12:00		15/01/1993 09:30		18/12/1992 00:45	5.50%
1993	12/01/1994 12:00		27/02/1994 03:00		22/02/1994 22:00	13.60%
1994			10/03/1995 12:30		10/03/1995 11:45	21.20%
1995			08/01/1996 19:30		14/01/1996 22:15	21.00%
1996			20/02/1997 01:00		31/08/1997 22:30	27.00%
1997			18/11/1997 13:00		08/01/1998 16:30	29.30%
1998			31/12/1998 10:00		29/12/1998 14:45	58.30%
1999			24/12/1999 20:00		21/12/1999 03:45	63.30%
2000			28/09/2001 21:15	11.70%	17/10/2000 23:45	3.90%
2001	22/05/2002 03:30	60.90%	01/02/2002 23:30	68.40%	22/05/2002 03:30	29.30%
2002	10/06/2003 06:15	64.80%			10/06/2003 00:15	23.20%
2003	04/02/2004 18:45	79.10%	04/02/2004 20:30	75.60%	22/08/2004 21:30	52.70%
2004	29/10/2004 13:00	74.10%	29/10/2004 12:45	100.00%	08/01/2005 08:15	100.00%

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1956	25/09/1957 17:00	No supporting Water Level Data	Same as Comparison Stations
1957	28/01/1958 06:00	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1958	19/01/1959 18:30	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1959	03/02/1960 16:00	No supporting Water Level Data	Different to Ovens, peak covered in Ovens
1960	04/12/1960 10:00	No supporting Water Level Data	Different to adjacent peaks
1961	13/12/1961 07:00	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1962	14/03/1963 20:00	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1963	19/03/1964 18:00	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1964	13/12/1964 08:00	No supporting Water Level Data	Same as Comparison Stations
1965	15/02/1966 12:00	No supporting Water Level Data	Same as Comparison Stations
1966	28/02/1967 01:00	No supporting Water Level Data	Different to Leemount Upstream
1967	19/10/1967 07:00	No supporting Water Level Data	Different to adjacent peaks
1968	21/01/1969 00:00	No supporting Water Level Data	Same as Comparison Stations
1969	23/01/1970 11:30	No supporting Water Level Data	Same as Comparison Stations
1970	26/11/1970 18:30	No supporting Water Level Data	Same as Comparison Stations
1971	02/02/1972 20:00	No supporting Water Level Data	Different to Leemount Upstream, No comparison available with Ovens
1972	19/01/1973 11:00	No supporting Water Level Data	Same as Comparison Station
1973	12/09/1974 22:00	No supporting Water Level Data	Different to Leemount Upstream
1974	26/01/1975 20:00	No supporting Water Level Data	Different to Leemount Upstream, No comparison available with Ovens
1975	25/10/1975 13:00	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1976	20/01/1977 22:00	No supporting Water Level Data	Same as Comparison Stations
1977	22/02/1978 17:30	No supporting Water Level Data	Same as Comparison Stations
1978	07/12/1978 19:00	No supporting Water Level Data	Same as Comparison Stations
1979	27/12/1979 01:30	No supporting Water Level Data	Different to adjacent peaks
1980	07/06/1981 21:00	No supporting Water Level Data	Different to adjacent peaks
1981	19/06/1982 06:00	No supporting Water Level Data	Different to Ovens, peak covered in Ovens
1982	08/11/1982 08:30	No supporting Water Level Data	Same as Comparison Stations
1983	26/01/1984 17:00	No supporting Water Level Data	Same as Comparison Stations
1984	08/02/1985 10:00	No supporting Water Level Data	Same as Comparison Stations
1985	06/08/1986 17:30	No supporting Water Level Data	Known Flood Event
1986	13/12/1986 11:00	No supporting Water Level Data	Same as Comparison Stations
1987	12/01/1988 23:00	No supporting Water Level Data	Different to Leemount Upstream
1988	22/10/1988 03:00	No supporting Water Level Data	Same as Comparison Stations
1989	06/02/1990 18:30	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1990	04/01/1991 20:00	No supporting Water Level Data	Same as Comparison Stations
1991	25/11/1991 11:30	No supporting Water Level Data	Different to Leemount Upstream
1992	15/01/1993 12:00	No supporting Water Level Data	Different to Ovens (Ovens with significant data missing)
1993	12/01/1994 12:00	No supporting Water Level Data	Different to adjacent peaks
1994			No Data
1995			No Data
1996			No Data
1997			No Data
1998			No Data
1999			No Data
2000			No Data
2001	22/05/2002 03:30	01/10/2001-21/02/2002	No Summer months, Same as Ovens
2002	10/06/2003 06:15	18/11/2000-12/02/2003, 18/08/2003-30/09/2003	Same as Ovens, No winter data
2003	04/02/2004 18:45	01/10/2003-17/12/2003	Missing October to Mid December
2004	29/10/2004 13:00	29/06/2005-30/09/2005	No Summer Months

Results:



Results and Conclusions:

A recorder has been in place since 1956. Only chart data is available between 1956 and 1993 with a digitized recorder being installed in 2001 and removed in 2005. The graph above presents the confidence that has been determined from the available data for 19012 based on the analysis above (comparison with nearby stations and a data availability check). Though recorders have been in place since 1956, only 19 years can confirmed as the definite Amax. Amax data is available from ESB from 1956-1993 however no supporting water level data is available so AMaxs are difficult to confirm.

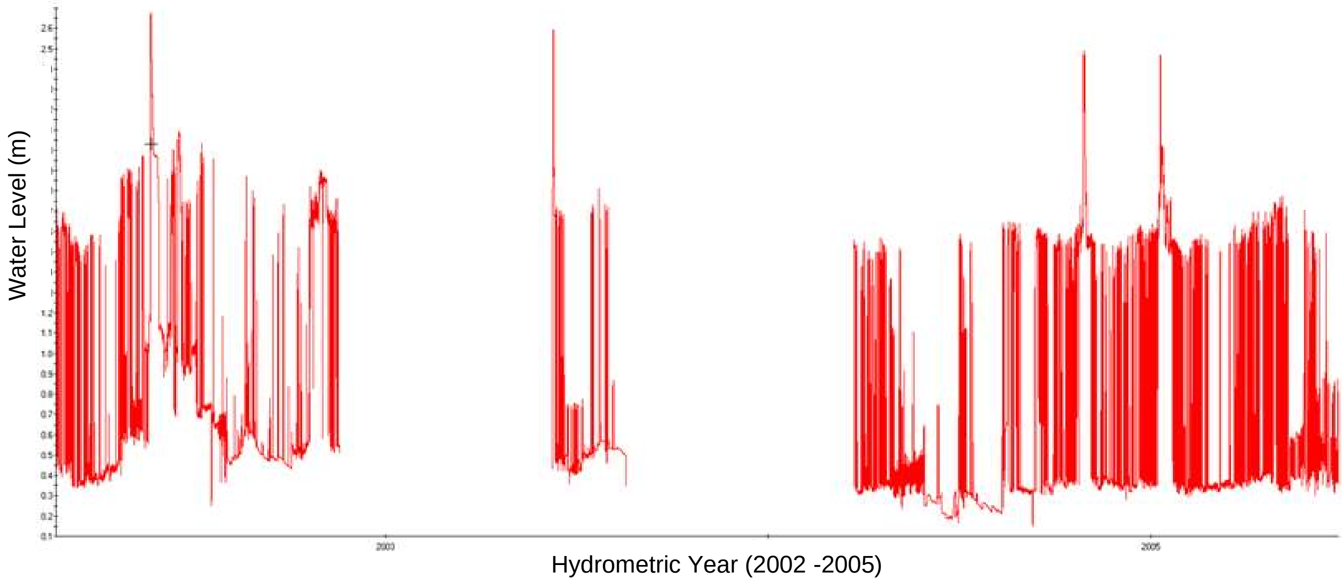
Station		Inniscarra 19013
Location:	155921 , 71955	
Comparison Stations	19011 - Leemount Upstream (Approximately 4.9km away)	
	19016 - Ovens (Approximately 2.23km away)	
Data Availability	1942 - 2000	Amax data, No supporting water level data
	2001 - 2004	Amax and Water Level Data

Amax Comparison with Nearby Stations:

HY	INNISCARRA 19013	%Data	LEEMOUNT US 19011	%Data	OVENS 19016	%Data
1942	17/01/1943					
1943	02/09/1944 13:00					
1944	02/02/1945 13:15					
1945	12/08/1946 21:00					
1946	15/01/1947 18:30					
1947	05/01/1948 03:00					
1948	02/12/1948 15:00					
1949	26/10/1949 03:00		26/10/1949 03:30		26/10/1949 05:15	82.30%
1950	11/01/1951 14:00		11/01/1951 20:00		11/01/1951 18:45	88.50%
1951	15/11/1951 09:00		27/12/1951 23:00		01/10/1951 00:00	90.20%
1952	28/10/1952 15:30		28/10/1952 14:00		28/10/1952 11:45	94.30%
1953	03/12/1953 21:00		03/12/1953 18:00		04/12/1953 06:45	88.60%
1954	01/03/1955 19:00		01/03/1955 22:00		30/11/1954 07:15	81.10%
1955	13/12/1955 04:00		13/12/1955 07:00		13/12/1955 11:30	96.40%
1956	25/09/1957 12:00		25/09/1957 13:00		25/09/1957 17:00	80.20%
1957	28/01/1958 06:00		28/01/1958 06:00		23/12/1957 13:00	18.60%
1958	04/11/1958 22:30		22/01/1959 07:00		26/09/1959 11:00	1.20%
1959	03/02/1960 02:00		03/02/1960 15:00		01/01/1960 06:00	94.50%
1960	27/01/1961 02:00		26/11/1960 14:00		25/01/1961 16:15	51.40%
1961	20/01/1962 15:00		13/12/1961 13:00		16/01/1962 06:00	80.80%
1962	14/03/1963 05:00		14/03/1963 22:00		05/11/1962 02:45	17.60%
1963	19/03/1964 04:30		19/03/1964 20:00		17/08/1964 08:30	31.90%
1964	12/12/1964 18:30		13/12/1964 10:00		13/12/1964 19:15	92.70%
1965	15/02/1966 10:00		15/02/1966 15:00		15/02/1966 20:15	92.50%
1966	23/02/1967 00:15		28/03/1967 02:30		28/02/1967 00:45	71.90%
1967	23/08/1968 14:00		09/01/1968 03:00		16/01/1968 13:00	97.80%
1968	21/01/1969 00:30		21/01/1969 01:00		20/01/1969 23:30	74.70%
1969	23/10/1970 11:00		23/01/1970 13:00		21/01/1970 16:30	84.30%
1970	26/11/1970 04:00		26/01/1971 23:30		24/11/1970 00:00	51.10%
1971	24/05/1972 02:30		15/01/1972 12:30			
1972	28/05/1973 14:00		20/01/1973 06:30			
1973	12/09/1974 21:30		12/01/1974 13:00			

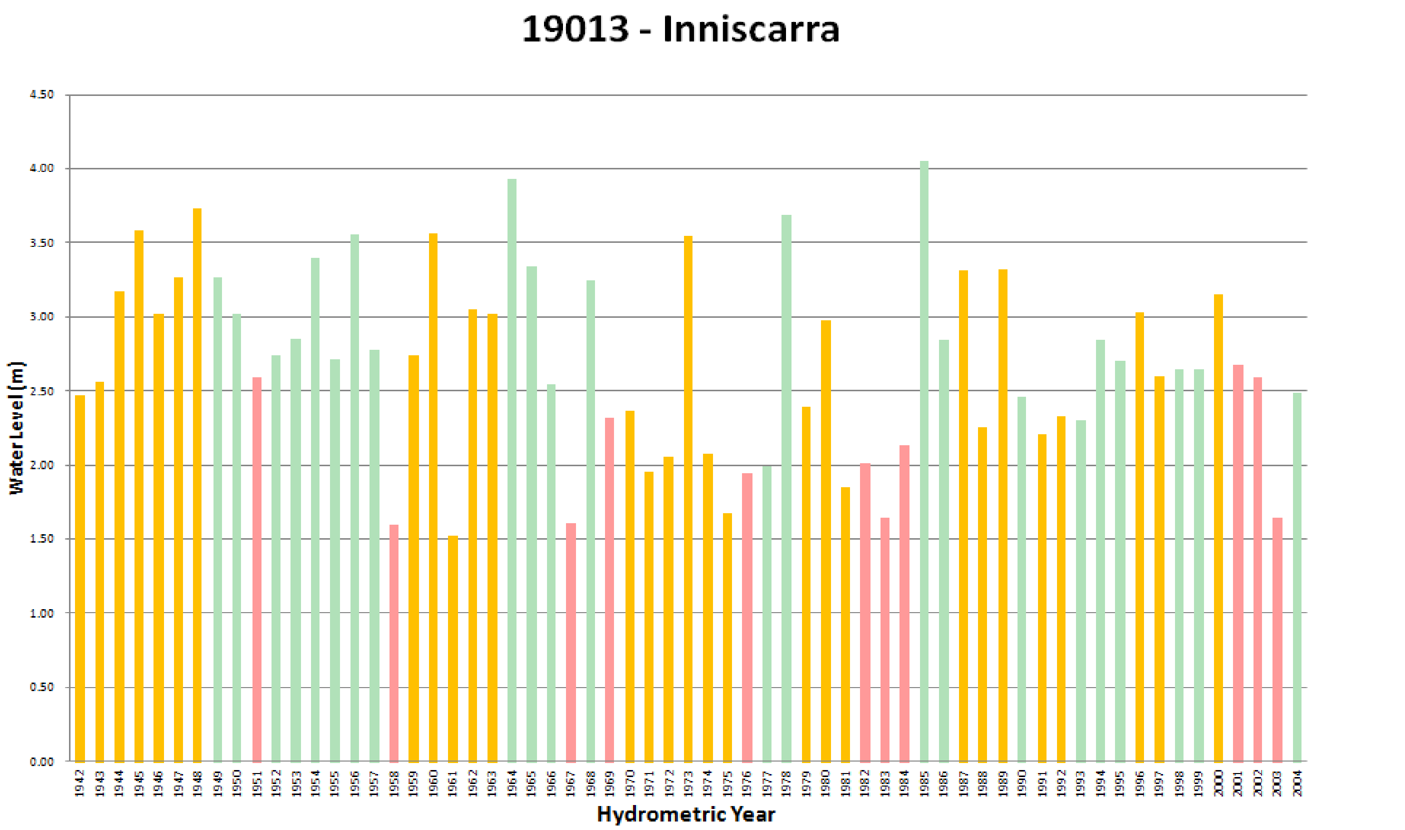
HY	INNISCARRA 19013	%Data	LEEMOUNT US 19011	%Data	OVENS 19016	%Data
1974	28/01/1975 17:00		25/01/1975 11:30			
1975	25/10/1975 14:00		20/10/1975 11:00		17/05/1976 19:45	37.90%
1976	28/04/1977 09:30		20/01/1977 22:30		20/01/1977 21:30	92.60%
1977	24/02/1978 14:00		23/02/1978 21:00		23/02/1978 04:30	92.00%
1978	07/12/1978 21:00		07/12/1978 20:00		07/12/1978 23:45	86.20%
1979	15/12/1979 17:00		30/09/1980 10:00		05/12/1979 12:30	83.20%
1980	02/11/1980 14:00		02/10/1980 20:00		02/11/1980 21:15	90.70%
1981	22/02/1982 06:00		19/06/1982 07:40		21/02/1982 15:00	79.30%
1982	02/10/1982 18:00		08/11/1982 08:30		01/11/1982 08:45	58.00%
1983	24/10/1983 21:00		26/01/1984 16:30		26/01/1984 23:30	63.10%
1984	14/08/1985 17:00		08/02/1985 10:00		08/02/1985 13:45	17.50%
1985	06/08/1986 13:00		06/08/1986 17:00		01/12/1985 15:45	15.40%
1986	13/12/1986 07:00		13/12/1986 09:00		13/12/1986 01:30	17.70%
1987	31/12/1987 05:00		31/12/1987 05:30		13/01/1988 00:30	19.80%
1988	14/03/1989 19:30		21/10/1988 03:00		21/10/1988 13:45	17.50%
1989	06/02/1990 22:00		06/02/1990 18:00		17/12/1989 06:00	15.60%
1990	04/01/1991 15:00		04/01/1991 19:00		05/01/1991 01:00	10.90%
1991	05/11/1991 08:30		07/03/1992 04:00		25/11/1991 06:45	14.90%
1992	30/05/1993 09:00		15/01/1993 09:30		18/12/1992 00:45	5.50%
1993	27/02/1994 08:00		27/02/1994 03:00		22/02/1994 22:00	13.60%
1994	10/03/1995 11:00		10/03/1995 12:30		10/03/1995 11:45	21.20%
1995	08/01/1996 20:00		08/01/1996 19:30		14/01/1996 22:15	21.00%
1996	19/02/1997 23:30		20/02/1997 01:00		31/08/1997 22:30	27.00%
1997	19/11/1997		18/11/1997 13:00		08/01/1998 16:30	29.30%
1998	31/12/1998 12:00		31/12/1998 10:00		29/12/1998 14:45	58.30%
1999	21/12/1999 14:00		24/12/1999 20:00		21/12/1999 03:45	63.30%
2000	30/11/2000 07:00		28/09/2001 21:15	11.70%	17/10/2000 23:45	3.90%
2001	22/05/2002 05:00	60.90%	01/02/2002 23:30	68.40%	22/05/2002 03:30	29.30%
2002	10/06/2003 07:45	32.70%			10/06/2003 00:15	23.20%
2003	18/08/2004 16:15	52.50%	04/02/2004 20:30	75.60%	22/08/2004 21:30	52.70%
2004	29/10/2004 02:45	74.40%	29/10/2004 12:45	100.00%	08/01/2005 08:15	100.00%

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1942	17/01/1943	No Supporting Water Level Data	No Comparison Site Available
1943	02/09/1944 13:00	No Supporting Water Level Data	
1944	02/02/1945 13:15	No Supporting Water Level Data	
1945	12/08/1946 21:00	No Supporting Water Level Data	
1946	15/01/1947 18:30	No Supporting Water Level Data	
1947	05/01/1948 03:00	No Supporting Water Level Data	
1948	02/12/1948 15:00	No Supporting Water Level Data	
1949	26/10/1949 03:00	No Supporting Water Level Data	Peak same as adjacent sites
1950	11/01/1951 14:00	No Supporting Water Level Data	Peak same as adjacent sites
1951	15/11/1951 09:00	No Supporting Water Level Data	Different to adjacent sites (events do not align at comparison sites)
1952	28/10/1952 15:30	No Supporting Water Level Data	Peak same as adjacent sites
1953	03/12/1953 21:00	No Supporting Water Level Data	Peak same as adjacent sites
1954	01/03/1955 19:00	No Supporting Water Level Data	Different to Ovens (81% Data, ovens hasdata for 01/03/1955 timing of the event)
1955	13/12/1955 04:00	No Supporting Water Level Data	Peak same as adjacent sites
1956	25/09/1957 12:00	No Supporting Water Level Data	Peak same as adjacent sites
1957	28/01/1958 06:00	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1958	04/11/1958 22:30	No Supporting Water Level Data	Different to adjacent sites (events do not align at comparison sites)
1959	03/02/1960 02:00	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1960	27/01/1961 02:00	No Supporting Water Level Data	Different to Leemount
1961	20/01/1962 15:00	No Supporting Water Level Data	Different to Leemount
1962	14/03/1963 05:00	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1963	19/03/1964 04:30	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1964	12/12/1964 18:30	No Supporting Water Level Data	Peak same as adjacent sites
1965	15/02/1966 10:00	No Supporting Water Level Data	Peak same as adjacent sites
1966	23/02/1967 00:15	No Supporting Water Level Data	Peak same as adjacent sites
1967	23/08/1968 14:00	No Supporting Water Level Data	Different to adjacent sites with same event timing
1968	21/01/1969 00:30	No Supporting Water Level Data	Peak same as adjacent sites
1969	23/10/1970 11:00	No Supporting Water Level Data	Different to adjacent sites with same event timing
1970	26/11/1970 04:00	No Supporting Water Level Data	Different to Leemount
1971	24/05/1972 02:30	No Supporting Water Level Data	Different to Leemount
1972	28/05/1973 14:00	No Supporting Water Level Data	Different to Leemount
1973	12/09/1974 21:30	No Supporting Water Level Data	Different to Leemount
1974	28/01/1975 17:00	No Supporting Water Level Data	Same as Leemount, No Comparison available for Ovens
1975	25/10/1975 14:00	No Supporting Water Level Data	Different to Ovens
1976	28/04/1977 09:30	No Supporting Water Level Data	Different to adjacent sites with same event timing
1977	24/02/1978 14:00	No Supporting Water Level Data	Peak same as adjacent sites
1978	07/12/1978 21:00	No Supporting Water Level Data	Peak same as adjacent sites
1979	15/12/1979 17:00	No Supporting Water Level Data	Different to Leemount
1980	02/11/1980 14:00	No Supporting Water Level Data	Different to Leemount
1981	22/02/1982 06:00	No Supporting Water Level Data	Different to Leemount
1982	02/10/1982 18:00	No Supporting Water Level Data	Different to adjacent sites with same event timing
1983	24/10/1983 21:00	No Supporting Water Level Data	Different to adjacent sites with same event timing
1984	14/08/1985 17:00	No Supporting Water Level Data	Different to adjacent sites with same event timing
1985	06/08/1986 13:00	No Supporting Water Level Data	Known Flood Event in Area
1986	13/12/1986 07:00	No Supporting Water Level Data	Peak same as adjacent sites
1987	31/12/1987 05:00	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1988	14/03/1989 19:30	No Supporting Water Level Data	Different to adjacent Stations
1989	06/02/1990 22:00	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1990	04/01/1991 15:00	No Supporting Water Level Data	Peak same as adjacent sites
1991	05/11/1991 08:30	No Supporting Water Level Data	Different to adjacent Stations
1992	30/05/1993 09:00	No Supporting Water Level Data	Different to adjacent Stations
1993	27/02/1994 08:00	No Supporting Water Level Data	Peak same as adjacent sites
1994	10/03/1995 11:00	No Supporting Water Level Data	Peak same as adjacent sites
1995	08/01/1996 20:00	No Supporting Water Level Data	Peak same as adjacent sites
1996	19/02/1997 23:30	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1997	19/11/1997	No Supporting Water Level Data	Different to Ovens, Significant Data gaps in Ovens Record
1998	31/12/1998 12:00	No Supporting Water Level Data	Peak same as adjacent sites
1999	21/12/1999 14:00	No Supporting Water Level Data	Peak same as adjacent sites
2000	30/11/2000 07:00	No Supporting Water Level Data	Different to adjacent Stations
2001	22/05/2002 05:00	01/10/2001 - 22/02/2002	No Winter Data, Missing Leemount Upstream Peak
2002	10/06/2003 07:45	18/11/2002 - 09/06/2002	No Winter Months
2003	18/08/2004 16:15	01/10/2003 - 23/03/2004	No Winter Months
2004	29/10/2004 02:45	29/06/2005 - 30/09/2005	No Summer Months, adjacent peak period covered

Results:



Results and Conclusions:

A recorder has been in place since 1942 with a digitized recorder being installed in 2001. Unfortunately there is significant gaps in the recordings from the digitized recorder and only data up as far as 2004 is available at present. The graph above presents the confidence that has been determined from the available data for 19013 based on the analysis above (comparison with nearby stations and a data availability check). Though recorders have been in place since 1942, only 23 years can confirmed as the definite Amax. Amax data is available from ESB from 1942-2000 however no supporting water level data is available so AMaxs are difficult to confirm.

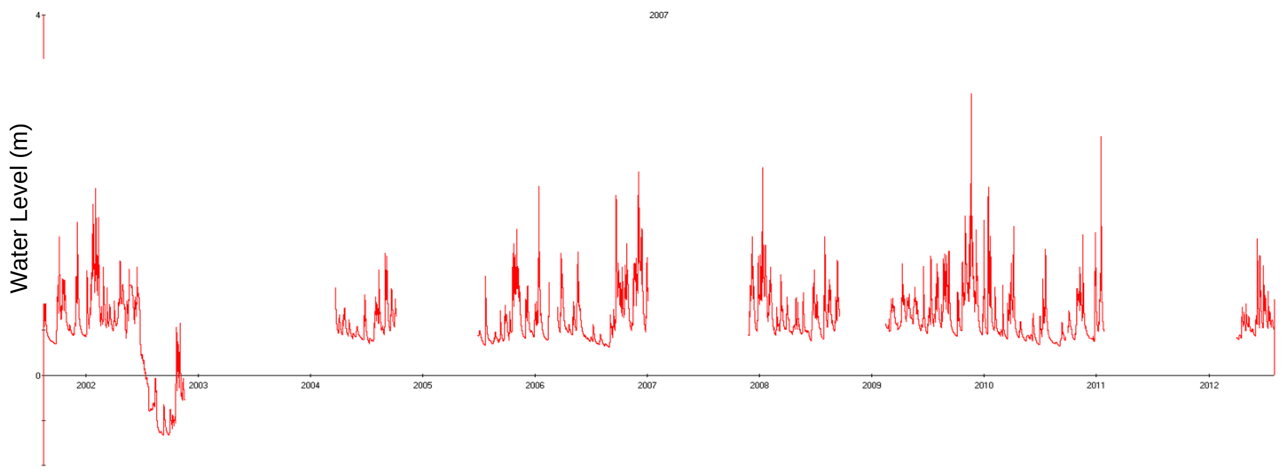
Station 19014		Lee Dromcarra
Location:	129670 , 67519	
Comparison Stations:	19031 - Macroom (Sullane) (Approximately 7.5km away)	
	19027 - Kill (Laney) (Approximately 9.6km away)	
	19016 - 19016 (Approximately 25.4km away, chosen for length of record)	
Data Availability:	1949 - 1974	Amax data, No supporting water level data
	1975 - 1976	No Data
	1997 - 1986	Amax data, No supporting water level data
	1987	No Data
	1988 - 1995	Amax data, No supporting water level data
	1996 - 1999	No Data
	2000 - 2012	Amax and Water Level Data

Amax Comparison with Nearby Stations:

Hydrometric Year	Lee Dromcarra	% Data	Macroom -Sullane	% Data	Kill	% Data	Ovens	% Data
1947	04/10/1947 21:00							
1948	02/12/1948 06:00							
1949	25/10/1949 20:00						26/10/1949 05:15	82%
1950	11/01/1951 11:00						11/01/1951 18:45	89%
1951	24/12/1951 14:00						01/10/1951 00:00	90%
1952	28/10/1952 05:00						28/10/1952 11:45	94%
1953	03/12/1953 17:30						04/12/1953 06:45	89%
1954	01/03/1955 07:00						30/11/1954 07:15	81%
1955	26/03/1956 00:00						13/12/1955 11:30	96%
1956	25/09/1957 00:30						25/09/1957 17:00	80%
1957	27/08/1958 10:00						23/12/1957 13:00	19%
1958	09/05/1959 23:00						26/09/1959 11:00	1%
1959	07/10/1959 16:00						01/01/1960 06:00	95%
1960	27/01/1961 11:00						25/01/1961 16:15	51%
1961	15/03/1962 11:30						16/01/1962 06:00	81%
1962	15/03/1963 02:00						05/11/1962 02:45	18%
1963	19/03/1964 15:30						17/08/1964 08:30	32%
1964	13/12/1964 04:00						13/12/1964 19:15	93%
1965	17/12/1965 23:30						15/02/1966 20:15	93%
1966	15/12/1966 21:00						28/02/1967 00:45	72%
1967	22/12/1967 23:45						16/01/1968 13:00	98%
1968	22/11/1968 14:30						20/01/1969 23:30	75%
1969	21/01/1970 20:45						21/01/1970 16:30	84%
1970	18/11/1970 21:00						24/11/1970 00:00	51%
1971	03/02/1972 10:00							
1972	01/12/1972 20:00							
1973	11/05/1974 23:00							
1974	18/01/1975 22:00							

Hydrometric Year	Lee Dromcarra	% Data	Macroom -Sullane	% Data	Kill	% Data	Ovens	% Data
1975							17/05/1976 19:45	38%
1976							20/01/1977 21:30	93%
1977	24/10/1977 10:00						23/02/1978 04:30	92%
1978	07/12/1978 15:00						07/12/1978 23:45	86%
1979	15/12/1979 14:00						05/12/1979 12:30	83%
1980	02/11/1980 08:00						02/11/1980 21:15	91%
1981	03/01/1982 23:00						21/02/1982 15:00	79%
1982	16/10/1982 21:00		25/09/1983 02:00				01/11/1982 08:45	58%
1983	15/12/1983 02:00		17/10/1983 12:00				26/01/1984 23:30	63%
1984	13/11/1984 00:00		29/11/1984 11:00		08/02/1985 05:00		08/02/1985 13:45	18%
1985	06/08/1986 13:00		21/12/1985 13:00		06/08/1986 00:30		01/12/1985 15:45	15%
1986	27/03/1987 14:15		12/12/1986 22:00		08/12/1986 15:30		13/12/1986 01:30	18%
1987			28/12/1987 23:00		12/01/1988 16:30		13/01/1988 00:30	20%
1988	14/03/1989 04:00		09/03/1989 21:00		14/10/1988 03:00		21/10/1988 13:45	18%
1989	06/02/1990 17:00		04/02/1990 18:30		06/02/1990 09:30		17/12/1989 06:00	16%
1990	02/01/1991 01:50		02/10/1990 21:00		01/01/1991 17:30		05/01/1991 01:00	11%
1991	24/04/1992 02:00				25/11/1991 00:30		25/11/1991 06:45	15%
1992	15/01/1993 07:00				15/01/1993 05:30		18/12/1992 00:45	6%
1993	22/02/1994 13:00				22/02/1994 18:00		22/02/1994 22:00	14%
1994	27/01/1995 15:00				09/03/1995 22:00		10/03/1995 11:45	21%
1995	16/10/1995 23:00				21/11/1995 03:00		14/01/1996 22:15	21%
1996					27/08/1997 01:00		31/08/1997 22:30	27%
1997					17/11/1997 18:00		08/01/1998 16:30	29%
1998					29/12/1998 12:00		29/12/1998 14:45	58%
1999					22/12/1999 06:30		21/12/1999 03:45	63%
2000	21/08/2001 16:15	12%	21/08/2001 16:45	12%	30/11/2000 01:30		17/10/2000 23:45	4%
2001	01/02/2002 12:15	100%	03/12/2001 21:45	77%	03/12/2001 18:30	61%	22/05/2002 03:30	29%
2002	02/11/2002 17:15	13%	11/09/2003 23:45	58%	14/09/2003 12:45	50%	10/06/2003 00:15	23%
2003	31/08/2004 16:00	52%	22/08/2004 16:30	54%	23/11/2003 20:15	81%	22/08/2004 21:30	53%
2004	24/07/2005 14:15	27%	04/10/2004 06:15	5%	08/01/2005 00:15	100%	08/01/2005 08:15	100%
2005	13/01/2006 08:00	93%			13/01/2006 06:45	93%	03/11/2005 15:45	93%
2006	03/12/2006 02:15	26%			03/12/2006 02:00	26%	03/12/2006 16:45	26%
2007	10/01/2008 08:30	81%			10/01/2008 06:30	81%	10/01/2008 05:45	68%
2008	08/09/2009 15:30	63%			11/07/2009 18:30	63%	31/08/2009 11:15	63%
2009	19/11/2009 14:00	98%			19/11/2009 15:00	25%	19/11/2009 21:30	95%
2010	16/01/2011 01:30	32%					17/11/2010 10:15	46%
2011	07/06/2012 20:30	33%	07/06/2012 22:45	24%	28/06/2012 04:30	33%	28/06/2012 19:15	29%
2012							19/11/2012 14:15	1%

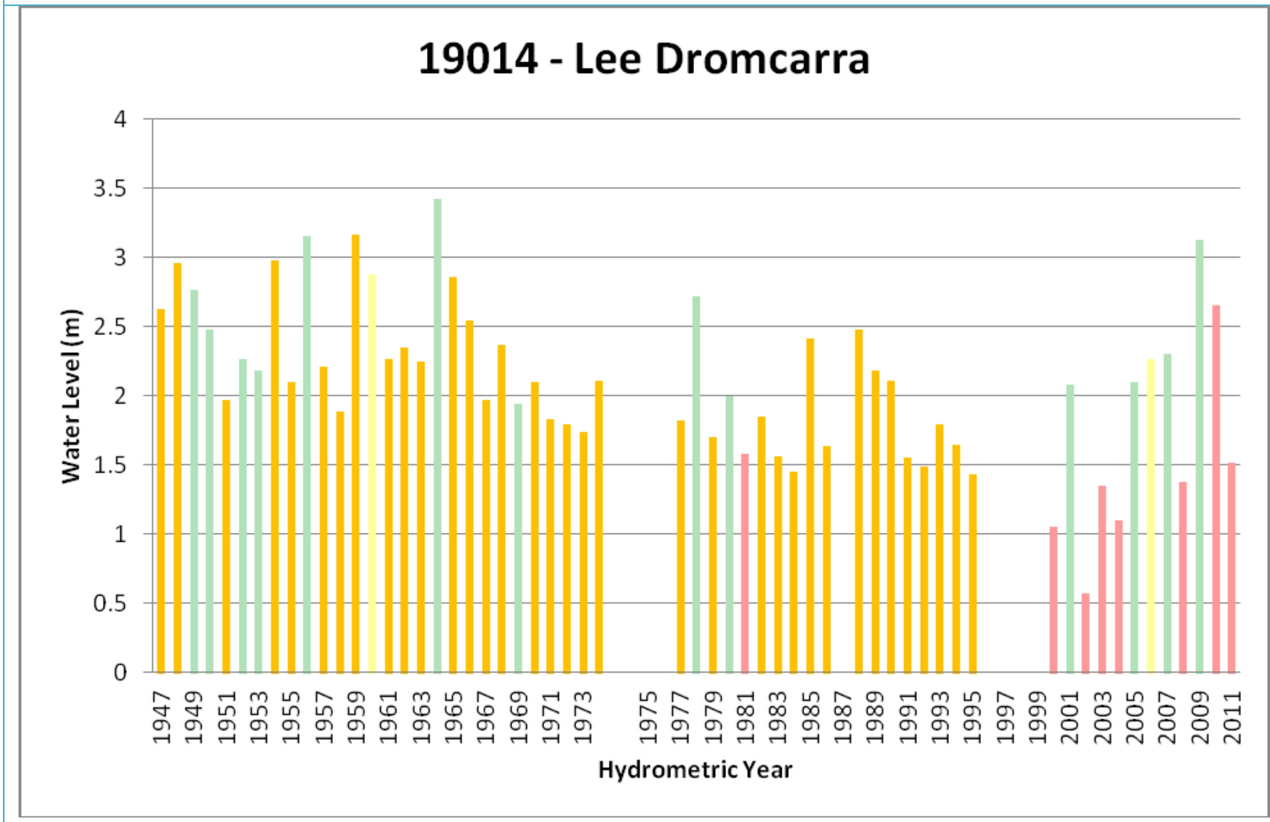
Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1947	04/10/1947 21:00		Record appears complete, different to comparison site
1948	02/12/1948 06:00		Record appears complete, different to comparison site
1949	25/10/1949 20:00		Similiar Peak to Ovens (82% Coverage)
1950	11/01/1951 11:00		Similiar Peak to Ovens (88% Coverage)
1951	24/12/1951 14:00		Record appears complete, different to comparison site
1952	28/10/1952 05:00		Similiar Peak to Ovens (94% Coverage)
1953	03/12/1953 17:30		Similiar Peak to Ovens (88% Coverage)
1954	01/03/1955 07:00		Record appears complete, different to comparison site
1955	26/03/1956 00:00		Record appears complete, different to comparison site
1956	25/09/1957 00:30		Similiar Peak to Ovens (80% Coverage)
1957	27/08/1958 10:00		Record appears complete, different to comparison site
1958	09/05/1959 23:00		Record appears complete, different to comparison site
1959	07/10/1959 16:00		Record appears complete, different to comparison site
1960	27/01/1961 11:00		Similiar Peak to Ovens (51% Coverage)
1961	15/03/1962 11:30		Record appears complete, different to comparison site
1962	15/03/1963 02:00		Record appears complete, different to comparison site
1963	19/03/1964 15:30		Record appears complete, different to comparison site
1964	13/12/1964 04:00		Similiar Peak to Ovens (93% Coverage)
1965	17/12/1965 23:30		Record appears complete, different to comparison site
1966	15/12/1966 21:00		Record appears complete, different to comparison site
1967	22/12/1967 23:45		Record appears complete, different to comparison site
1968	22/11/1968 14:30		Record appears complete, different to comparison site
1969	21/01/1970 20:45		Similiar Peak to Ovens (84% Coverage)
1970	18/11/1970 21:00		Record appears complete, different to comparison site
1971	03/02/1972 10:00		Record appears complete, different to comparison site
1972	01/12/1972 20:00		Record appears complete, different to comparison site
1973	11/05/1974 23:00		Record appears complete, different to comparison site
1974	18/01/1975 22:00		Record appears complete, different to comparison site

Hydrometric Year	Recorded Amax	Missing Data	Notes
1975			
1976			
1977	24/10/1977 10:00		Record appears complete, different to comparison site
1978	07/12/1978 15:00	14/01/79-06/02/79	Similiar to Peak in Ovens (86% Data)
1979	15/12/1979 14:00		Record appears complete, different to comparison site
1980	02/11/1980 08:00		Similiar to Peak in Ovens (90% Data)
1981	03/01/1982 23:00	01/1982-09/1982	9 months of Data Missing
1982	16/10/1982 21:00		Record appears complete, different to comparison site
1983	15/12/1983 02:00		Record appears complete, different to comparison site
1984	13/11/1984 00:00		Record appears complete, different to comparison site
1985	06/08/1986 13:00		Record appears complete, different to comparison site
1986	27/03/1987 14:15		Record appears complete, different to comparison site
1987			
1988	14/03/1989 04:00		Record appears complete, different to comparison site
1989	06/02/1990 17:00		Record appears complete, different to comparison site
1990	02/01/1991 01:50		Record appears complete, same as comparison site but insignificant data at comparison site
1991	24/04/1992 02:00		Record appears complete, different to comparison site
1992	15/01/1993 07:00		Record appears complete, different to comparison site
1993	22/02/1994 13:00		Record appears complete, same as comparison site but insignificant data at comparison site
1994	27/01/1995 15:00		Record appears complete, different to comparison site
1995	16/10/1995 23:00		Record appears complete, different to comparison site
1996			
1997			
1998			
1999			
2000	21/08/2001 16:15	01/10/2000-16/08/2001	Insignificant Data
2001	01/02/2002 12:15		Complete record
2002	02/11/2002 17:15	18/11/2002-30/09/2003	(Negative data 01/10/2002-18/11/2002)
2003	31/08/2004 16:00	01/10/2003-22/03/2004	No Winter Months
2004	24/07/2005 14:15	05/10/2004-29/06/2005	No Winter Months
2005	13/01/2006 08:00	16/02/2006-15/03/2006	Good data coverage
2006	03/12/2006 02:15	14/01/2007-30/09/2007	Same as adjacent Stations, Known Flood Event, Missing 9 months of data
2007	10/01/2008 08:30	01/10/2007-27/11/2007	Same as comparison sites
2008	08/09/2009 15:30	01/11/2008-15/02/2009	No Winter Months
2009	19/11/2009 14:00		Known Flood Event, Good Data Coverage
2010	16/01/2011 01:30	26/02/2011-30/09/2011	Insignificant Data
2011	07/06/2012 20:30	01/10/2011-02/04/2012, 01/08/2012-30/09/2012	Insignificant Data

Results:



Results and Conclusions:

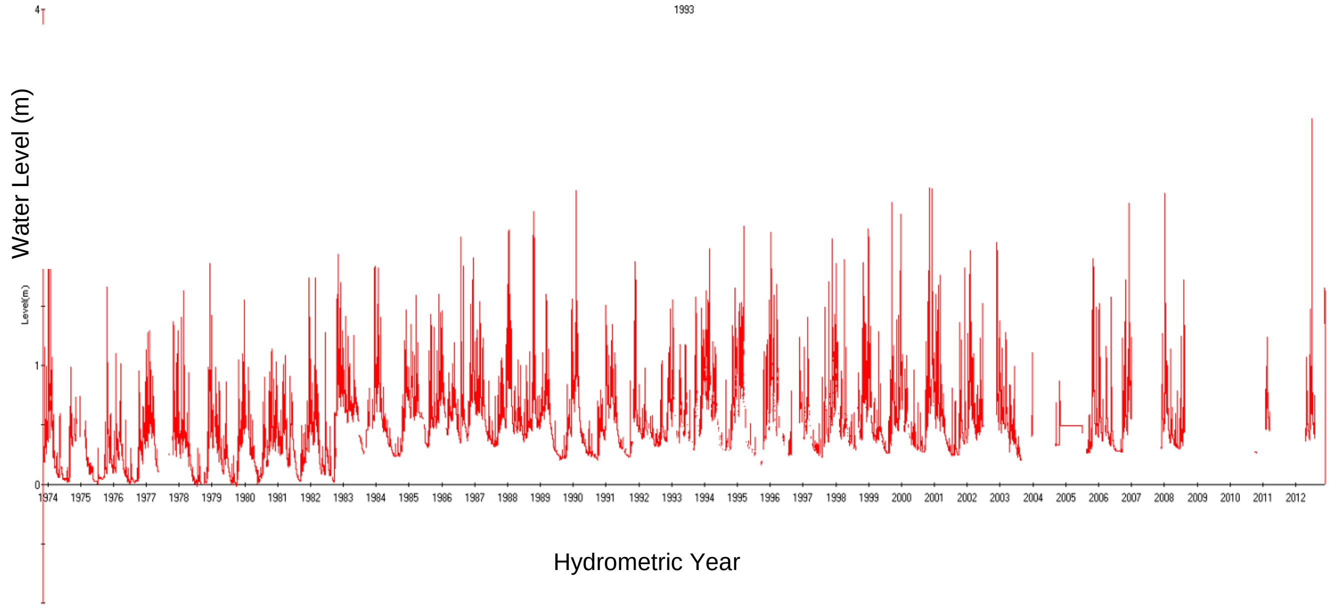
The graph above presents the confidence that has been determined from the available data for 19014 based on the analysis above (comparison with nearby stations and a data availability check). Though recorders have been in place since 1949, a total of 64 years of data only 13 yeasr can confirmed as the definite Amax. Based on the Amax data available from ESB from 1947 -1995, 34 years appear to have complete records, however these AMaxs cannot be confirmed due to the lack of supporting water level data.

Station 19015		Healy's Bridge
Location:	161140 , 73266	
	19018 - Tower (Approximately 2m away)	
Comparison Stations	19017 - Bawnnafinny (Approximately 2.1km away)	
	19045 - Gothic (Approximately 2.3m away)	
	19016 - Ovens (Approximately 6.3km away)	
Data Availability	1973 - 2007	Amax and Water Level Data
	2008 - 2009	No Data
	2010 - 2012	Amax and Water Level Data

Amax Comparison with Nearby Stations:

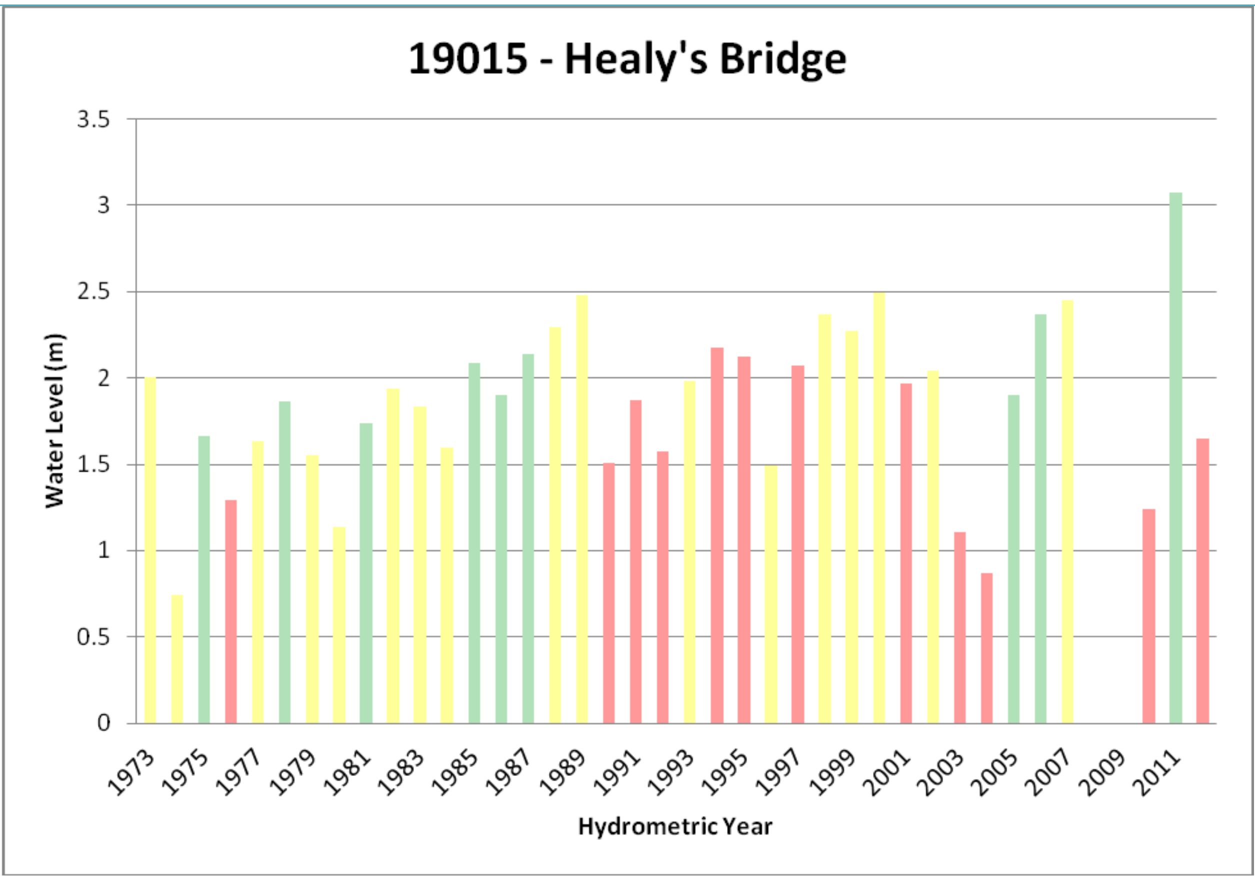
HY	Healy's Bridge	% Data	Tower	% Data	Gothic	% Data	Ovens	% Data	Bawnnafinny	% Data
1973	07/01/1974 13:00	84%								
1974	27/12/1974 10:15	69%								
1975	23/10/1975 03:15	97%					17/05/1976 19:45	38%		
1976	03/02/1977 09:00	63%	24/08/1977 10:00	16%			20/01/1977 21:30	93%	15/05/1977 11:45	4%
1977	22/02/1978 17:00	85%	22/02/1978 15:45	20%			23/02/1978 04:30	92%	23/12/1977 03:15	24%
1978	07/12/1978 02:00	91%	07/12/1978 00:15	20%			07/12/1978 23:45	86%	07/12/1978 14:15	20%
1979	27/12/1979 02:15	84%	26/12/1979 21:15	23%			05/12/1979 12:30	83%	22/10/1979 18:30	28%
1980	02/11/1980 12:30	98%	31/05/1981 02:30	25%			02/11/1980 21:15	91%	31/05/1981 01:30	31%
1981	21/02/1982 08:00	94%	13/12/1981 16:30	24%			21/02/1982 15:00	79%	21/02/1982 00:00	22%
1982	08/11/1982 08:00	90%	08/11/1982 06:45	27%			01/11/1982 08:45	58%	08/11/1982 05:45	25%
1983	16/12/1983 18:30	91%	16/12/1983 14:45	31%			26/01/1984 23:30	63%	26/01/1984 08:45	20%
1984	20/03/1985 19:30	98%	08/02/1985 05:15	20%			08/02/1985 13:45	18%	08/02/1985 05:00	20%
1985	06/08/1986 05:45	97%	01/12/1985 12:45	46%			01/12/1985 15:45	15%	06/08/1986 02:45	79%
1986	12/12/1986 22:30	100%	12/12/1986 21:00	48%			13/12/1986 01:30	18%	12/12/1986 20:45	100%
1987	19/01/1988 01:00	100%	18/01/1988 23:30	58%			13/01/1988 00:30	20%	12/01/1988 18:15	100%
1988	21/10/1988 22:15	98%	11/10/1988 09:30	42%			21/10/1988 13:45	18%	09/03/1989 19:45	92%
1989	06/02/1990 13:00	98%	06/02/1990 10:00	42%			17/12/1989 06:00	16%	05/02/1990 13:00	100%
1990	01/01/1991 18:00	85%	07/03/1991 04:00	39%			05/01/1991 01:00	11%	01/01/1991 20:30	98%
1991	25/11/1991 04:00	85%	24/11/1991 00:30	20%			25/11/1991 06:45	15%	24/04/1992 03:45	100%
1992	19/09/1993 06:00	41%	19/09/1993 22:15	36%			18/12/1992 00:45	6%	10/06/1993 04:30	40%
1993	22/02/1994 15:00	28%	26/02/1994 23:15	43%			22/02/1994 22:00	14%	03/05/1994 05:45	16%
1994	09/03/1995 21:45	26%	10/03/1995 00:30	35%	02/03/1995 19:30	87%	10/03/1995 11:45	21%		
1995	08/01/1996 19:00	22%	08/01/1996 19:00	89%	08/01/1996 18:00	100%	14/01/1996 22:15	21%		
1996	27/08/1997 03:00	24%	28/10/1996 20:45	96%	25/10/1996 20:45	100%	31/08/1997 22:30	27%		
1997	17/11/1997 21:30	32%	18/11/1997 04:00	89%	08/01/1998 20:30	100%	08/01/1998 16:30	29%		
1998	18/09/1999 15:45	60%	18/09/1999 15:30	94%	01/11/1998 05:15	9%	29/12/1998 14:45	58%		
1999	24/12/1999 20:45	40%	24/12/1999 16:15	43%			21/12/1999 03:45	63%		
2000	05/11/2000 15:30	89%					17/10/2000 23:45	4%		
2001	01/02/2002 14:00	73%					22/05/2002 03:30	29%		
2002	21/11/2002 06:00	75%					10/06/2003 00:15	23%		
2003	26/12/2003 09:45	10%					22/08/2004 21:30	53%		
2004	24/10/2004 06:15	86%			22/03/2005 01:30	66%	08/01/2005 08:15	100%		
2005	03/11/2005 01:00	93%			03/11/2005 20:30	40%	03/11/2005 15:45	93%		
2006	03/12/2006 05:00	26%					03/12/2006 16:45	26%		
2007	10/01/2008 10:45	72%			10/01/2008 10:15	97%	10/01/2008 05:45	68%		
2008					31/01/2009 03:00	99%	31/08/2009 11:15	63%		
2009					19/11/2009 17:45	100%	19/11/2009 21:30	95%		
2010	18/02/2011 20:15	21%			17/11/2010 02:30	100%	17/11/2010 10:15	46%		
2011	28/06/2012 07:30	29%			28/06/2012 06:30	92%	28/06/2012 19:15	29%		
2012	19/11/2012 07:15	1%			25/01/2013 19:30	56%	19/11/2012 14:15	1%		

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1973	07/01/1974 13:00	01/10/1973 -11/11/1973, 26/01/1974-29/01/1974, 02/02/1974-04/02/1974	Insignificant Winter Data
1974	27/12/1974 10:15	05/10/1974-19/10/1974, 16/11/1974-21/12/1974, 28/12/1974-22/02/1975, 29/03/1975-05/04/1975	Insignificant Winter Data
1975	23/10/1975 03:15	06/03/1976-10/03/1976, 19/06/1976-26/06/1976	Only Minor Gaps, same as comparison site
1976	03/02/1977 09:00	14/05/1977-30/09/1977	No Summer Data
1977	22/02/1978 17:00	01/10/1977-22/10/1977, 01/09/1977-30/09/1978	No Event alignment with adjacent stations
1978	07/12/1978 02:00	01/10/1978-08/10/1978, 20/01/1979-03/02/1979,12/05/1979-26/05/1979, Negative datum from 27/09/1979	Good Coverage, same as neighbouring stations
1979	27/12/1979 02:15	12/01/1980-09/02/1980, 14/06/1960-21/06/1960, 23/08/1980-30/08/1980, 12/09/1980,20/09/1980	Good data coverage, different to comparison sites
1980	02/11/1980 12:30	22/05/1981-27/05/1981, 30/05/1981-03/06/1981	Good data coverage, missing data of neighbouring site peak
1981	21/02/1982 08:00	13/11/1981-21/11/1981, 21/08/1982-04/09/1982	Good Coverage, same as neighbouring stations
1982	08/11/1982 08:00	09/10/1982-14/10/1982, Scattered data from 18/06/1983 onwards	Scattered Summer Data
1983	16/12/1983 18:30	15/10/1983-19/11/1983	Missing one month of winter data, some alignment in peaks
1984	20/03/1985 19:30	02/02/1985 - 09/02/1985, Irregular between 19/04/1985 and 02/08/1985	Scattered Summer data
1985	06/08/1986 05:45	08/02/1986 - 19/02/1986	Good Data Coverage
1986	12/12/1986 22:30		Good Data Coverage
1987	19/01/1988 01:00		Good Data Coverage
1988	21/10/1988 22:15	03/02/1989-10/02/1989	Good Data Coverage
1989	06/02/1990 13:00	19/01/1990-26/01/1990	Good Data Coverage
1990	01/01/1991 18:00	08/02/1991-22/02/1991, Scattered Data from 28/06/1991-28/09/1991	Scattered Summer Data
1991	25/11/1991 04:00	01/10/1991-01/11/1991,09/04/1992-14/08/1992, 09/08/1992-14/08/1992	Some Peak Alignment but significant data gaps
1992	19/09/1993 06:00	Scattered Data from 31/12/1992	Insignificant Data
1993	22/02/1994 15:00	Scattered Data throughout year	Insignificant Data
1994	09/03/1995 21:45	Scattered Data throughout year	Insignificant Data
1995	08/01/1996 19:00	Scattered Data throughout year	Peak Alignment but significant data gaps
1996	27/08/1997 03:00	Scattered Data throughout year	Insignificant Data
1997	17/11/1997 21:30	Scattered Data throughout year	Insignificant Data
1998	18/09/1999 15:45	Scattered Data throughout the year	Some Peak Alignment but significant data gaps
1999	24/12/1999 20:45	Scattered Data throughout year	Insignificant Data
2000	05/11/2000 15:30	07/11/2000-11/11/2000,19/11/2000-30/11/2000, 24/12/200-01/01/2001	Gaps in winter data
2001	01/02/2002 14:00	25/06/2002 - 30/09/2002	No Summer Data
2002	21/11/2002 06:00	01/10/2002-18/11/2002, 18/08/2003-30/09/2003	Gaps in Data, No peak alignment
2003	26/12/2003 09:45	01/10/2003 -17/12/2003, 30/12/2003-07/09/2004	Insignificant Data
2004	24/10/2004 06:15	01/10/2004-19/05/2005	Insignificant Data
2005	03/11/2005 01:00	15/02/2006-15/03/2006	Good Data Coverage, Aligns with neighbouring peaks
2006	03/12/2006 05:00	03/01/2007-30/09/2007	Known Flood Event
2007	10/01/2008 10:45	01/10/2007-28/11/2007, 15/08/2008-30/09/2008	Missing 1.5 months of Winter Data
2008			
2009			
2010	18/02/2011 20:15	01/10/2010-27/01/2011,12/03/2011-30/09/2011	Insignificant Data
2011	28/06/2012 07:30	01/10/2011-18/04/2012, 01/08/2012-30/09/2012	Insignificant Data
2012		One Day of data	Insignificant Data

Results:



Results and Conclusions:

The graph above presents the confidence that has been determined from the available data for 19015 based on the analysis above (comparison with nearby stations and a data availability check). Data is available from 1973 to present day. Of the 39 years of data confidence in the capturing of the Amax is only available for 8 Hydrometric Years. Data availabilty was very high in the 1980's however, the standard of recordings are particularly poor between 1992 and 1999. For these years the data availability suffers from the data recorder being very inconsistent, where it is possible to analyse base flow when the recorder is recording intermittently but it may be possible to miss intense events. Since 2008 the level of recordings are particularly poor.

Station 19016		Ovens
Location:	154934 , 69976	
Comparison Stations	19014 - Lee Dromcarra (25.3km, Chosen for Length of Record)	
	19028 - Dripsey (Approximately 8.1km away)	
	19018 - Tower (Approximately 6.2m away)	
	19015 - Healys' Bridge (Approximately 6.3km away)	

19014 was used for comparison purposes between 1949 - 1970 as it was the only other station with records for that period of time. 19028, 19018 and 19015 are closer stations and were used to compare from 1971 to present day records.

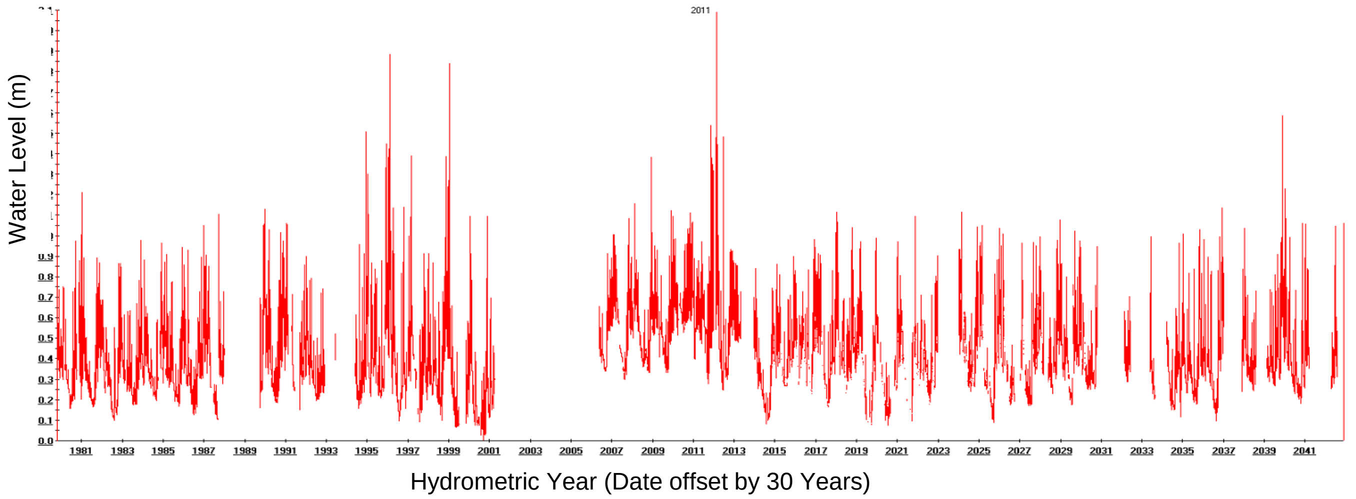
Data Availability		
	1949 - 1970	Amax and Water Level Data
	1971 - 1974	No Data
	1975 - 2011	Amax and Water Level Data

Amax Comparison with Nearby Stations:

HY	OVENS - 19016	% Data	LEE DROMCARRA - 19014
1949	26/10/1949 05:15	82.30%	25/10/1949 20:00
1950	11/01/1951 18:45	88.50%	11/01/1951 11:00
1951	01/10/1951 00:00	90.20%	24/12/1951 14:00
1952	28/10/1952 11:45	94.30%	28/10/1952 05:00
1953	04/12/1953 06:45	88.60%	03/12/1953 17:30
1954	30/11/1954 07:15	81.10%	01/03/1955 07:00
1955	13/12/1955 11:30	96.40%	26/03/1956 00:00
1956	25/09/1957 17:00	80.20%	25/09/1957 00:30
1957	23/12/1957 13:00	18.60%	27/08/1958 10:00
1958	26/09/1959 11:00	1.20%	09/05/1959 23:00
1959	01/01/1960 06:00	94.50%	07/10/1959 16:00
1960	25/01/1961 16:15	51.40%	27/01/1961 11:00
1961	16/01/1962 06:00	80.80%	15/03/1962 11:30
1962	05/11/1962 02:45	17.60%	15/03/1963 02:00
1963	17/08/1964 08:30	31.90%	19/03/1964 15:30
1964	13/12/1964 19:15	92.70%	13/12/1964 04:00
1965	15/02/1966 20:15	92.50%	17/12/1965 23:30
1966	28/02/1967 00:45	71.90%	15/12/1966 21:00
1967	16/01/1968 13:00	97.80%	22/12/1967 23:45
1968	20/01/1969 23:30	74.70%	22/11/1968 14:30
1969	21/01/1970 16:30	84.30%	21/01/1970 20:45
1970	24/11/1970 00:00	51.10%	18/11/1970 21:00
1971			03/02/1972 10:00

HY	OVENS - 19016	% Data	DRIPSEY - 19028	% Data	TOWER - 19018	% Data	HEALY'S BRIDGE - 19015	% Data
1972								
1973							07/01/1974 13:00	83.60%
1974							27/12/1974 10:15	69.10%
1975	17/05/1976 19:45	37.90%					23/10/1975 03:15	96.90%
1976	20/01/1977 21:30	92.60%			24/08/1977 10:00	15.80%	03/02/1977 09:00	63.30%
1977	23/02/1978 04:30	92.00%			22/02/1978 15:45	20.10%	22/02/1978 17:00	84.50%
1978	07/12/1978 23:45	86.20%			07/12/1978 00:15	20.40%	07/12/1978 02:00	91.30%
1979	05/12/1979 12:30	83.20%			26/12/1979 21:15	22.50%	27/12/1979 02:15	84.30%
1980	02/11/1980 21:15	90.70%			31/05/1981 02:30	24.80%	02/11/1980 12:30	97.90%
1981	21/02/1982 15:00	79.30%			13/12/1981 16:30	23.50%	21/02/1982 08:00	94.30%
1982	01/11/1982 08:45	58.00%			08/11/1982 06:45	27.10%	08/11/1982 08:00	90.20%
1983	26/01/1984 23:30	63.10%			16/12/1983 14:45	30.50%	16/12/1983 18:30	90.70%
1984	08/02/1985 13:45	17.50%	08/02/1985 05:30	28.90%	08/02/1985 05:15	20.10%	20/03/1985 19:30	97.60%
1985	01/12/1985 15:45	15.40%	06/08/1986 02:00	32.30%	01/12/1985 12:45	46.20%	06/08/1986 05:45	96.50%
1986	13/12/1986 01:30	17.70%	08/12/1986 15:30	25.30%	12/12/1986 21:00	47.50%	12/12/1986 22:30	100.00%
1987	13/01/1988 00:30	19.80%	18/01/1988 23:45	24.50%	18/01/1988 23:30	57.60%	19/01/1988 01:00	100.30%
1988	21/10/1988 13:45	17.50%	14/10/1988 06:00	15.90%	11/10/1988 09:30	42.00%	21/10/1988 22:15	98.10%
1989	17/12/1989 06:00	15.60%	06/02/1990 08:00	12.50%	06/02/1990 10:00	42.10%	06/02/1990 13:00	98.00%
1990	05/01/1991 01:00	10.90%	01/01/1991 11:00	9.00%	07/03/1991 04:00	38.80%	01/01/1991 18:00	84.70%
1991	25/11/1991 06:45	14.90%	25/11/1991 05:15	14.10%	24/11/1991 00:30	19.50%	25/11/1991 04:00	85.00%
1992	18/12/1992 00:45	5.50%	19/09/1993 09:15	74.50%	19/09/1993 22:15	36.40%	19/09/1993 06:00	41.20%
1993	22/02/1994 22:00	13.60%	22/02/1994 16:45	88.70%	26/02/1994 23:15	43.00%	22/02/1994 15:00	27.80%
1994	10/03/1995 11:45	21.20%	09/03/1995 21:15	87.60%	10/03/1995 00:30	35.30%	09/03/1995 21:45	25.70%
1995	14/01/1996 22:15	21.00%	14/01/1996 13:00	86.30%	08/01/1996 19:00	89.10%	08/01/1996 19:00	21.80%
1996	31/08/1997 22:30	27.00%	27/08/1997 00:15	73.90%	28/10/1996 20:45	95.80%	27/08/1997 03:00	23.60%
1997	08/01/1998 16:30	29.30%	17/11/1997 17:00	90.50%	18/11/1997 04:00	88.90%	17/11/1997 21:30	32.20%
1998	29/12/1998 14:45	58.30%	29/12/1998 13:00	95.70%	18/09/1999 15:30	93.80%	18/09/1999 15:45	60.10%
1999	21/12/1999 03:45	63.30%	24/12/1999 10:00	92.00%	24/12/1999 16:15	43.10%	24/12/1999 20:45	40.00%
2000	17/10/2000 23:45	3.90%	02/11/2000 17:15	46.10%			05/11/2000 15:30	89.10%
2001	22/05/2002 03:30	29.30%	01/02/2002 12:15	89.80%			01/02/2002 14:00	73.00%
2002	10/06/2003 00:15	23.20%	02/10/2002 13:15	44.40%			21/11/2002 06:00	74.60%
2003	22/08/2004 21:30	52.70%	03/02/2004 11:00	47.80%			26/12/2003 09:45	10.30%
2004	08/01/2005 08:15	100.00%	24/07/2005 05:15	25.60%			24/10/2004 06:15	86.10%
2005	03/11/2005 15:45	92.80%	03/11/2005 00:30	92.80%			03/11/2005 01:00	92.70%
2006	03/12/2006 16:45	26.20%	03/12/2006 03:15	26.20%			03/12/2006 05:00	26.20%
2007	10/01/2008 05:45	67.90%	10/01/2008 07:15	72.70%			10/01/2008 10:45	72.30%
2008	31/08/2009 11:15	62.70%	11/07/2009 21:00	62.20%				
2009	19/11/2009 21:30	95.10%	19/11/2009 15:45	90.70%				
2010	17/11/2010 10:15	45.70%	17/11/2010 01:45	32.50%			18/02/2011 20:15	20.60%
2011	28/06/2012 19:15	28.70%	28/06/2012 05:15	33.40%			28/06/2012 07:30	28.70%

Data Availability:

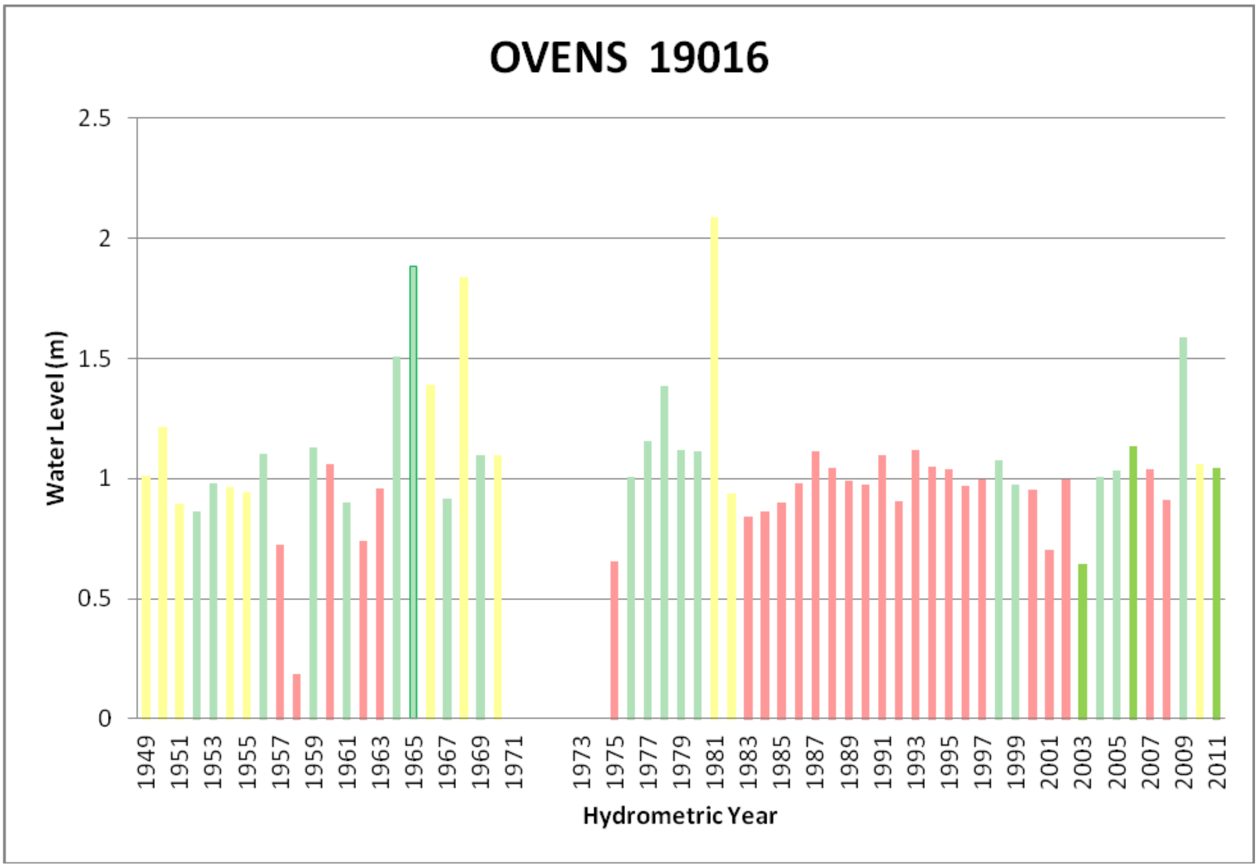


Results:

HY	Recorded Max	Missing Data	Notes
1949	26/10/1949 05:15	01/10/1949 - 22/10/1949 18/02/1950 - 11/03/1950 , 11/04/1950-23/04/1950, 19/05/1950 - 04/06/1950	>80%, Same as Lee Dromcarra, but may be effected by missing data
1950	11/01/1951 18:45	06/10/1950 - 16/11/1950	>80%, Same as Lee Dromcarra, but may be effected by missing data
1951	01/10/1951 00:00	14/12/1951 -01/01/1952, 01/03/1952 - 24/07/1952	Missing data for adjacent station event
1952	28/10/1952 11:45	21/02/1953 - 14/03/1953	>80%, Similiar Events
1953	04/12/1953 06:45		>80%, Similiar Events
1954	30/11/1954 07:15	01/10/1954-11/10/1954, 18/12/1954 - 01/01/1955, 12/02/1955 - 26/02/1955, 30/07/1955 - 13/08/1955	>80%, Different event, May be affected by missing data
1955	13/12/1955 11:30	31/03/1956 - 07/04/1956	Missing data for adjacent station event
1956	25/09/1957 17:00	04/05/1957 - 17/06/1957, 31/08/1957 - 22/09/1957	>80%, Similiar Events
1957	23/12/1957 13:00	Significant Missing Data	Insignificant Data Coverage
1958	26/09/1959 11:00	Significant Missing Data	Insignificant Data Coverage
1959	01/01/1960 06:00	22/04/1960 - 30/04/1960	Good Confidence in Amax
1960	25/01/1961 16:15	Significant Missing Data	Insignificant Data Coverage
1961	16/01/1962 06:00	07/10/1961 - 15/10/1961, 04/11/1961 -12/11/1961, 07/12/1961 -16/12/1961, 24/02/1962 - 06/03/1962,09/03/1962 -24/03/1962, 31/05/1962-18/06/1962, 07/07/1962 - 17/07/1962	Numerous gaps in data but not long enough to effect peak
1962	05/11/1962 02:45	Significant Missing Data	Insignificant Data Coverage
1963	17/08/1964 08:30	Significant Missing Data	Insignificant Data Coverage
1964	13/12/1964 19:15	16/10-1964 - 18/10/1964 , 30/01/1965 - 07/02/1965, 25/06/1965 -03/07/1965	>80%, Similiar Events
1965	15/02/1966 20:15	02/11/1965 - 14/11/1965 , 19/03/1966 -23/03/1966	Good Confidence in Amax
1966	28/02/1967 00:45	08/10/1966 -13/10/1966, 03/12/1966 - 27/12/1966, 22/04/1967 - 03/05/1967, 03/06/1967 -27/06/1967, 08/07/1967 - 22/07/1967	Missing weeks in record, ,may affect Amax
1967	16/01/1968 13:00	21/08/1968 - 27/08/1968	>80%, Different event. Good Confidence in Peak
1968	20/01/1969 23:30	No Data after 08/07/1969	No Data after 08/07/1969 (Missing Summer months)
1969	21/01/1970 16:30	07/07/1970 - 24/07/1970	>80%, Similiar Events
1970	24/11/1970 00:00	03/04/1971 - 03/04/1971	Missing 5 months of Data (Summer Months)
1971		No Data	No Data

	Recorded Max	Missing Data	Notes
1972			
1973			
1974			
1975	17/05/1976 19:45	01/10/1975 - 15/05/1975	Only Data for Summer Months
1976	20/01/1977 21:30	08/01/1977 - 11/01/1977 , 14/05/1977 - 18/05/1977, 09/07/1977 - 14/07/1977	Peaks not effected by missing data
1977	23/02/1978 04:30	21/01/1978 - 29/01/1978, 29/04/1978 - 06/05/1978, 26/08/1971 - 04/07/1978	Peaks not effected by missing data
1978	07/12/1978 23:45	13/04/1974 - 24/04/1974, 14/07/1979 - 22/07/1979	Peaks not effected by missing data
1979	05/12/1979 12:30	05/04/1980 - 26/04/1980, 26/06/1980 - 08/07/1980, 19/07/1980 - 29/07/1980	Peaks not effected by missing data
1980	02/11/1980 21:15	14/11/1980 - 08/12/1980, 03/02/1981 - 18/02/1981	Peaks not effected by missing data
1981	21/02/1982 15:00	06/11/1981 - 15/11/1981, 26/12/1981-06/02/1982, 02/07/1981 -21/07/1981	Similiar Peak to adjacent stations however significant data shortages in winter
1982	01/11/1982 08:45	29/04/1983 - 30/09/1983	Missing Data from April to September
1983	26/01/1984 23:30	01/10/1983 - 24/12/1983,	Missing three months of data, Missing data for peaks of adjacent stations
1984	08/02/1985 13:45	Significant Missing Data	Scattered Yearly Data
1985	01/12/1985 15:45	Significant Missing Data	Scattered Yearly Data
1986	13/12/1986 01:30	Significant Missing Data	Scattered Yearly Data
1987	13/01/1988 00:30	Significant Missing Data	Scattered Yearly Data
1988	21/10/1988 13:45	Significant Missing Data	Scattered Yearly Data
1989	17/12/1989 06:00	Significant Missing Data	Scattered Yearly Data
1990	05/01/1991 01:00	Significant Missing Data	Scattered Yearly Data
1991	25/11/1991 06:45	Significant Missing Data	Scattered Yearly Data
1992	18/12/1992 00:45	Significant Missing Data	Scattered Yearly Data
1993	22/02/1994 22:00	Significant Missing Data	Scattered Yearly Data
1994	10/03/1995 11:45	Significant Missing Data	Scattered Yearly Data
1995	14/01/1996 22:15	Significant Missing Data	Scattered Yearly Data
1996	31/08/1997 22:30	Significant Missing Data	Scattered Yearly Data
1997	08/01/1998 16:30	Significant Missing Data	Scattered Yearly Data
1998	29/12/1998 14:45	28/03/1999 -07/05/1999, 29/05/1999 - 09/06/1999, 20/07/1999 - 23/08/1999	Appears Peaks do not effected missing data
1999	21/12/1999 03:45	10/01/2000 - 22/01/2000, 14/02/2000 - 24/03/2000	Appears Peaks do not effected missing data
2000	17/10/2000 23:45	17/10/2000 - 30/09/2001	Missing almost complete year's data
2001	22/05/2002 03:30	01/10/2001 - 20/02/2002, 06/06/2002 - 30/09/2002	Significant Missing Data
2002	10/06/2003 00:15	01/10/2002 - 26/05/2005, 18/08/2003 - 30/09/2003	Missing almost complete year's data
2003	22/08/2004 21:30	01/10/2003 - 22/03/2007	Missing Winter Months
2004	08/01/2005 08:15		100% coverage
2005	03/11/2005 15:45	15/02/2006 - 17/03/2006	Missing a month in February however aligns with adjacent peaks
2006	03/12/2006 16:45	03/01/2007 - 30/09/2007	Only three months of data
2007	10/01/2008 05:45	01/10/2007 - 27/11/2007, 31/07/2008 - 30/09/2008	Missing October, November, August and September
2008	31/08/2009 11:15	01/10/2008 - 14/02/2009	Missing Winter Data
2009	19/11/2009 21:30		Known Flood Event, Good Data Coverage
2010	17/11/2010 10:15	16/03/2011 - 30/09/2011	Missing Summer Months
2011	28/06/2012 19:15	1/10/2011 - 18/04/2012, 01/08/2012 -30/09/2012	Missing winter Months

Results:



Results and Conclusions:

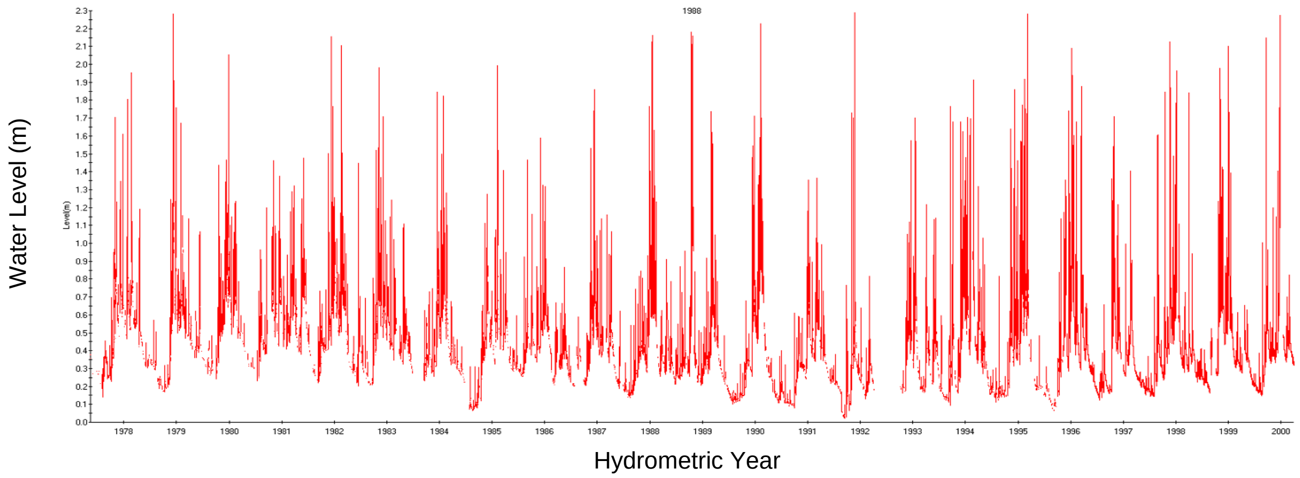
The graph above presents the confidence that has been determined from the available data for 19016 based on the analysis above (comparison with nearby stations and a data availability check). Data is available from 1949 to present day. Of the 62 years of data confidence in the capturing of the Amax is only available for 22 Hydrometric Years. The standard of recordings are particularly poor between 1983 and 1997. For these years the data availability suffers from the data recorder being very inconsistent, where it is possible to analyse base flow when the recorder is recording intermittently but it may be possible to miss intense events. The standard of recordings improved in the past years however, there is still signifcant gaps in data availablity.

Station 19018		Tower
Location:	159003 , 74757	
Comparison Stations:	19017- Bawnnafinny (Approximately 1.7km away)	
	19015 - Healy's Bridge (Approximately 2.5km away)	
	19045 - Gothic (Approximately 2.5km away)	
Data Availability:	1976 - 1999	Amax and Water Level Data

Amax Comparison with Nearby Stations:

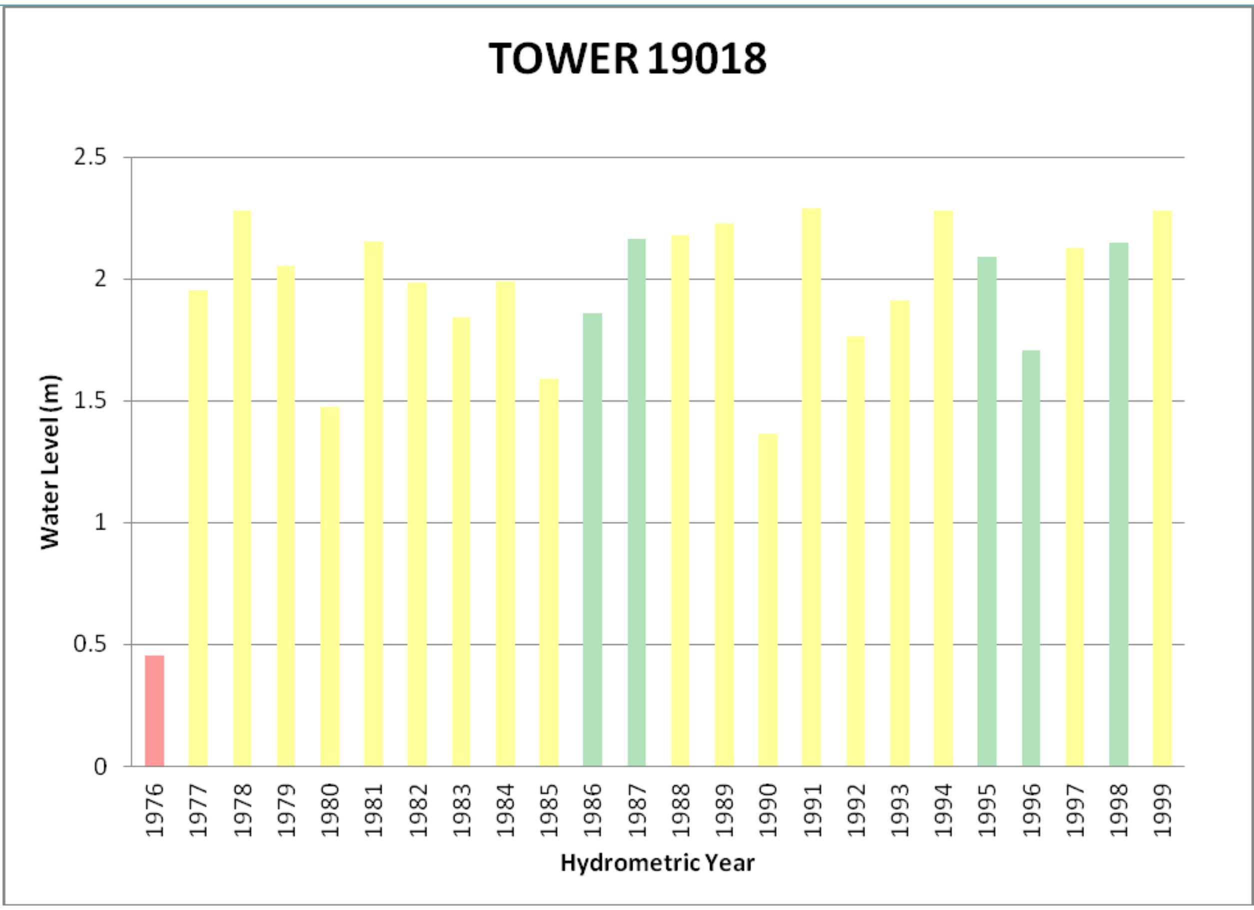
	TOWER 19018	% Data	BAWNNAFINNY	% Data	HEALY'S BRIDGE	% Data	GOTHIC	% Data
1976	24/08/1977 10:00	15.80%	15/05/1977 11:45	3.70%	03/02/1977 09:00	63.30%		
1977	22/02/1978 15:45	20.10%	23/12/1977 03:15	24.20%	22/02/1978 17:00	84.50%		
1978	07/12/1978 00:15	20.40%	07/12/1978 14:15	19.50%	07/12/1978 02:00	91.30%		
1979	26/12/1979 21:15	22.50%	22/10/1979 18:30	28.00%	27/12/1979 02:15	84.30%		
1980	31/05/1981 02:30	24.80%	31/05/1981 01:30	30.80%	02/11/1980 12:30	97.90%		
1981	13/12/1981 16:30	23.50%	21/02/1982 00:00	22.30%	21/02/1982 08:00	94.30%		
1982	08/11/1982 06:45	27.10%	08/11/1982 05:45	25.20%	08/11/1982 08:00	90.20%		
1983	16/12/1983 14:45	30.50%	26/01/1984 08:45	19.60%	16/12/1983 18:30	90.70%		
1984	08/02/1985 05:15	20.10%	08/02/1985 05:00	20.40%	20/03/1985 19:30	97.60%		
1985	01/12/1985 12:45	46.20%	06/08/1986 02:45	78.70%	06/08/1986 05:45	96.50%		
1986	12/12/1986 21:00	47.50%	12/12/1986 20:45	99.70%	12/12/1986 22:30	100.00%		
1987	18/01/1988 23:30	57.60%	12/01/1988 18:15	99.50%	19/01/1988 01:00	100.30%		
1988	11/10/1988 09:30	42.00%	09/03/1989 19:45	92.00%	21/10/1988 22:15	98.10%		
1989	06/02/1990 10:00	42.10%	05/02/1990 13:00	100.00%	06/02/1990 13:00	98.00%		
1990	07/03/1991 04:00	38.80%	01/01/1991 20:30	98.00%	01/01/1991 18:00	84.70%		
1991	24/11/1991 00:30	19.50%	24/04/1992 03:45	99.70%	25/11/1991 04:00	85.00%		
1992	19/09/1993 22:15	36.40%	10/06/1993 04:30	40.40%	19/09/1993 06:00	41.20%		
1993	26/02/1994 23:15	43.00%	03/05/1994 05:45	16.40%	22/02/1994 15:00	27.80%		
1994	10/03/1995 00:30	35.30%			09/03/1995 21:45	25.70%	02/03/1995 19:30	87.20%
1995	08/01/1996 19:00	89.10%			08/01/1996 19:00	21.80%	08/01/1996 18:00	100.30%
1996	28/10/1996 20:45	95.80%			27/08/1997 03:00	23.60%	25/10/1996 20:45	100.00%
1997	18/11/1997 04:00	88.90%			17/11/1997 21:30	32.20%	08/01/1998 20:30	99.50%
1998	18/09/1999 15:30	93.80%			18/09/1999 15:45	60.10%	01/11/1998 05:15	9.20%
1999	24/12/1999 16:15	43.10%			24/12/1999 20:45	40.00%		

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1976	24/08/1977 10:00	01/10/1976 - 08/08/1977	
1977	22/02/1978 15:45	Scattered data throughout	Scattered data throughout, Some Similiar Peaks
1978	07/12/1978 00:15	Scattered data throughout	Scattered data throughout, Similiar Peaks
1979	26/12/1979 21:15	Scattered data throughout	Scattered data throughout, Similiar Peaks
1980	31/05/1981 02:30	Scattered data throughout	Scattered data throughout, Similiar Peaks
1981	13/12/1981 16:30	Scattered data throughout	Scattered data throughout, Different to adjacent peaks
1982	08/11/1982 06:45	05/03/1983-29/03/1983, 26/06/1983-18/09/1983	No Summer data, scattered winter data
1983	16/12/1983 14:45	Scattered data throughout	Scattered data throughout, Different to adjacent peaks
1984	08/02/1985 05:15	Scattered data throughout	Scattered data throughout, Different to adjacent peaks
1985	01/12/1985 12:45	Scattered data throughout	Missing data for known flood event in the area
1986	12/12/1986 21:00	Scattered data throughout	Equal to nearby station with good water level coverage
1987	18/01/1988 23:30	Scattered data throughout	Though data scattered, baseflow can be analysed, unless isolated intense good confidence in peaks
1988	11/10/1988 09:30	27/10/1988-03/11/1989	Missing October, Scattered summed months, similiar to adjacent peaks
1989	06/02/1990 10:00	Scattered data throughout	Appears to me peak, same as adjacent stations, may of missed intense event
1990	07/03/1991 04:00	Scattered data throughout	Appears to me peak, same as adjacent stations, may of missed intense event
1991	24/11/1991 00:30	06/04/1994-30/09/1991	No Data for 'Bawwnafinny Event'
1992	19/09/1993 22:15	Scattered data throughout	Scattered data throughout, Similiar Peaks
1993	26/02/1994 23:15	Scattered data throughout	Scattered data throughout, Some alignment in adjacent peaks
1994	10/03/1995 00:30	Scattered data throughout	Scattered data throughout, Similiar Peaks
1995	08/01/1996 19:00		Good Data Coverage, some scattered data during summer months
1996	28/10/1996 20:45	16/12/1996 - 01/01/1996	
1997	18/11/1997 04:00	08/01/1998 - 19/01/1998, 25/08/1998-01/09/1998	
1998	18/09/1999 15:30	01/10/1998-06/10/1998, 08/04/1999-21/04/1999	
1999	24/12/1999 16:15	24/12/1999-18/01/2000, 30/03/2000-30/09/2000	Recorder cuts on peak of event

Results:



Results and Conclusions:

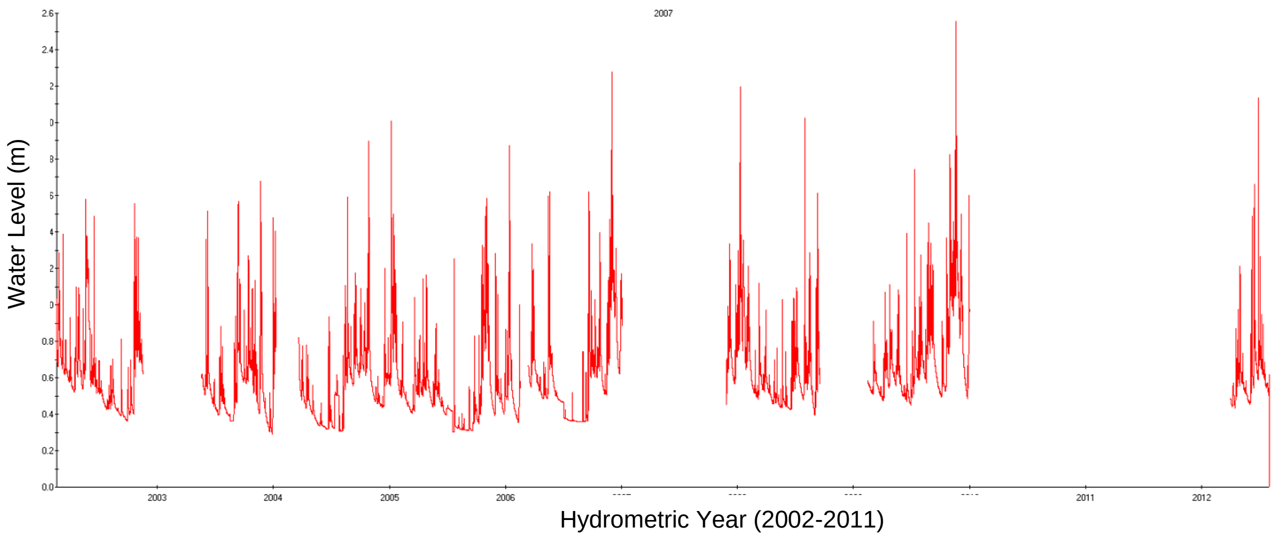
The graph above presents the confidence that has been determined from the available data for 19018 based on the analysis above (comparison with nearby stations and a data availability check). Recorders were put in place in 1976 and removed in 1999, with continuous 15 minute interval water level data available throughout. Of the 23 years of data confidence in the capturing of the Amax is only available for five Hydrometric Years. The record suffers from the data recorder being very inconsistent up to 1994, where it is possible to analyse base flow when the recorder is recording intermittently but it may be possible to miss intense events. The recorder cut during the peak of the 19/11/2009 event and was never repaired.

Station 19027		Kill Laney
Location:	**136455 , 74301**	
Comparison Stations	**19031 - Macroom -Sullane (Approximately 2.5km away)**	
	19028 - Dripsey (Approximately 12km away)	
	19014 - Lee Dromcarra (Approximately 9.6km away)	
Data Availability	**1984 - 2001**	**Amax data, No supporting water level data**
	2002 - 2009	**Amax and Water Level Data**
	2010	**No Data**
	2011	**Amax and Water Level Data**

Amax Comparison with Nearby Stations:

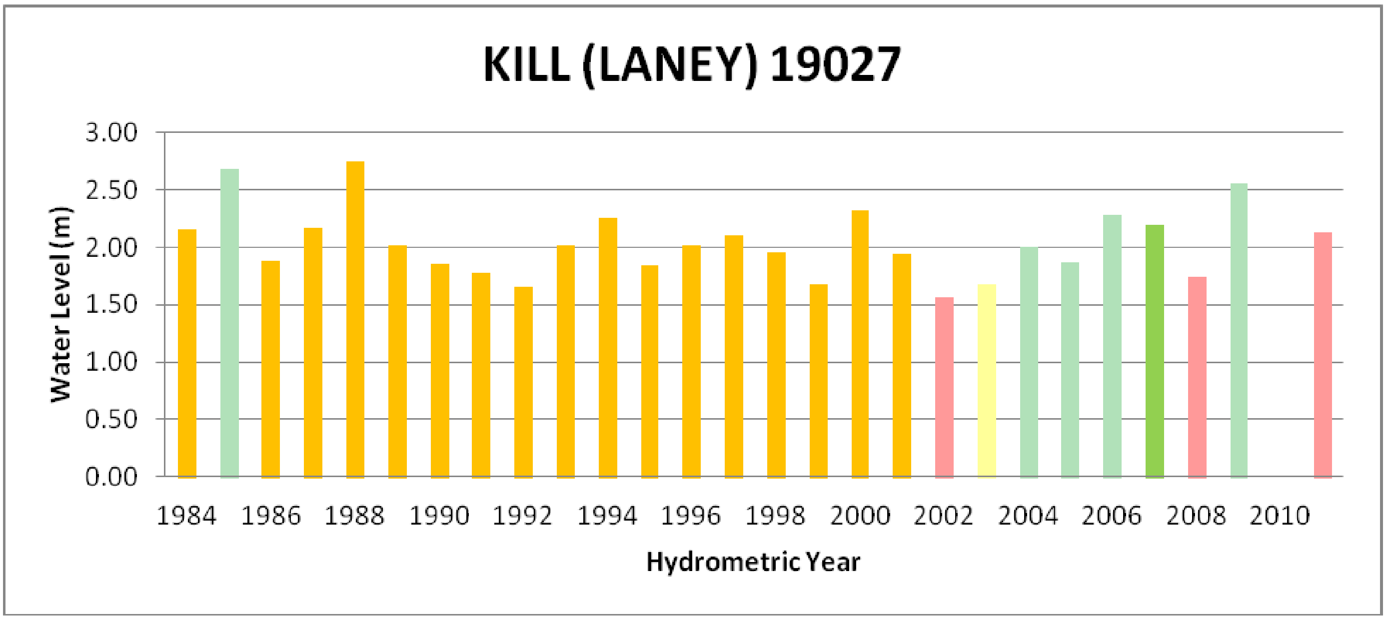
HY	KILL 19027		%Data	DRIPSEY 19028	%Data	MACROOM 19031	%Data	LEE DROMCARRA	%Data
1984	2.15	**08/02/1985 05:00**		08/02/1985 05:30	29%	29/11/1984 11:00		13/11/1984 00:00	
1985	2.68	**06/08/1986 00:30**		06/08/1986 02:00	32%	21/12/1985 13:00		06/08/1986 13:00	
1986	1.89	**08/12/1986 15:30**		08/12/1986 15:30	25%	12/12/1986 22:00		27/03/1987 14:15	
1987	2.17	**12/01/1988 16:30**		18/01/1988 23:45	25%	28/12/1987 23:00			
1988	2.75	**14/10/1988 03:00**		14/10/1988 06:00	16%	09/03/1989 21:00		14/03/1989 04:00	
1989	2.02	**06/02/1990 09:30**		06/02/1990 08:00	13%	04/02/1990 18:30		06/02/1990 17:00	
1990	1.85	**01/01/1991 17:30**		01/01/1991 11:00	9%	02/10/1990 21:00		02/01/1991 01:50	
1991	1.78	**25/11/1991 00:30**		25/11/1991 05:15	14%			24/04/1992 02:00	
1992	1.66	**15/01/1993 05:30**		19/09/1993 09:15	75%			15/01/1993 07:00	
1993	2.02	**22/02/1994 18:00**		22/02/1994 16:45	89%			22/02/1994 13:00	
1994	2.25	**09/03/1995 22:00**		09/03/1995 21:15	88%			27/01/1995 15:00	
1995	1.84	**21/11/1995 03:00**		14/01/1996 13:00	86%			16/10/1995 23:00	
1996	2.02	**27/08/1997 01:00**		27/08/1997 00:15	74%				
1997	2.10	**17/11/1997 18:00**		17/11/1997 17:00	91%				
1998	1.95	**29/12/1998 12:00**		29/12/1998 13:00	96%				
1999	1.68	**22/12/1999 06:30**		24/12/1999 10:00	92%				
2000	2.32	**30/11/2000 01:30**		02/11/2000 17:15	46%	21/08/2001 16:45	12%	21/08/2001 16:15	12%
2001	1.94	**03/12/2001 18:30**		01/02/2002 12:15	90%	03/12/2001 21:45	77%	01/02/2002 12:15	100%
2002	1.566	**14/09/2003 12:45**	**50%**	02/10/2002 13:15	44%	11/09/2003 23:45	58%	02/11/2002 17:15	13%
2003	1.678	**23/11/2003 20:15**	**81%**	03/02/2004 11:00	48%	22/08/2004 16:30	54%	31/08/2004 16:00	52%
2004	2.007	**08/01/2005 00:15**	**100%**	24/07/2005 05:15	26%	04/10/2004 06:15	5%	24/07/2005 14:15	27%
2005	1.873	**13/01/2006 06:45**	**93%**	03/11/2005 00:30	93%			13/01/2006 08:00	**93%**
2006	2.278	**03/12/2006 02:00**	**26%**	03/12/2006 03:15	26%			03/12/2006 02:15	26%
2007	2.194	**10/01/2008 06:30**	**81%**	10/01/2008 07:15	73%			10/01/2008 08:30	81%
2008	1.741	**11/07/2009 18:30**	**63%**	11/07/2009 21:00	62%			08/09/2009 15:30	63%
2009	2.555	**19/11/2009 15:00**	**25%**	19/11/2009 15:45	91%			19/11/2009 14:00	98%
2010				17/11/2010 01:45	33%			16/01/2011 01:30	32%
2011	2.136	**28/06/2012 04:30**	**33%**	28/06/2012 05:15	33%	07/06/2012 22:45	24%	07/06/2012 20:30	33%

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1984	08/02/1985 05:00		Record appears complete
1985	06/08/1986 00:30		Known Flood Event in Area
1986	08/12/1986 15:30		Record appears complete
1987	12/01/1988 16:30		Record appears complete
1988	14/10/1988 03:00		Record appears complete
1989	06/02/1990 09:30		Record appears complete
1990	01/01/1991 17:30	28/09/1991-30/09/1991	Only one months of data
1991	25/11/1991 00:30	01/10/1991-08/11/1991	Missing October and November
1992	15/01/1993 05:30		Record appears complete
1993	22/02/1994 18:00		Record appears complete
1994	09/03/1995 22:00		Record appears complete
1995	21/11/1995 03:00	06/10/1995-11/10/1995	Missing One week in October
1996	27/08/1997 01:00		Record appears complete
1997	17/11/1997 18:00		Record appears complete
1998	29/12/1998 12:00	01/09/1998-18/09/1998	2 Weeks missing in October
1999	22/12/1999 06:30		Record appears complete
2000	30/11/2000 01:30		Record appears complete
2001	03/12/2001 18:30	01/10/2001 - 19/02/2002	Record appears complete
2002	14/09/2003 12:45	18/11/2002 - 21/05/2003	Missing Winter Months
2003	23/11/2003 20:15	11/01/2004-22/03/2004	Missing January- March
2004	08/01/2005 00:15		Complete Data Set
2005	13/01/2006 06:45	16/02/2006-15/03/2006	Missing on Months data
2006	03/12/2006 02:00	04/01/2007-30/09/2007	Known Flood Event in Area
2007	10/01/2008 06:30	01/10/2007-27/11/2007, 17/09/2008-30/09/2008	Missing October to November, good peak alignment with adjacent stations
2008	11/07/2009 18:30	01/10/2007-15/02/2008	Missing Winter Data
2009	19/11/2009 15:00	31/12/2009-30/09/2010	Known Flood Event in Area
2010			No Data
2011	28/06/2012 04:30	01/10/2012 - 01/04/2012, 31/07/2012 - 30/09/2012	Only 3 months of data

Results:



Results and Conclusions:

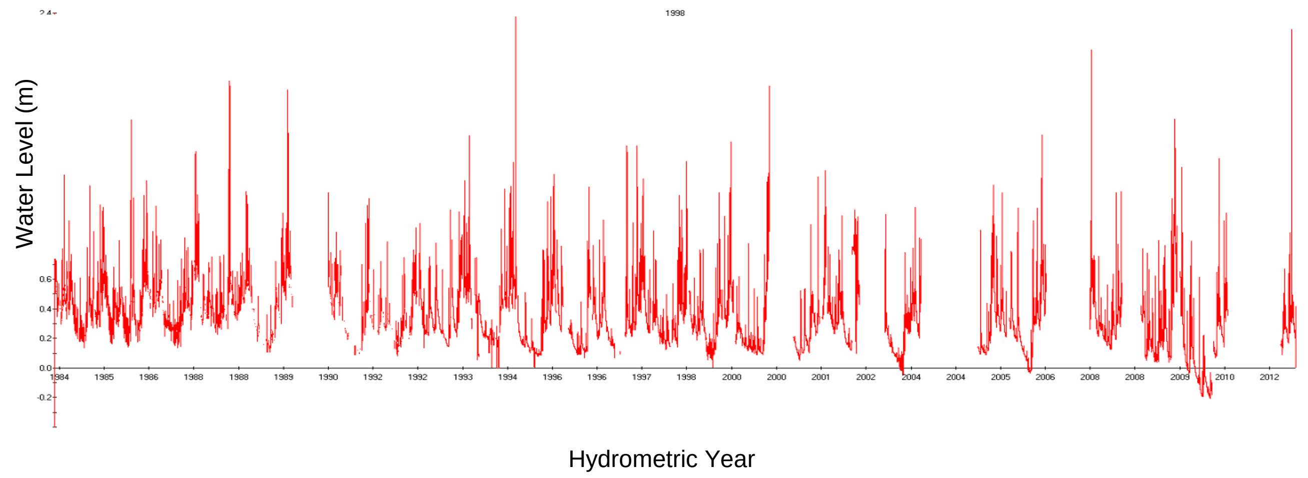
The graph above presents the confidence that has been determined from the available data for 19027 based on the analysis above (comparison with nearby stations and a data availability check). Recorders have been in place since 1984, with continuous 15 minute interval water level data available from 2002. Of the 27 years of data confidence in the capturing of the Amax is only available for six Hydrometric Years. Based on the Amax data available from ESB from 1984 -2001, 17 years of data are available but only one year, 1985 (Hurricane Charlie Event) can there be significant confidence the actual Amax has been captured due to the lack of supporting data.

Station 19028	Dripsey	
Location:	148463 , 74959	
Comparison Stations	19031 - Macroom (Sullane) (Approximately 13.8km away)	
	19016 - Ovens (Approximately 18.9km away)	
	19027 - Kill (Laney) (Approximately 12 km away)	
Data Availability	1984 - 2011	Amax and Water Level Data

Amax Comparison with Nearby Stations:

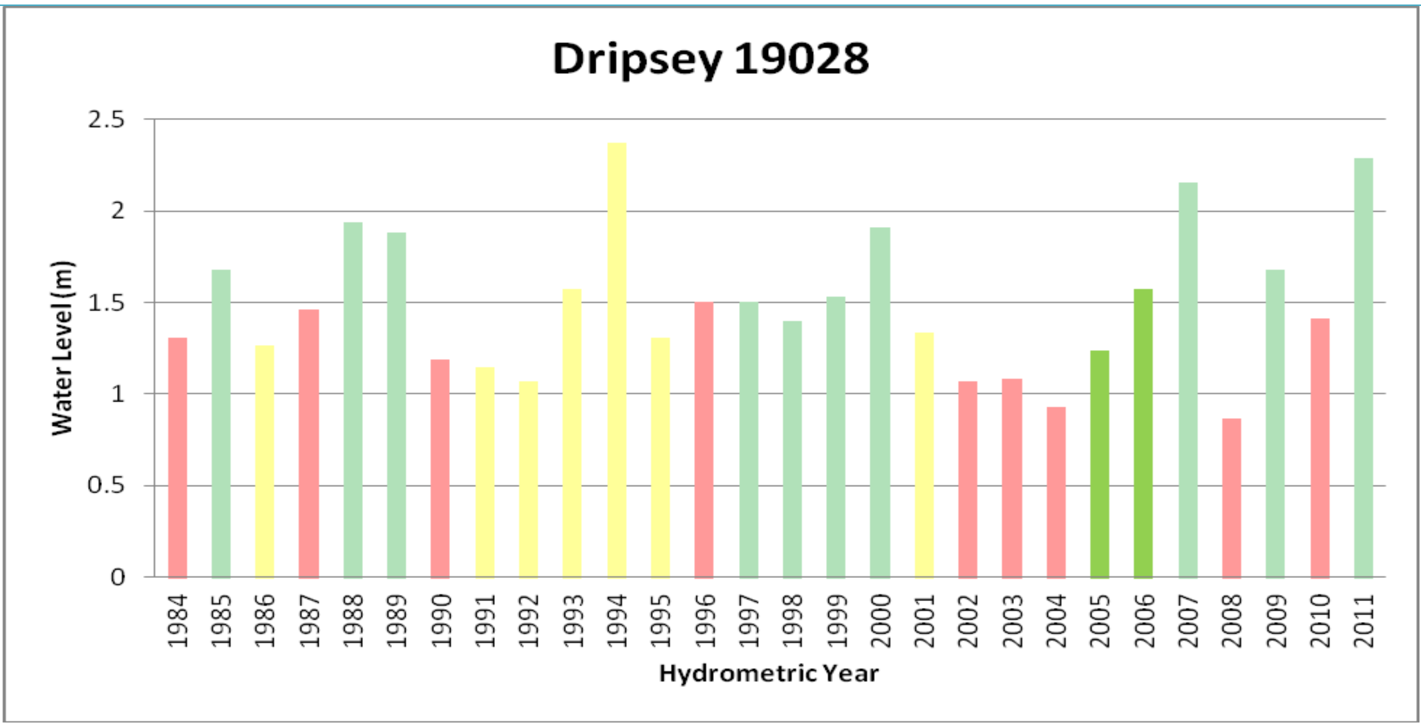
HY	DRIPSEY 19028	%Data	KILL 19027	%Data	MACROOM 19031	%Data	OVENS 19016	%Data
1984	**08/02/1985 05:30**	**29%**	08/02/1985 05:00		29/11/1984 11:00		08/02/1985 13:45	18%
1985	**06/08/1986 02:00**	**32%**	06/08/1986 00:30		21/12/1985 13:00		01/12/1985 15:45	15%
1986	**08/12/1986 15:30**	**25%**	08/12/1986 15:30		12/12/1986 22:00		13/12/1986 01:30	18%
1987	**18/01/1988 23:45**	**25%**	12/01/1988 16:30		28/12/1987 23:00		13/01/1988 00:30	20%
1988	**14/10/1988 06:00**	**16%**	14/10/1988 03:00		09/03/1989 21:00		21/10/1988 13:45	18%
1989	**06/02/1990 08:00**	**13%**	06/02/1990 09:30		04/02/1990 18:30		17/12/1989 06:00	16%
1990	**01/01/1991 11:00**	**9%**	01/01/1991 17:30		02/10/1990 21:00		05/01/1991 01:00	11%
1991	**25/11/1991 05:15**	**14%**	25/11/1991 00:30				25/11/1991 06:45	15%
1992	**19/09/1993 09:15**	**75%**	15/01/1993 05:30				18/12/1992 00:45	6%
1993	**22/02/1994 16:45**	**89%**	22/02/1994 18:00				22/02/1994 22:00	14%
1994	**09/03/1995 21:15**	**88%**	09/03/1995 22:00				10/03/1995 11:45	21%
1995	**14/01/1996 13:00**	**86%**	21/11/1995 03:00				14/01/1996 22:15	21%
1996	**27/08/1997 00:15**	**74%**	27/08/1997 01:00				31/08/1997 22:30	27%
1997	**17/11/1997 17:00**	**91%**	17/11/1997 18:00				08/01/1998 16:30	29%
1998	**29/12/1998 13:00**	**96%**	29/12/1998 12:00				29/12/1998 14:45	58%
1999	**24/12/1999 10:00**	**92%**	22/12/1999 06:30				21/12/1999 03:45	63%
2000	**02/11/2000 17:15**	**46%**	30/11/2000 01:30		21/08/2001 16:45	12%	17/10/2000 23:45	4%
2001	**01/02/2002 12:15**	**90%**	03/12/2001 18:30		03/12/2001 21:45	77%	22/05/2002 03:30	29%
2002	**02/10/2002 13:15**	**44%**	14/09/2003 12:45	50%	11/09/2003 23:45	58%	10/06/2003 00:15	23%
2003	**03/02/2004 11:00**	**48%**	23/11/2003 20:15	81%	22/08/2004 16:30	54%	22/08/2004 21:30	53%
2004	**24/07/2005 05:15**	**26%**	08/01/2005 00:15	100%	04/10/2004 06:15	5%	08/01/2005 08:15	100%
2005	**03/11/2005 00:30**	**93%**	13/01/2006 06:45	93%			03/11/2005 15:45	93%
2006	**03/12/2006 03:15**	**26%**	03/12/2006 02:00	26%			03/12/2006 16:45	26%
2007	**10/01/2008 07:15**	**73%**	10/01/2008 06:30	81%			10/01/2008 05:45	68%
2008	**11/07/2009 21:00**	**62%**	11/07/2009 18:30	63%			31/08/2009 11:15	63%
2009	**19/11/2009 15:45**	**91%**	19/11/2009 15:00	25%			19/11/2009 21:30	95%
2010	**17/11/2010 01:45**	**33%**					17/11/2010 10:15	46%
2011	**28/06/2012 05:15**	**33%**	28/06/2012 04:30	33%	07/06/2012 22:45	24%	28/06/2012 19:15	29%

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1984	08/02/1985 05:30	Scattered Winter Data	
1985	06/08/1986 02:00	Scattered Data	Known Flood Event in the Area
1986	08/12/1986 15:30	Scattered Data, may affect the peak Amax	
1987	18/01/1988 23:45	Insignificant Data, Large gaps in Winter Data	
1988	14/10/1988 06:00	Significant Data Missing	
1989	06/02/1990 08:00	Scattered Winter Data	
1990	01/01/1991 11:00	15/03/1991-30/09/1991	Scattered Winter Data,
1991	25/11/1991 05:15	Scattered Data throughout Year	Appear to have captured peak same as adjacent stations
1992	19/09/1993 09:15	Scattered Data throughout Year	Different to adjacent stations however both Amax timings were recorded
1993	22/02/1994 16:45		Affected by changing datum
1994	09/03/1995 21:15	20/10/1994-28/10/1994, 25/11/1994-05/12/1994, 26/12/1994-06/01/1995,17/02/1995-24/02/1995	Missing Adjacent Site Peak recording
1995	14/01/1996 13:00	31/01/1996-09/02/1996, 29/03/1996-10/05/1996	
1996	27/08/1997 00:15	24/05/1997-15/08/1997	Missing summer months, Missing comparison site peaks
1997	17/11/1997 17:00	12/03/1998-31/03/1998, 10/08/1998-01/09/1999	
1998	29/12/1998 13:00	04/01/1999 - 20/01/1999	Same as comparison stations, good data coverage
1999	24/12/1999 10:00	18/10/1999-03/11/1999, 16/01/2000-31/01/2000	Same as comparison stations, good data coverage
2000	02/11/2000 17:15	03/11/2000-19/05/2001	Significant data missing
2001	01/02/2002 12:15	17/12/2001-08/01/2002, 06/02/2002-20/02/2002	Change of datum on 07/09/2002
2002	02/10/2002 13:15	10/11/2002-02/06/2003	Significant data missing
2003	03/02/2004 11:00	12/03/2004-30/09/2004	Negative readings in October, No Summer recordings, No alignment with adjacent peaks
2004	24/07/2005 05:15	01/10/2004-29/06/2005	Significant data missing
2005	03/11/2005 00:30	16/02/2006-15/03/2006	Negative recordings, Same as Ovens
2006	03/12/2006 03:15	01/01/2007-30/09/2007	Only 3 months of data, peak the same as adjacent peaks
2007	10/01/2008 07:15	01/10/2007-27/12/2007, 17/09/2005-30/09/2006	Insignificant Data
2008	11/07/2009 21:00	01/10/2008-15/02/2009	Missing Winter Data
2009	19/11/2009 15:45		Known Flood Event in the Area, Some negative recordings
2010	17/11/2010 01:45	01/10/2010-27/01/2010	No Winter data
2011	28/06/2012 05:15	01/10/20011-01/04/2012, 01/08/2012-30/09/2012	Insignificant Data

Results:



Results and Conclusions:

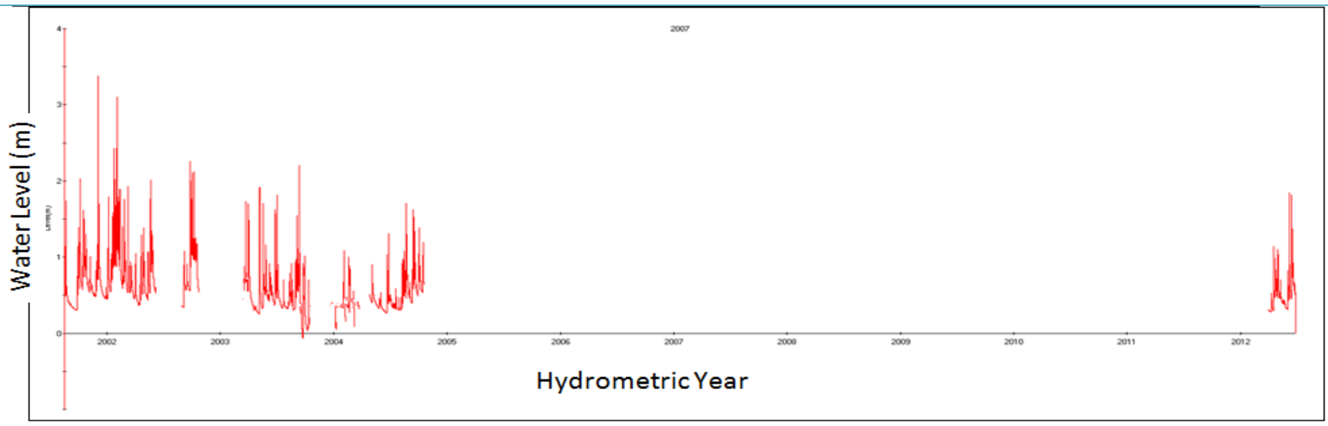
The graph above presents the confidence that has been determined from the available data for 19028 based on the analysis above (comparison with nearby stations and a data availability check). Recorders have been in place since 1984, with continuous 15 minute interval water level data available throughout. Of the 27 years of data confidence in the capturing of the Amax is only available for seven Hydrometric Years. The record suffers from the data recorder being very inconsistent up to 1992, where it is possible to analyse base flow when the recorder is inconsistent but it may be possible to miss intense events. Another issue found was there appears to be a change of datum, which adds to the uncertainity and this can be seen through negative recordings.

Station 19031		Macroom - Sullane
Location:	134743 , 73133	
Comparison Stations	19027 - Kill (Laney) (Approximately 2.5km away)	
	19014 - Lee Dromcarra (Approximately 7.5km away)	
Data Availability	1982 - 1990	Amax data, No supporting water level data
	1991 - 1999	No Data
	2000 - 2004	Amax and Water Level Data
	2005 - 2010	No Data
	2011	Amax and Water Level Data

Amax Comparison with Nearby Stations:

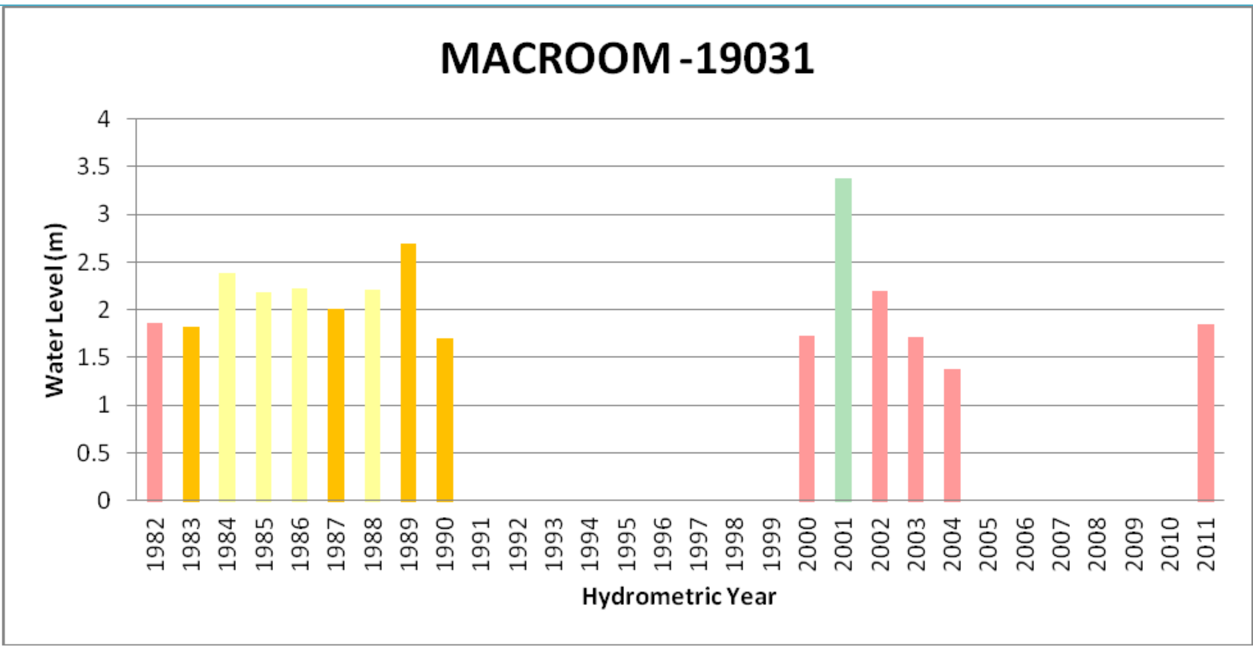
	Macroom -Sullane	% Data	Lee Dromcarra	% Data	Kill	% Data
1982	25/09/1983 02:00		16/10/1982 21:00			
1983	17/10/1983 12:00		15/12/1983 02:00			
1984	29/11/1984 11:00		13/11/1984 00:00		08/02/1985 05:00	
1985	21/12/1985 13:00		06/08/1986 13:00		06/08/1986 00:30	
1986	12/12/1986 22:00		27/03/1987 14:15		08/12/1986 15:30	
1987	28/12/1987 23:00				12/01/1988 16:30	
1988	09/03/1989 21:00		14/03/1989 04:00		14/10/1988 03:00	
1989	04/02/1990 18:30		06/02/1990 17:00		06/02/1990 09:30	
1990	02/10/1990 21:00		02/01/1991 01:50		01/01/1991 17:30	
1991			24/04/1992 02:00		25/11/1991 00:30	
1992			15/01/1993 07:00		15/01/1993 05:30	
1993			22/02/1994 13:00		22/02/1994 18:00	
1994			27/01/1995 15:00		09/03/1995 22:00	
1995			16/10/1995 23:00		21/11/1995 03:00	
1996					27/08/1997 01:00	
1997					17/11/1997 18:00	
1998					29/12/1998 12:00	
1999					22/12/1999 06:30	
2000	21/08/2001 16:45	12.40%	21/08/2001 16:15	12.40%	30/11/2000 01:30	
2001	03/12/2001 21:45	77.20%	01/02/2002 12:15	100.00%	03/12/2001 18:30	61.10%
2002	11/09/2003 23:45	57.50%	02/11/2002 17:15	13.40%	14/09/2003 12:45	49.60%
2003	22/08/2004 16:30	54.40%	31/08/2004 16:00	52.40%	23/11/2003 20:15	80.70%
2004	04/10/2004 06:15	4.80%	24/07/2005 14:15	26.90%	08/01/2005 00:15	100.00%
2005			13/01/2006 08:00	92.80%	13/01/2006 06:45	92.80%
2006			03/12/2006 02:15	26.20%	03/12/2006 02:00	26.20%
2007			10/01/2008 08:30	81.00%	10/01/2008 06:30	80.70%
2008			08/09/2009 15:30	62.50%	11/07/2009 18:30	62.50%
2009			19/11/2009 14:00	97.50%	19/11/2009 15:00	25.20%
2010			16/01/2011 01:30	32.40%		
2011	07/06/2012 22:45	23.80%	07/06/2012 20:30	33.00%	28/06/2012 04:30	33.20%

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
1982	25/09/1983 02:00	01/10/1982-23/06/1983	Only Summer Months Data
1983	17/10/1983 12:00		Appears Complete Record
1984	29/11/1984 11:00	21/11/1984-23/11/1984, 24/03/1985-07/04/1985	2 weeks in November, 2 weeks in March/April Missing
1985	21/12/1985 13:00	10/03/1986-24/03/1986, 09/08/1986-12/09/1986	Approximately six weeks of data missing
1986	12/12/1986 22:00	07/10/1986-20/11/1986	October- November Data Missing
1987	28/12/1987 23:00	29/09/1988-30/09/1988	Only one day missing
1988	09/03/1989 21:00	01/10/1988-10/10/1988	10 days missing in October
1989	04/02/1990 18:30		Appears complete record
1990	02/10/1990 21:00		Appears Complete record
1991			
1992			
1993			
1994			
1995			
1996			
1997			
1998			
1999			
2000	21/08/2001 16:45	01/10/2000-24/08/2001	One Month of Data
2001	03/12/2001 21:45	08/06-31/08/2002	No Summer Data, Known Flood Event
2002	21/08/2001 16:45	25/10/2002-23/03/2010	No Winter Months
2003	22/08/2004 16:30	17/10/2003-28/04/2004	Majority of Data missing from 17/10/2003 - 28/04/2004 (No Winter Months)
2004	04/10/2004 06:15		Only 16 days of Data
2005			
2006			
2007			
2008			
2009			
2010			
2011	07/06/2012 22:45	01/10/2011-04/04/2012, 27/06/2012-27/06/2012	Data only available from 04/04/2012 -27/06/2012

Results:



Results and Conclusions:

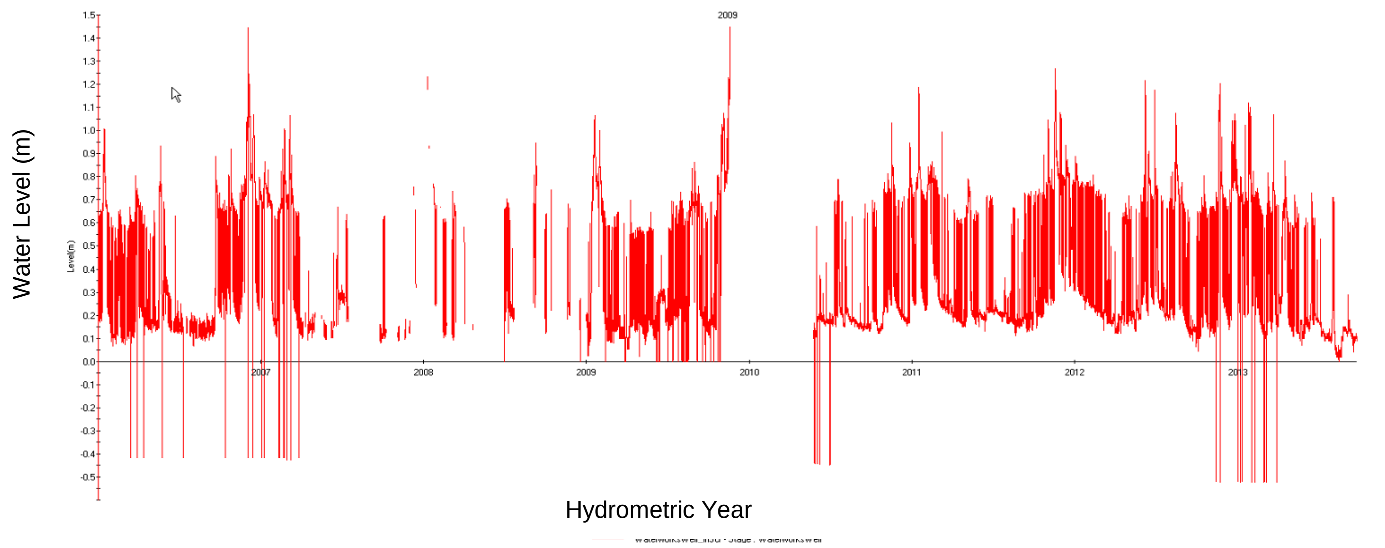
The graph above presents the confidence that has been determined from the available data for 19031 based on the analysis above (comparison with nearby stations and a data availability check). Though recorders have been in place intermittently since 1982, only one year can confirmed as the definite Amax. Based on the Amax data available from ESB from 1982 -1990, 4 years (1983, 1987, 1989 and 1990) appear to have complete records, however these AMaxs cannot be confirmed due to the lack of water level data.

Station		Waterworks
Location:	**165136 , 71400**	
Comparison Stations	**19015 - Healy's Bridge (Approximately 5.2km away)**	
	19045 - Gothic (Approximately 5.3km away)	
Data Availability	**2002 - 2009**	**Amax and Water Level Data**
Gauge Datum	Correspondence received with the flow data indicated that the datum changed on 12/6/12 from 3.19m to 3.262m, however it was also stated that the weir is not even and it is difficult to pinpoint the crest level and that, therefore, another survey could easily give a different datum. From plotting stage data for the Waterworks Weir in the hydrometric database, no marked differences between stage recorded before and after the change in datum was observed. Considering this, and also the difficulty in measuring datum, the original data supplied has been reviewed, and we have **not** adjusted stage before 12/6/12 to correct for the present datum.	

Amax Comparison with Nearby Stations:

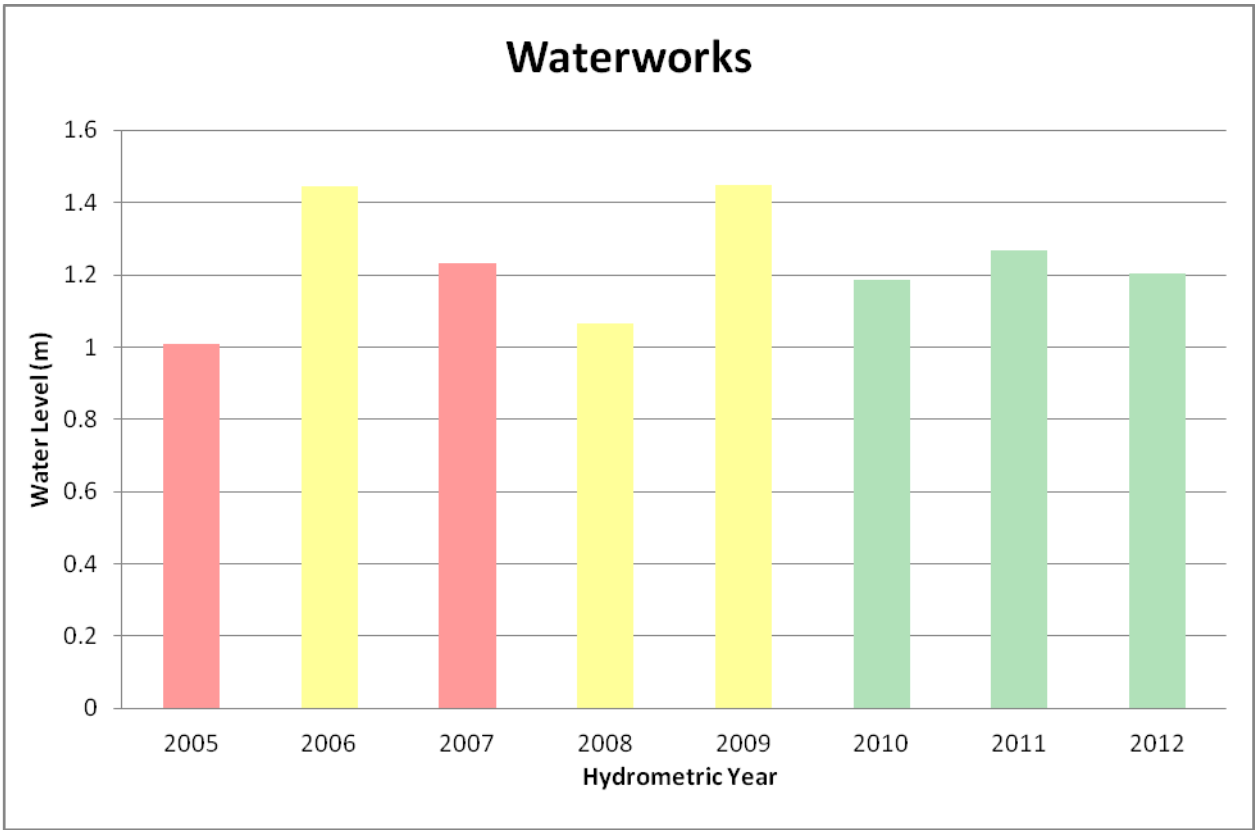
HY	Waterworks	% DATA	Healy's Bridge	% DATA	Gothic	% DATA
2005	**15/01/2006 11:45**	**67%**	03/11/2005 01:00	93%	03/11/2005 20:30	40%
2006	**03/12/2006 13:45**	**65%**	03/12/2006 05:00	26%	03/11/2005 20:30	0%
2007	**10/01/2008 23:45**	**21%**	10/01/2008 10:45	72%	10/01/2008 10:15	97%
2008	**19/01/2009 22:45**	**78%**			31/01/2009 03:00	99%
2009	**19/11/2009 16:45**	**49%**			19/11/2009 17:45	100%
2010	**16/01/2011 16:45**	**98%**	18/02/2011 20:15	21%	17/11/2010 02:30	100%
2011	**19/11/2011 08:45**	**100%**	28/06/2012 07:30	29%	28/06/2012 06:30	92%
2012	**22/11/2012 09:45**	**98%**		1%	25/01/2013 19:30	56%

Data Availability:



Hydrometric Year	Recorded Amax	Missing Data	Notes
2005	15/01/2006 11:45	01/10/2005-01/01/2006, 10/07/2006-18/07/2006, 20/07/2006-25/07/2008	No Winter Months
2006	03/12/2006 13:45	05/04/2007-17/04/2007, 02/05/2007-22/05/2007, 28/05/2007-08/06/2007, 17/07/2007-25/09/2007	Very Scatter Summer Data
2007	10/01/2008 23:45	Very Scattered Data	Insignifcant Data
2008	19/01/2009 22:45	Scattered Data up to 01/01/2009	
2009	19/11/2009 16:45	19/11/2009-25/05/2010	Gauge went down during known flood event
2010	16/01/2011 16:45		Complete data set
2011	19/11/2011 08:45		Complete data set
2012	22/11/2012 09:45		Complete data set

Results:



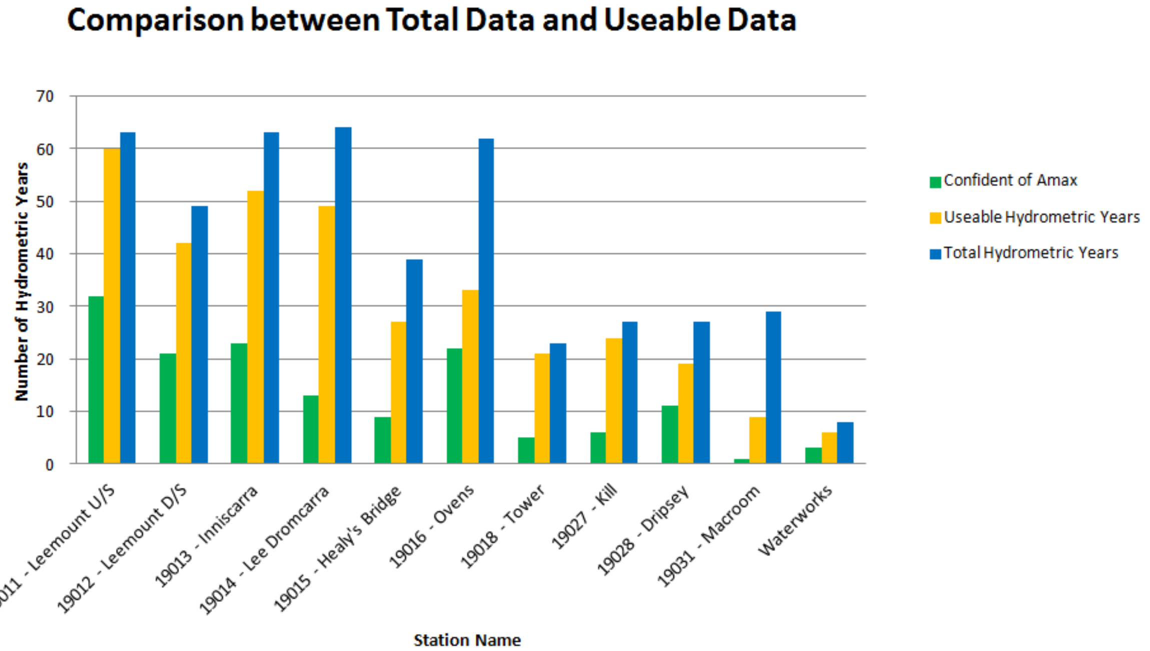
Results and Conclusions:

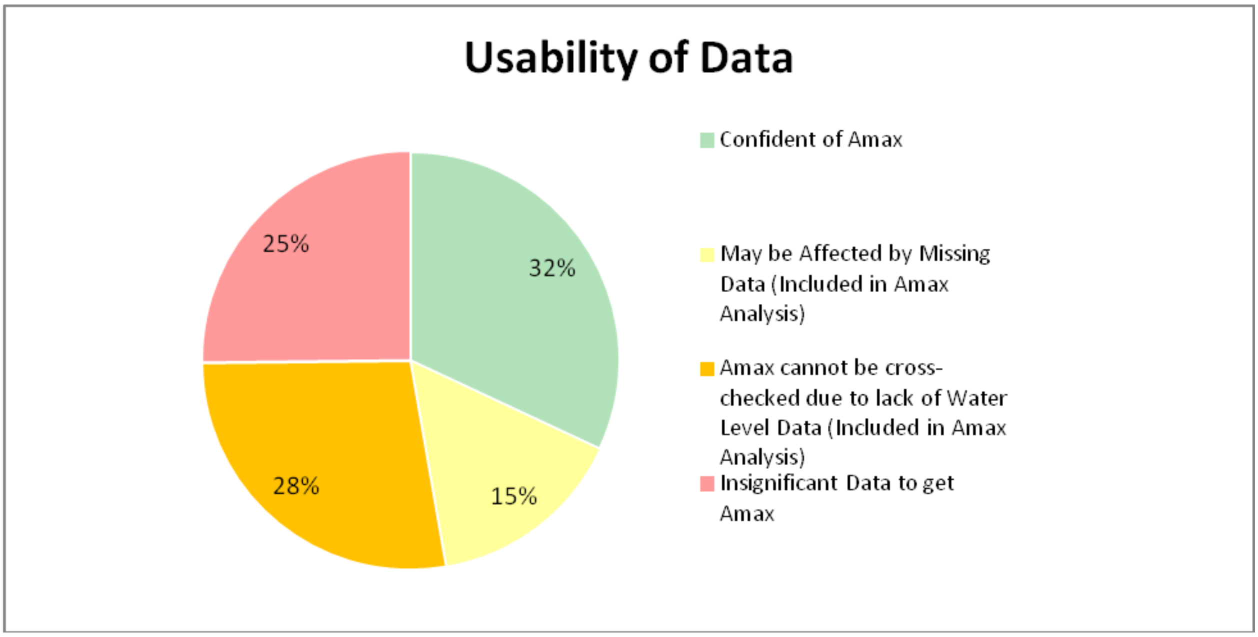
The graph above presents the confidence that has been determined from the available data for Waterworks gauge located at the Salmon Weir on Western Road based on the analysis above (comparison with nearby stations and a data availability check). The gauge was only installed in 2005, in the beginning it had a tendency to go down regularly but this has improved with most recent data set being complete. Of the 8 years of record confidence in the capturing of the Amax is available for three years, the most recent three years.

Summary of Results for Hydrometric Gauges:

The data availability for each of the stations has been determined in the previous sections. For figure below compares the total hydrometric years against the usable hydrometric years for each of the stations. The pie chart over leaf shows the quality of the data available with only 31% (141 of 454 years of data) providing confident Amaxs. The limitations in data availability can be clearly seen. By including all data in a usable record besides the years that have been highlighted red and shown to have insignificant data, it will reduce the certainty of recorders having captured the correct Amax for a hydrometric year however it should improve the calculations of the Qmed by using a longer record for each of the stations.

Station	Total	Confident of Amax	May be Affected by Missing Data (Included in Amax Analysis)	Amax cannot be cross-checked due to lack of Water Level Data (Included in Amax Analysis)	Insignificant Data to get Amax	Useable Data
19011 - Leemount U/S	63	32	5	23	3	60
19012 - Leemount D/S	49	21	2	19	7	42
19013 - Inniscarra	63	23	-	29	11	52
19014 - Lee Dromcarra	64	13	2	34	15	49
19015 - Healy's Bridge	39	9	18		12	27
19016 - Ovens	62	22	11		32	33
19018 - Tower	23	5	16		2	21
19027 - Kill	27	6	1	17	3	24
19028 - Dripsey	27	11	8		8	19
19031 - Macroom	29	1	4	4	20	9
Waterworks	8	3	3		2	6
Total	454	146	70	126	115	342



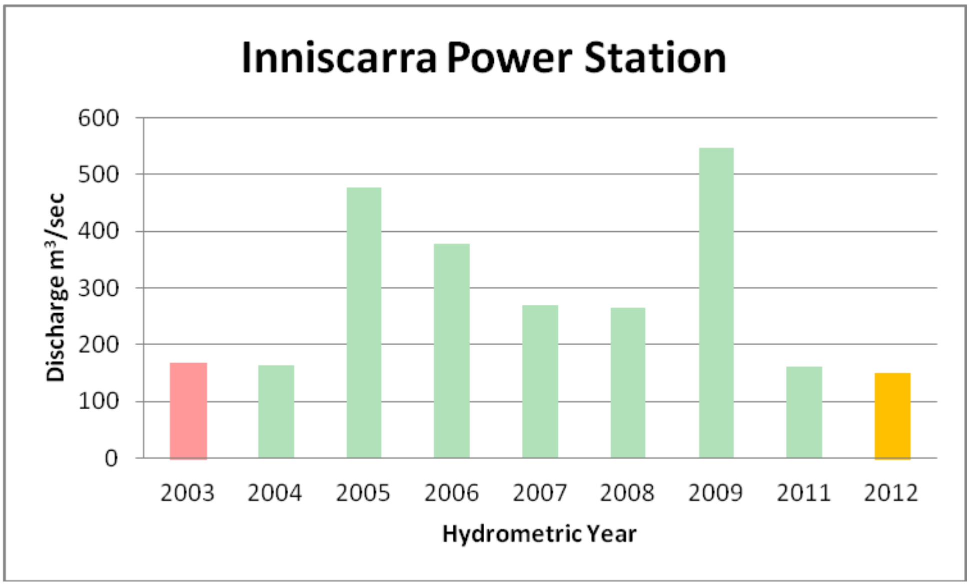

Usability of Data
32%
15%
28%
25%
Confident of Amax
May be Affected by Missing Data (Included in Amax Analysis)
Amax cannot be cross-checked due to lack of Water Level Data (Included in Amax Analysis)
Insignificant Data to get Amax

Reservoir Data Analysis:

There are two reservoirs that control the River Lee at Inniscarra (19093) and Carrigadrohid (19090). Data is available from 2003 and 2004 respectively. The analysis below shows the maximum discharge rates per annum and the percentage data availablility.

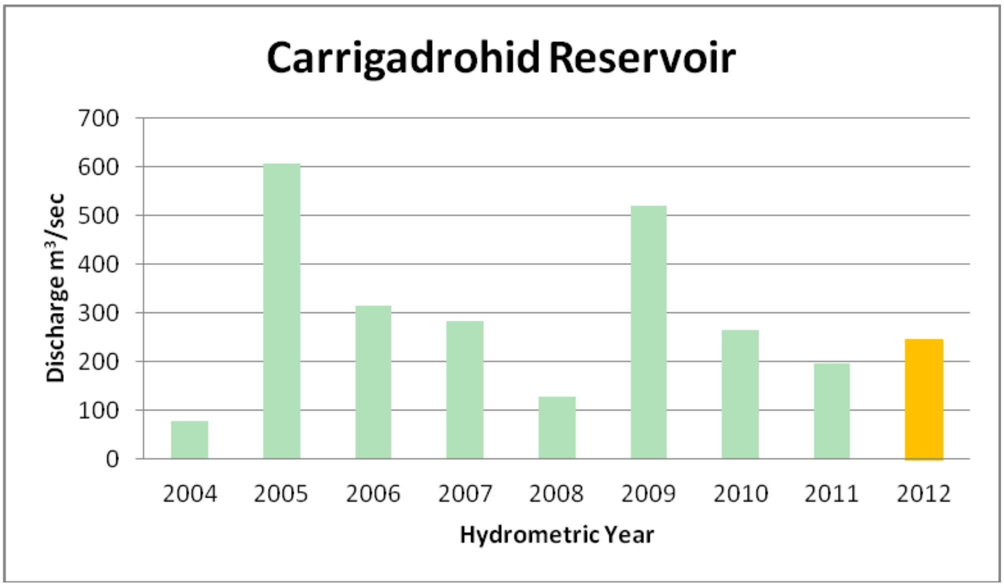
Reservoir	19093 - Inniscarra Power Station
Location:	**154478 , 72243**
Data Availability	**2003-2012**

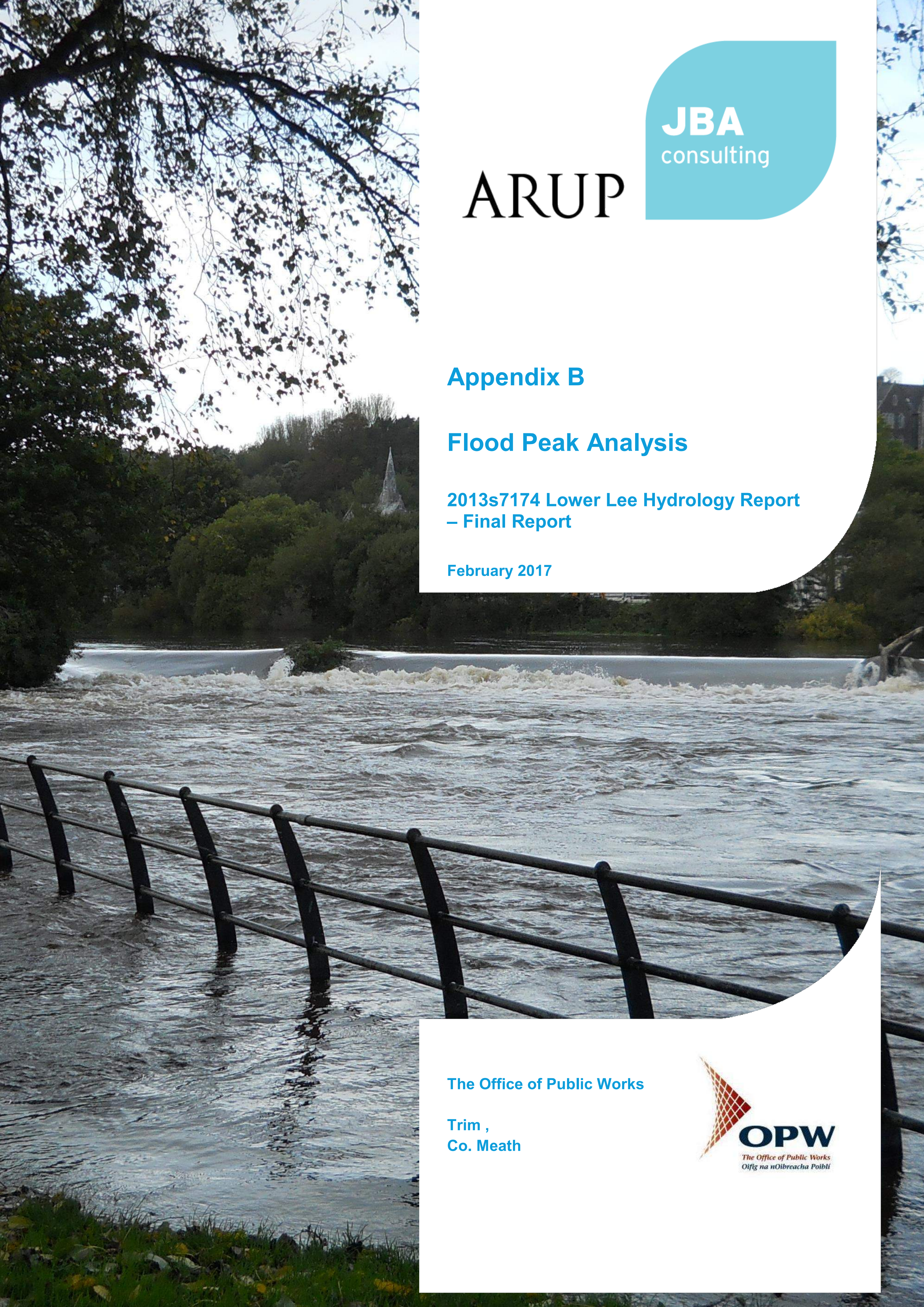
HY	Date	Discharge	Data %
2003	19/05/2004 13:00	167.71	13.60%
2004	29/10/2004 12:00	163.05	99.40%
2005	01/04/2006 16:00	476.79	100.00%
2006	07/10/2006 10:00	379.26	100.00%
2007	07/07/2008 22:00	270.31	100.30%
2008	25/05/2009 12:00	264.91	100.00%
2009	20/11/2009 03:00	548.08	100.00%
2011	18/11/2011 13:00	161.83	100.30%
2012	23/12/2012 16:00	150.96	85.60%



Reservoir	**19090 - Carrigadrohid Reservoir**
Location:	**140597 , 71893**
Data Availabilty	**2004-2012**

HY	Date	Discharge	Data %
2004	29/10/2004 14:00	79.27	100.00%
2005	03/04/2006 23:00	607.461	100.00%
2006	03/12/2006 11:00	315.516	100.00%
2007	10/01/2008 13:00	284.34	100.30%
2008	12/01/2009 18:00	127.074	100.00%
2009	19/11/2009 17:00	518.839	100.00%
2010	02/08/2011 18:00	264.98	100.00%
2011	18/11/2011 19:00	196.72	100.30%
2012	13/11/2012 18:00	246.27	85.60%





ARUP

JBA consulting

Appendix B

Flood Peak Analysis

2013s7174 Lower Lee Hydrology Report – Final Report

February 2017

The Office of Public Works

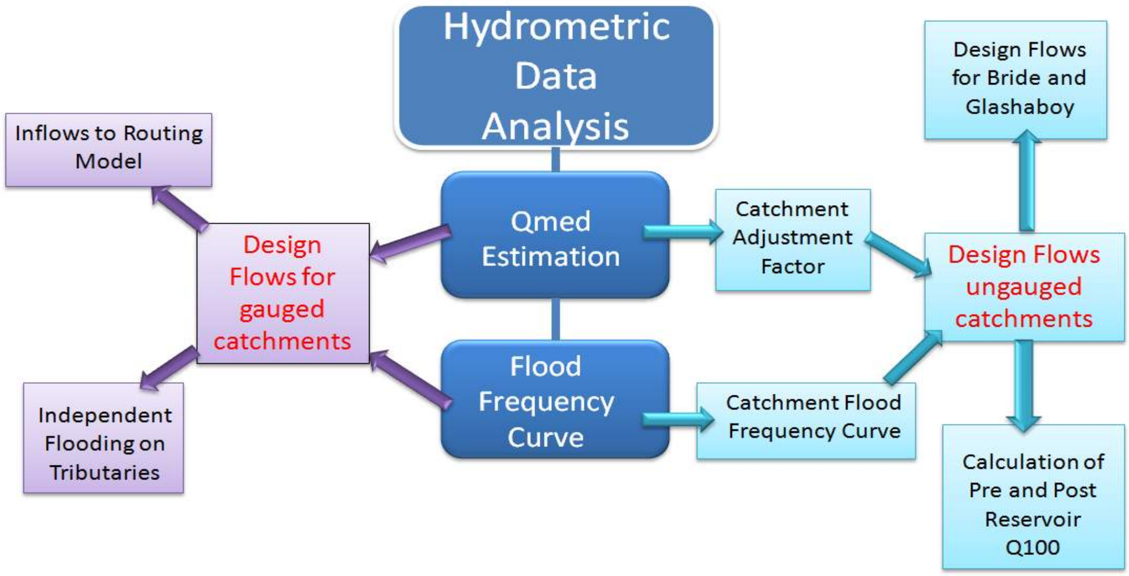
Trim ,
Co. Meath

OPW
The Office of Public Works
Oifig na nOibreacha Poiblí

B Flood Peak Analysis

Though conventional methods cannot be used to determine the design flood for the Lower Lee in Cork they will be used to aid the study and provide the required validation of the continuous simulated model results. The following sections detail the hydrological process undertaken to derive the flows at gauged and ungauged locations. The analysis is focused on maximising the potential accuracy of design flow estimates derived from continuous simulation that will in-turn be used for subsequent hydraulic modelling and to validate flood routing model. Figure B-1 shows a schematic of the analysis carried out based on the hydrometric station data and how it aids the overall study.

Figure B-1: Hydrological Process



The hydrometric data analysis is summarised as:

1. Review of data availability and quality at the hydrometric stations (Outline in Appendix A)
2. Estimation of Qmed at gauging stations using single site analysis, FSU and FSR rainfall runoff methods.
3. A relationship between Qmed determined using single site analysis and Qmed calculated using the FSU regression equation based on catchment characteristics was found at each of the gauged catchments. The average of the ratio between gauged and ungauged Qmed estimation for the gauged tributaries within the catchment was determined to give a catchment Qmed adjustment factor. Qmed for ungauged catchments is calculated using the FSU Qmed regression equation and then adjusted using this Qmed adjustment factor. This technique ensures that all flood estimates are correlated to actual flood flow records.
4. The relationship between the Qmed and other more extreme floods is defined by the growth curve. Flood growth curves were derived from analysis of annual maximum flows either at the site of interest (single-site analysis) or at a group of gauging stations chosen from a wide area (pooled analysis). Growth curves were derived at each of the hydrometric stations using single-site analysis, FSU pooling groups and FSR rainfall runoff methodologies. At each site the most appropriate flood frequency curve was chosen based on analysis of historical events. A catchment preferred flood frequency curve was determined which will be applied to give estimated flows at ungauged locations.
5. The flow estimates for both gauged and ungauged catchments of a particular exceedance probability is then simply calculated as the product of QMED and the value of the growth curve for that probability (known as the growth rate).
6. The flow estimates at the gauged locations will be compared against the inflows generated using the continuous simulation model to help validate the model.

7. Using the FSU Qmed regression equation based on catchment characteristics prior to the construction of the reservoir a Q100 estimate was calculated at the Waterworks Weir pre and post the construction of the reservoirs. The 100 year flood generated using the continuous simulation model is compared against this Q100 calculated using the FSU Qmed regression equation.

B.1 Overview of methods for estimation of Qmed for gauged catchments

Qmed is defined as the flood that is expected to occur or be exceeded, on average, every other year. In statistical terms the flood is said to occur or exceeded on average once every two years and have a 50% probability of annual exceedence. Due to the impact of the reservoir on gauges downstream of the dams only gauges upstream of the reservoir and tributaries that join downstream were analysed. In essence, this study is only concerned with calculating inflow boundaries for the tributaries for the routing model so it is not necessary to analyse the gauges downstream of the reservoir. Qmed has been estimated using the following methods:

- Single Site Analysis
- Flood Studies Update (FSU)
- FSR Rainfall Runoff Methods

Though the Qbar is the standard representation for flood estimation using the FSR rainfall runoff method, which has a return period of 2.33 years, JFes, JBA's web based flood estimation software calculates Qmed to allow comparison with other flood estimation methods. The results for the above estimation techniques were then compared against the Qmed estimate from Lee CFRAM where possible for comparative purposes.

B.1.2 Single Site Analysis to Estimate Qmed

The most reliable estimates of QMED are obtained directly from suitable quality flood peak data, as the median of the annual maximum series. Single site analysis was completed on the useable record for each of the stations.

Details of the rating curves that have been applied to the verified annual maximums are contained in individual gauge summary sheets at the end of this appendix. In summary, Lee CFRAM ratings have been applied to Lee Dromcarra, Macroom, Tower and Ovens and the ESB ratings have been applied to the remaining. Though there are 49 and 33 years of useable annual maximum detailed in Appendix A the validated/applied length of record has been reduced 22 and 26 years respectively due to the difficulty in obtaining a rating curves for Lee Dromcarra prior to 1977 and at the Ovens gauge between 1971-1984.

Results from single site analysis are presented in Table B-1.

Table B-1: Qmed Estimation from Single Site Analysis

Station Name	Qmed (m^3/sec)	Validated/ Applied Length of Record
19014 - Lee Dromcarra	81.51	22
19015 - Healy's Bridge	62.64	27
19016 - Ovens	26.63	26
19018 - Tower	70.14	21
19027- Kill	50.17	24
19028 - Dripsey	40.96	19
19031 - Macroom	148.00	9

B.1.3 Estimation of Qmed using Flood Studies Update

The Flood Studies Update (FSU) method to estimate Qmed as described in research reports produced from FSU work packages 2.2 and 2.3 has been used. Qmed can be estimated using a regression equation based on seven different physical catchment descriptors, in conjunction with an urban adjustment, developed in FSU work package 2.3.

The multivariate regression equation was developed on the basis of data from 199 gauged catchments, linking QMED to a set of catchment descriptors.

$QMED_{rural}=1.237x10^{-5}AREA^{0.937}BFIsoils^{-0.922}SAAR^{1.306}FARL^{2.21}DRAIND^{0.341}S1085^{0.185}(1+ARTDRAIN2)^{0.408}$

Where: AREA is the catchment area (km^2).
BFIsoils is the base flow index derived from soils data
SAAR is long-term mean annual rainfall amount in mm
FARL is the flood attenuation by reservoir and lake
DRAIND is the drainage density
S1085 is the slope of the main channel between 10% and 85% of its length measured from the catchment outlet (m/km).
ARTDRAIN2 is the percentage of the catchment river network included in the Drainage Schemes

Because FSU methods are not fully released for general use at the time of writing, it was necessary to make some decisions about how to apply the methods presented in the reports, and to develop software to enable application of the methods. Qmed was calculated using JBA's web-based flood estimation software, JFes. The results FSU estimation can be seen in Table B-2.

Table B-2: Qmed Estimation from FSU

Station Name	Qmed (m3/sec)
19014 - Lee Dromcarra	81.23
19015 - Healy's Bridge	40.85
19016 - Ovens	21.70
19018 - Tower	33.82
19027- Kill	30.06
19028 - Dripsey	20.14
19031 - Macroom	80.23

B.1.4 Qmed Estimation from Rainfall Run-off Methods

The unit hydrograph method most widely used in Ireland and the UK for ungauged catchments is the FSR triangular unit hydrograph and design storm method. This method estimates the design flood hydrograph, describing the timing and magnitude of flood peak and flood volume (area beneath hydrograph). This method requires the catchment response characteristics (time to peak, tp), design rainstorm characteristics (return period, storm duration, rainfall depth and profile) and runoff / loss characteristics (percentage runoff and baseflow).

The UK Natural Environmental Research Council (1975) carried out a comprehensive flood study involving a large number of catchments from throughout Britain including many Irish catchments. The unit hydrograph prediction equation was derived from 1,631 events from 143 gauged catchments (the hydrograph method only included one Irish catchment) ranging in size from 3.5 to 500km2. The result was a triangular Unit Hydrograph described by the time to peak Tp of the catchment derived from catchment characteristics. The instantaneous triangular unit hydrograph is defined by a time to peak Tp, a peak flow in cumecs/100km^2 Qp = 220/Tp and a base length TB = 2.52Tp.

The FSR rainfall-runoff method relies on rainfall frequency statistics to provide inputs to a model that converts rainfall to runoff. The rainfall-runoff model separates a flood hydrograph into a baseflow component and a rapid runoff component. The rapid runoff is found by estimating the component of rainfall that contributes to runoff (the effective rainfall), and converting the effective rainfall to flow by use of a unit hydrograph. The unit hydrograph describes the theoretical response of the catchment to an input of a unit depth of rainfall over a unit of time.

The steps in the model are:

- Determine the parameters of the unit hydrograph, either from flood event data or from catchment characteristics;
- Determine the percentage runoff to convert total rainfall to effective rainfall;
- Construct the design storm by determining its duration, depth and profile;
- Combine the effective rainfall profile with the unit hydrograph by convolution to give the flood hydrograph;

- Add baseflow to the flood hydrograph.

Parameters to carry out the rainfall-runoff method were estimated using catchment characteristics. The rainstorm profile used in this analysis is the FSR 75% Winter profile and 50% Summer profile for Ireland. The unit hydrograph describes the theoretical response of the catchment to an input of a unit depth of rainfall over a unit of time. Table B-3 provides results from the FSR RR method at the gauged catchment locations.

Table B-3: Qmed Estimation from FSR Rainfall Runoff

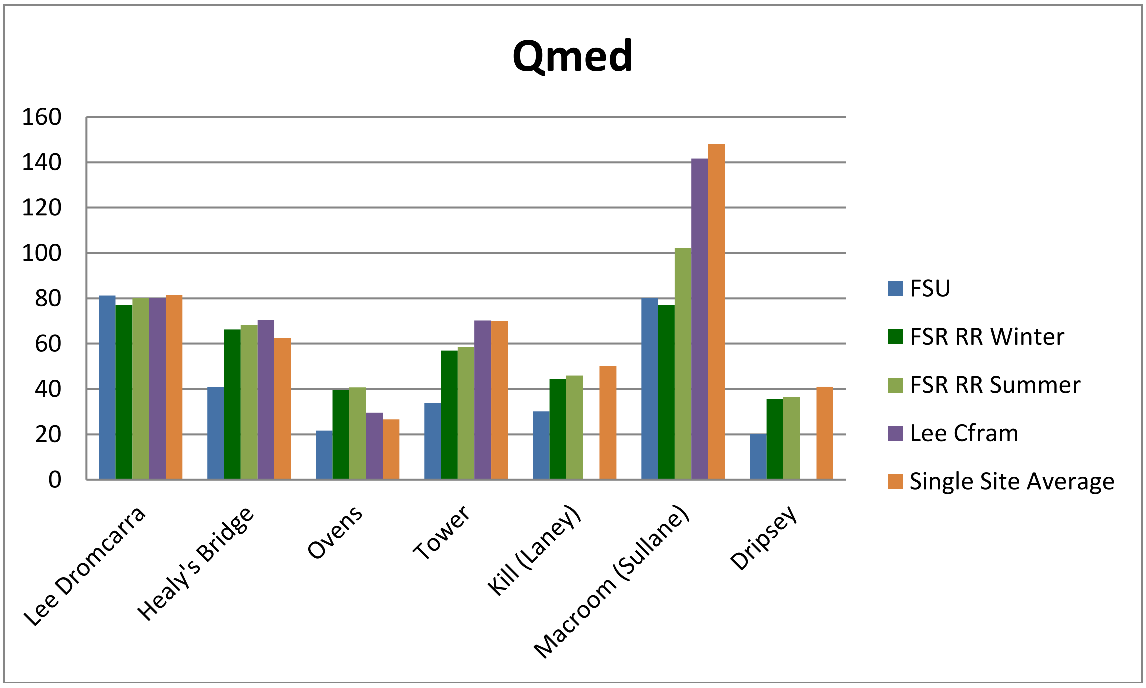
Station Name	FSR RR Winter	FSR RR Summer
19014 - Lee Dromcarra	76.96	80.08
19015 - Healy's Bridge	66.33	68.26
19016 - Ovens	39.57	40.65
19018 - Tower	56.99	58.50
19027- Kill	44.36	45.99
19028 - Dripsey	35.48	36.49
19031 - Macroom	76.96	102.17

B.1.5 Comparison of Methods

Figure B-2 shows the results of the Qmed Estimation from the various different methods. It is evident at all gauging stations, with the exception of Lee Dromcarra, that the FSU estimate is significantly lower than the single site estimate and is a poor representation of the gauged estimate. Lee Dromcarra gauging station has a low FARL value, due to the presence of Lough Allua and this has an attenuating effect on the flows.

The rainfall runoff estimates generally fit well with the single site analysis, with the exception of Ovens which is affected by karst and Macroom which as previously discussed suffers from data shortages. As FSU is the preferred means of flood estimation in Ireland for ungauged catchments, it has been chosen to calculate the median flows for the tributaries. It will, however, need to be adjusted by a catchment adjustment factor to bring the low FSU estimates in line with single site analysis and provide a truer representation of median flows.

Figure B-2: Comparison of Qmed Estimation



B.1.6 Calculation of a Qmed catchment adjustment factor

It is possible to improve on the initial estimate of Qmed by refining it using the process of data transfer, in which a representative gauged catchment with suitable quality data is identified and an adjustment factor for Qmed calculated as the ratio of the gauged to the ungauged estimate of Qmed at the gauging station. This factor is then used to adjust the initial estimate of Qmed at an ungauged site or gauging site with poor data records, under the assumption that the factorial error in the Qmed regression model is similar for two catchments. In the terminology of the FSU research reports, the gauging station where the adjustment factor is calculated is referred to as a donor site.

Table B-B-4 shows the results from the different Qmed estimation techniques, as plotted in Figure B-2B-2. The gauges have been classified according to catchment type.

Table B-4: Summary of Qmed

	FSU	FSR RR Winter	FSR RR Summer	Lee Cfram	Single Site Average	Adjustment Factor (Single Site/FSU)
Lake Influence						
Lee Dromcarra	81.228	76.96	80.08	80.22	81.51	1.00
Karst Influence						
Ovens	21.701	39.57	40.65	29.5	26.63	1.23
Tower	33.819	56.99	58.5	70.2	70.14	2.07
Excluded Catchments						
Macroom (Sullane)	80.225	76.96	102.17	141.7	148	1.84
Standard Catchment						
Kill (Laney)	30.061	44.36	45.99		50.17	1.67
Dripsey	20.137	35.48	36.49		40.96	2.03
Healy's Bridge	40.848	66.33	68.26	70.5	62.64	1.53
						1.75

The gauges have been classified according to catchment type.

(a) Lee Dromcarra which is influenced by Lough Allua

Lee Dromcarra is influenced by the Lough Allua and this has the effect of lowering the adjustment factor as the recorded Qmed is lower due to the effect of lake attenuating the flows.

(b) Catchments that are potentially influenced but Karst geology

When an annual maximum series plot of the recorded record at Ovens is analysed it was found that the karst influence attenuates the peak. At a certain point the groundwater influence is overcome and its flow values rise rapidly in more extreme events. This is different to the expected normal distribution of an annual maximum series in Ireland.

Tower gauge is also affected by a karst influence. At present discrepancies exist between Tower gauge and Healy's Bridge gauge, with Tower, a subcatchment of Healy's Bridge registering higher flow for the same event at Tower than Healy's Bridge. Healy's bridge has been calibrated using a recorded flow for the November 2009 event. There are a number of issues with the Tower gauge including its location upstream of the bridge with the effects of the bridge difficult to model and a lack of high flow gaugings. In the location of the tower gauge there are large floodplains that once inundated lead to a small rise in levels but a large rise in flows resulting in a rating that is very sensitivity to small changes in level. As a result of the confidence in the Healy's Bridge it has been included in the analysis and Tower has been excluded.

(c) Excluded Catchments

I large degree of uncertainty remains at Macoom and has there been excluded from the analysis. The limited data record, change in gauge location, the exclusion of the River Laney that joins the Sullane just upstream of the Macroom gauge in the development rating curve for the Macroom and a lack of flow gaugings has lead to its exclusion from the analysis.

(d) Standard tributaries

At the remaining catchments (Kill, Dripsey and Healy's Bridge) a Qmed adjustment factor was found to average 1.75 as shown in Table B-4. These three remaining stations were then weighted according to their record length to give a weighted catchment adjustment factor of 1.71 as shown in Table B-5 and this will be carried forward and applied to ungauged catchments further downstream and gauges with poor data records.

Table B-5: Weighted Catchment Adjustment Factor

Weighted Average				
	Years of Data	Weight	Adjustment Factor	
Kill (Laney)	24	0.34	1.67	0.57
Dripsey	19	0.27	2.03	0.55
Healy's Bridge	27	0.39	1.53	0.59
	70	Catchment Adj. Factor		1.71

B.1.7 Qmed estimation for the tributaries

Table B-6 shows the final Qmed estimation for the tributaries. A catchment adjustment factor of 1.71 has been applied to the FSU estimate for the ungauged tributaries, whilst the single site estimate has been taken for single site estimate has been taken at gauged locations.

Table B-6: Qmed Estimation for tributaries

Gauged Tributaries		
Station Name	**Qmed**	
19014 - Lee Dromcarra	81.51	
19015 - Healy's Bridge	62.64	
19016 - Ovens	26.63	
19027 - Kill	50.17	
19028 - Dripsey	40.96	
19031 - Macroom	148.00	
Ungauged Tributaries		
Location	**FSU**	**Qmed**
Blackpool - Orchard Court (167409, 73542)	7.63	13.04
Curraheen (162787,70649)	10.42	18.04
Glasheen (165300,69010)	2.45	4.24

B.2 Overview of methods for determination of flood frequency curve

The method for estimation of peak flows using an index flood method involves two stages. The first stage of the method involves estimating Qmed and in the second stage a flood growth curve is estimated. The growth curve is a dimensionless version of the flood frequency curve which defines how the flood magnitude grows as the probability reduces, i.e. for more extreme design floods. The design flood for a particular exceedance probability is then simply calculated as the product of Qmed and the value of the growth curve for that probability (known as the growth rate).

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

B.2.1 FSU using Pooled Analysis

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

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

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

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

Initially, no alterations were made to the pooling groups defined using the process defined above. Gauging stations had already been screened according to the quality of their flood peak data, as described above. Although there is some evidence from research on UK data[1] that flood growth curves are affected by additional catchment descriptors such as FARL, the FSU research found that FARL was not a useful variable for selection of pooling groups (uncertainty was greater when FARL was included than when it was excluded) and therefore no attempt was made to allow for the presence of lakes in the composition of pooling groups. Similarly, no allowance was made for arterial drainage in selecting pooling groups.

The contents of each pooling group created at the site of gauging stations are listed in gauge summary sheets. Most groups can be seen to contain gauges from a wide range of locations across Ireland, although there are few from the east coast, where the annual rainfall is low enough to exclude most gauged catchments from pooling groups created using characteristics of catchments in the Lower Lee.

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

Selection of statistical distribution

For pooled growth curves, WP 2.2 recommends considering 3-parameter distributions, because the extra data provided by the pooling group ensures that the standard error is lower than it would be for single-site analysis. The report states that either the generalised extreme value (GEV) or generalised logistic (GL) distributions are worth considering. For this study, GEV has been fitted for each pooled analysis. In the Lee CFRAM study, GEV was also found to be the most appropriate distribution. This finding is consistent with research carried out for the FSU.

Pooled flood growth curves have been fitted using the method of L-moments, as recommended in the FSU research. To calculate the pooled curve, the L-moments for each gauge in the pooling group have been weighted according to the record length of the gauge. This ensures that more weight is given to longer records, which provide more reliable estimates of the underlying flood frequency distribution. Results of the FSU growth curves can be found in for each of the hydrometric gauges in the individual gauge summary sheets. Figure B-3 shows the FSU growth curves for the gauges uninfluenced by the reservoir, needed for this study and the growth factors are listed in Table B-6.

Figure B-3: FSU Growth Curves

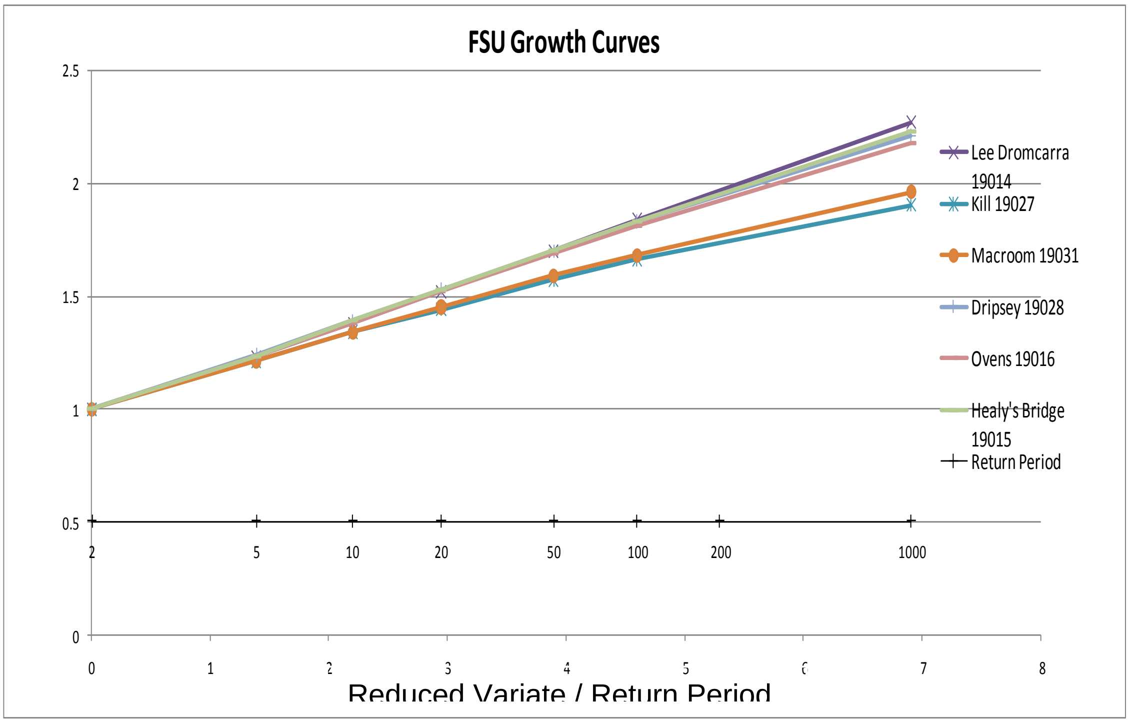


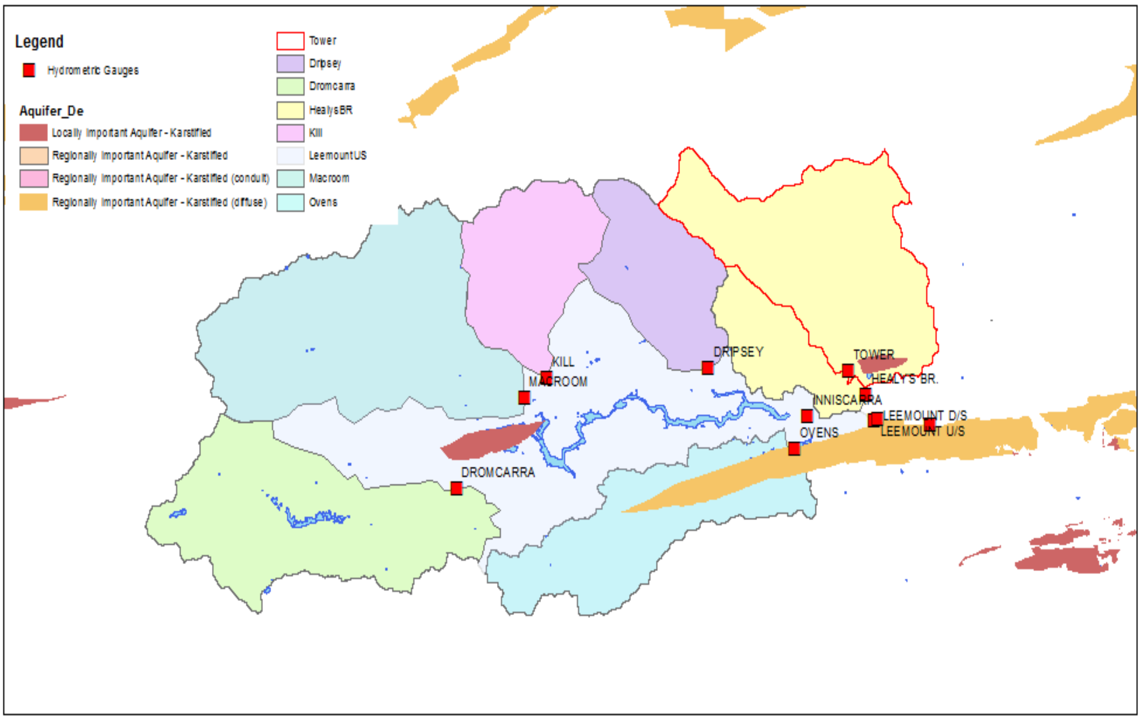
Table B-6: FSU Growth Factors

Return Period	Lee Dromcarra	Healy's Bridge	Ovens	Kill	Dripsey	Macroom
2	1	1	1	1	1	1
5	1.23	1.23	1.23	1.21	1.24	1.21
10	1.38	1.39	1.38	1.34	1.39	1.34
25	1.52	1.57	1.52	1.48	1.53	1.45
50	1.7	1.7	1.69	1.57	1.7	1.59
100	1.84	1.83	1.81	1.66	1.83	1.68
1000	2.27	2.23	2.18	1.9	1.90	1.96

B.2.2 FSU growth factors adjusted for Karst/Arterial Drainage

Figure B-4 shows the catchment areas overlaid on Geographical Survey of Ireland Map of groundwater's aquifer map. It highlights that the Ovens catchment is influenced considerably by karst geology.

Figure B-4: Karst areas within the Lower Lee Catchment



Pooling groups automatically selected as discussed in 0, were analysed to consider the affect of karst catchments and significant arterial drainage works. Each gauge in the pooling group was checked and any gauge with greater than 10% karst based on the Geological survey of Ireland Groundwater Aquifers Map were removed, along with any gauge with greater than 10% arterial drainage. This was completed for five of the six tributaries. As Ovens is significantly influenced by karst only gauges affected by arterial drainage were removed.

Table B-B-7 below outlines the results of the analysis. Three of the stations (Lee Dromcarra, Ovens and Macroom) suggest very little change. There is a reduction in the slope of the growth curve for Healy's Bridge and Dripsey, but an increase in slope for the Kill gauge. The results of the analysis does not produce any significant patterns. The selection of pooling groups based on eliminating karst and arterial drainage affected catchments lead to the exclusion of gauges that, based on FSU research, have been found to be most appropriate for pooling. For this reason, FSU growth factors based on the original analysis have been taken forward for use.

Table B-7: FSU Growth Factors accounting for karst and arterial Drainage

Return Period	Lee Dromcarra	Healy's Bridge	Ovens	Kill	Dripsey	Macroom
2	1	1	1	1	1	1
5	1.25	1.24	1.23	1.24	1.23	1.22
10	1.41	1.39	1.38	1.39	1.36	1.34
20	1.55	1.56	1.52	1.51	1.51	1.45
50	1.73	1.67	1.69	1.67	1.61	1.58
100	1.86	1.77	1.81	1.77	1.69	1.67
1000	2.25	2.05	2.18	2.06	1.74	1.91

B.2.3 FSR Rainfall Runoff Growth Factors

The design rainstorm duration is obtained from the FSR formula D = (1 + 0.001SAAR)Tp. Using the prescribed FSR rules for computing the storm duration, profile and percentage runoff a 140year return period design storm is required to produce the 100year design flood. The corresponding design rain storm in Table were used in order to generate the FSR rainfall runoff growth curve.

Table B-8, Figure B-5: FSR Rainfall Runoff Growth Curves and Table B-9 show the results from the analysis carried out on JBA's Flood Estimation Software (JFes).

Table B-8: FSR Design Rain Storms

Flood	Rain
2.33	2
5	8
10	17
20	35
30	50
50	81
100	140
250	300
500	520
1000	1000

Figure B-5: FSR Rainfall Runoff Growth Curves

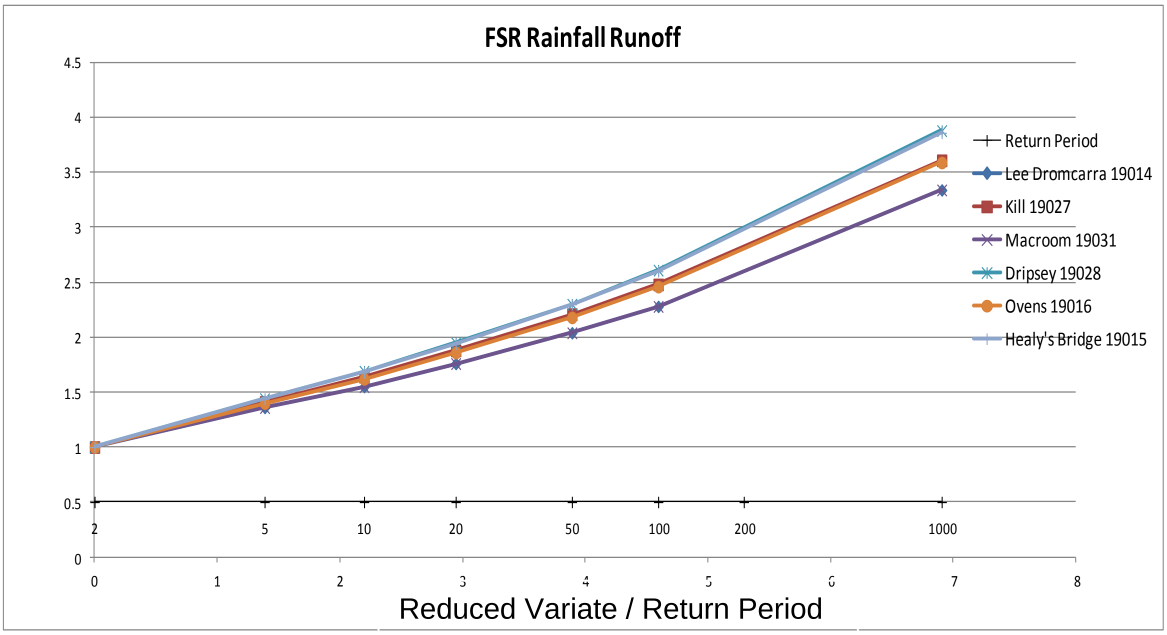


Table B-9: FSR Rainfall Runoff Growth Factors

Return Period	Lee Dromcarra	Healy's Bridge	Ovens	Kill	Dripsey	Macroom
2	1	1	1	1	1	1
5	1.36	1.44	1.4	1.41	1.44	1.39
10	1.55	1.68	1.62	1.64	1.69	1.60
25	1.81	2.01	1.86	1.94	2.02	1.89
50	2.04	2.3	2.18	2.21	2.3	2.14
100	2.28	2.6	2.46	2.48	2.61	2.40
1000	3.23	3.86	3.59	3.61	3.88	3.47

B.2.4 Extension of FSU growth curves to the 1000-year return period (0.1% AEP)

When historical events were analysed against the growth curve generated from FSU methodologies it was found that the highest couple of floods recorded at all sites have annual probabilities lower than 1% (i.e. more extreme). While this is theoretically possible, it is highly unlikely, and a more likely explanation would be that the pooled growth curve underestimates the true growth curve for the catchment in question.

Similarly, on reviewing flood outlines produced during the Western CFRAM using initial estimates of design flow based on FSU methodologies under estimates of extents was found. Some revisions to design flows were made in order to ensure flood levels and extents were not underestimated for the most extreme events. The initial flood outlines showed little out-of-bank flow in some areas even for the 1000-year flood, which was considered unlikely to be realistic. The revisions included applying the FSR rainfall-runoff method to estimate the gradient of the upper portion of the growth curve.

The reasons for preferring the rainfall-runoff method are that rainfall growth curves can generally be treated with more confidence than flood growth curves (owing to longer records, greater spatial consistency and fewer problems with data quality) and that adopting this method avoids the extremely low gradient growth curves that were derived at the hydrometric gauges using the FSU methods. At some gauges, the 1000-year flood was initially estimated to be as little as 14% greater than the 100-year flood. While there is no firm evidence on which to base estimates of floods as extreme as the 1000-year return period, this small growth rate was considered to be unrealistic. The corresponding percentages estimated from the FSR rainfall-runoff method did not fall below 45% (i.e. the 1000-year flood was at least 1.45 times greater than the 100-year flood).

In UK practice it is also common to see occasional very low rates of growth from 100-year to 1000-year floods, and a widespread approach is to derive the upper part of the flood growth curve from an alternative method, usually the ReFH rainfall-runoff method. Environment Agency guidelines[2] advocate this approach, and selection of the 100-year return period as a pivot point is near-ubiquitous in the UK. For this study, initially, a pivot point of the 100-year was chosen; however, when the results were compared against historical events it was found to be still underestimating the true growth curve.

In this study, a pivot point of the 50-year return period was analysed to try to generate more realistic results. The 50-year return period was chosen as a similar pivot point to that used in the Lee CFRAM and the results are shown in Table B-10. The extension of the growth curves was carried out by using the FSR rainfall-runoff flood frequency curve to estimate the ratios of the 50-year to 100-year and 100-year to 1000-year floods. These were then multiplied by the FSU estimate to give an adjusted estimation for the 100 and 1000 year event.

Table B-10: FSU growth factors adjusted from 50 yr pivot point

Return Period	Lee Dromcarra	Healy's Bridge	Ovens	Kill	Dripsey	Macroom
2	1	1	1	1	1	1
5	1.23	1.23	1.23	1.21	1.24	1.21
10	1.38	1.39	1.38	1.34	1.39	1.34
25	1.52	1.57	1.52	1.48	1.57	1.45
50	1.7	1.7	1.69	1.57	1.7	1.59
100	1.9	1.92	1.91	1.76	1.93	1.78
1000	2.69	2.85	2.78	2.56	2.87	2.58

B.2.5 Single Site Analysis

The statistical analysis of the annual maximum series at each of the gauging stations may provide a valuable check on the performance of other methods of flood estimation, and can be used to cautiously assist in the determination of appropriate growth curves.

2 Environment Agency (2012) Flood Estimation Guidelines.

FSU WP 2.2 recommends considering two parameter distributions for single-site growth curves, either the extreme value type 1 (EV1, known as the Gumbel) or the 2-parameter log-normal distribution (LN2). Restricting the number of parameters to two helps reduce the standard error of the fitted distribution, albeit at a cost of a potential greater bias compared with 3-parameter distributions. In this assessment, both distributions have been fitted, and the goodness-of-fit assessed visually. For some of the gauges the data did not plot sufficiently well to two parameter distributions and it was necessary to consider 3- parameter distribution. Generalised extreme value (GEV), generalised logistic (GL) and the 3 parameter log -normal distributions were applied. The most suitable distribution was chosen based on a visual assessment. The affect of the lack of data is evident with some stations plotting poorly to all distributions. The summary sheets show results of single site analysis. GEV was found to be the preferred plotting position. The results are shown in Table B-11 and Figure B-6. It is evident that the growth curves for the catchments follow a similar shape with the exception of Ovens, which is clearly affected by karst influence.

Table B-11: Single Site Analysis Growth Factors

Return Period	Lee Dromcarra	Healy's Bridge	Ovens	Kill	Dripsey	Macroom
2	1.00	1.00	1.00	1.00	1.00	1.00
5	1.44	1.42	1.52	1.32	1.47	1.41
10	1.78	1.70	1.94	1.56	1.81	1.74
20	2.15	2.06	2.65	1.89	2.27	2.24
50	2.58	2.32	3.34	2.16	2.64	2.69
100	2.99	2.59	4.19	2.46	3.03	3.20
200	3.46	2.85	5.27	2.78	3.48	3.80
500	4.14	3.20	7.11	3.25	4.02	4.74

Figure B-6: Single Site Growth Curves